ELEMENT RELIABILITY OF STRUCTURE BASED ON HASOFER-LIND RELIABILITY INDEX

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IN

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Submitted by:

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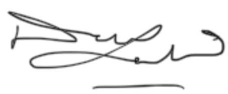
I, Deevit Deepak Sahni, 2K21/STE/09, student of MTech Structural Engineering, hereby declare that the project Dissertation titled "ELEMENT RELIABILITY OF STRUCTURE BASED ON HASOFER-LIND RELIABILITY INDEX" which is submitted by me to the Department of Civil Engineering, Delhi Technological University, Delhi in partial fulfilment of the requirement for the award of the degree of Master of Technology, is original and not copied from any source without proper citation. This work has not previously formed the basis for the award of any Degree, Diploma Associateship, Fellowship or other similar title or recognition.



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CERTIFICATE

I hereby certify that the Project Dissertation titled "ELEMENT RELIABILITY OF STRUCTURE BASED ON HASOFER-LIND RELIABILITY INDEX" which is submitted by Deevit Deepak Sahni, 2K21/STE/09 Department of Civil Engineering, Delhi Technological University, Delhi in partial fulfillment of the requirement for the award of the degree of Master of Technology, is a record of the project work carried out by the students under my supervision. To the best of my knowledge this work has not been submitted in part or full for any Degree or Diploma to this University or elsewhere.

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ABSTRACT

The deception in a system of structure in civil engineering often arises from uncertain conditions and various types of failures, including design, temporary, and failures caused by natural calamities. To address these challenges, engineers employ the art of formulating mathematical models that can answer questions related to the probability of a structure behaving in a specific way. These models take into account the randomness or incomplete knowledge of material properties, geometric dimensions, as well as the uncertainties associated with the loads and actions acting on the structure.

Reliability analysis extends the traditional deterministic analysis of structures, which assumes known and fixed parameters, by considering the uncertainties present in these parameters. It involves developing mathematical models that can provide insights into how a structure will behave when all material as well as geometric properties, and actions are uniquely defined. By incorporating probabilistic models for the uncertain variables, namely material strengths, dimensions, and loads, engineers can quantitatively assess the likelihood of failure or desired behaviour for a given structure.

The objective of reliability analysis is to gain a deeper understanding of how a structure will perform under uncertain conditions. By evaluating the reliability of a structure, engineers can make informed decisions regarding design choices, risk mitigation strategies, and maintenance planning to ensure the safety and performance of civil engineering structures.

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I would like to express my sincere appreciation and gratitude to everyone who has contributed to the successful completion of this project. Although it may not be possible to name everyone who has helped me along the way, I would like to recognize some individuals for their exceptional support and guidance.

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CHAPTER 1

INTRODUCTION

1.1 General

The deception of structural systems in civil engineering often arises from uncertain conditions and various types of failures, including design failures, temporary failures, and failures caused by natural hazards. To address these challenges, engineers employ the art of formulating mathematical models that can answer questions related to the probability of a structure behaving in a specific way. These models take into account the randomness or incomplete knowledge of material properties, geometric dimensions, and properties, as well as the uncertainties associated with the loads and actions acting on the structure.

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The objective of reliability analysis is to gain a deeper understanding of how a structure will perform under uncertain conditions. By evaluating the reliability of a structure, engineers can make informed decisions regarding design choices, risk mitigation strategies, and maintenance planning to ensure the safety and performance of civil engineering structures.

The outcomes can assist in assessing the reliability of a structure, ensuring that it has adequate load carrying capacity under a specific load configuration, even when considering minute details.

Modern software tools are available to investigate the reliability of structures, and one such software used in this project is called COMREL.

Essentially, any deviance from the maximum load parameter value or the load carrying capacity value of a structure, expressed in terms of a load parameter value in extreme conditions, raises concerns about the structure's safety. The analysis aids in determining the "minimum increase in the ultimate load value, with respect to the maximal load parameter evaluated with the highest level of confidence, that should be considered in the carrying capacity model to ensure that the structure will not face failure under normal service conditions, or at least, that the risk of failure is extremely low."

1.2 Objectives and basis of study

Following are the prime objectives:

- To evaluate elemental forces of an L-shaped asymmetrical structure by performing non-linear Time History Analysis using ETABS.
- To check the reliability of an L-shaped asymmetric structural element by using the Hasofer-Lind method in COMREL.

CHAPTER 2

LITERATURE REVIEW

Structural reliability analysis and design have been a topic of great interest to numerous scholars and researchers for a considerable period of time. Various approaches, analysis techniques, and design techniques have been developed and studied in this context. For this project, guidance was sought from the researches of well-known scholars in the field, whose works are discussed in detail in the following section.

Armen Der Kiureghian (2000), FORM and SORM were used to analyse the geometry of unpredictable seismic oscillations and their solutions. It looks into the issues with seismic oscillations that arise from discretizing the input process in the standardised normal random variable space. Simple geometric shapes are visible in linear systems under Gaussian excitation. In contrast, non-Gaussian responses display more complex, nonlinear geometric shapes. To solve these problems approximately, the first and second-order methods of reliability (FORM and SORM) are utilized.

Armen Der Kiureghian and Pei-Ling Liu (1986), proposed a thorough framework for doing analysis of first-order structural reliability when taking in consideration the missing probability data. The authors present a method that integrates partial probability information on uncertain variables beyond the subsequent moments in order to meet the requirements of coherence, invariance, operability, and simplicity. This comprises bounds, higher moments, partial joint distributions, and marginal distributions. The suggested approach is in line with Ditlevsen's generalised reliability index philosophy and is meant to supplement first and second-moment, and full-distribution of the reliability of the structure theories. The authors present new findings for joint distribution models with known marginals, which have broad applications in the fields of probability and statistics.

Chandra S. Putcha (1984), The majority of research papers in the field of reliability focus on assessing the probability of failure for different limit states or calculating the safety index (β). Conversely, only a limited number of papers have delved into the inverse formulation of reliability. The current study adopts a novel strategy by focusing on the development of different reinforced concrete components in order to achieve predetermined probabilities of failure. The reliability design issue is turned into a polynomial equation defined as a function of the design parameters, and probability-of-failure levels for various members are taken from the literature. Standard techniques are then utilized to solve the polynomial, producing the design values for the members that meet the targeted probability's failure. This methodology can be applied to any RC member and has practical relevance in the field of engineering.

A. Neuenhofer and A. Der Kiureghian (1992), introduced a novel response spectrum technique for analyzing seismic activity in linear multi-degree-of-freedom of structures with multiple supports, which are exposed to ground motions that vary spatially. The suggested approach takes into account a number of variables, including local soil conditions, wave passage, and loss of coherence with distance. It is based on the core ideas of randomised vibration theory and takes into account the correlations between the motions of the supports as well as other structural modes.

R. Ranganathan (1999), In practical engineering, decision-making processes often involve uncertainties that cannot be completely eliminated. These uncertainties manifest in various parameters encountered during analysis and design, making it impossible to ensure absolute safety. A rational criterion, proposed more than 25 years ago, addresses this challenge by considering the reliability or probability of survival of structures as a measure of their safety. The complement of reliability in reliability of structure, the chance of failure, statistically evaluates the safety of the structure. Probabilistic ideas are used in structural design and reliability analysis. It is now feasible to assess the degree of dependability for existing structures built in accordance with accepted criteria by using structural reliability theory.

CHAPTER 3

SUMMARY OF PROBABILISTIC VARIABLES

3.1 General

The typical deterministic design technique makes the assumption that there are no probabilistic fluctuations in any of the parameters. The loads placed on structures, such as living loads, wind loads, ocean waves, earthquakes, etc., are commonly understood to be random variables. Likewise, the material strengths (e.g., concrete, steel) and geometric parameters (e.g. section dimensions, effective depth, bar diameter) are subject to statistical variations. Therefore, in order to accurately assess the structural safety, it is necessary to consider the stochastic nature of these fundamental parameters. The structural safety is a statistical number since both the loads and the strengths are arbitrary variables.

To account for uncertainties in design parameters, a safety factor is incorporated by selecting the lowest strength value and the highest load value. This cautious methodology guarantees safety in the design process and ultimately yields cost-effective results.

3.2 Mean and Variance

The Sample mean serves as the ideal statistical measure to quantify the central value of a random variable. It effectively summarizes the distribution and represents the centre of gravity for the given data.

$$\bar{x} = \frac{1}{n}\sum_{i=1}^{n} x_i$$

The variability or spread of a dataset is a crucial aspect that characterizes the data. This dispersion can be exhibited using the sample variance, which can be further calculated as

$$s^2 = \frac{1}{n}\sum_{i=1}^{n} (x_i - \bar{x})^2$$

3.3 Probability Density Function

When examining a random function $x(t)$, the assessment involves analyzing the measured values and evaluating the corresponding time intervals between them. This analysis leads to the calculation of a ratio, which is determined as follows:

$$P(X_1 \leq X \leq X_2) = \frac{\Delta t_1 + \Delta t_2 + \cdots \Delta t_n}{T}$$

Furthermore, the probability distribution function P(X) offers the likelihood of X being within the range of values X1 and X2 during the random process. Similarly, the probability of $X(t)$ being smaller than a given value of X is represented as

$$P(X) = P[X(t) < X] = \lim_{t \to \infty} \sum_i \Delta T_i$$

The delta represents the function $X(t)$ where its value is less than the specified value for X. The function P(X) is referred to as the cumulative density function within the equation of function $X(t)$. When graphically represented, the cumulative density function is a monotonically increasing function.

3.4 Probability Distribution

It can be thought of as a function in mathematics that determines the odds of different experiment results. From a technical standpoint, the probability distribution characterizes a random event by assigning probabilities to specific events. Examples of such random events include the outcomes of experiments or surveys. The probability distribution is defined within an underlying sample space, which encompasses all possible outcomes of the observed random event.

3.4.1 Normal (Gaussian) Distribution

The Gaussian distribution, commonly referred to as the normal Gaussian distribution or bell curve, is a continuous probability distribution extensively utilized in statistics and probability theory. It is renowned for its symmetrical shape and is defined by two essential parameters: the mean(μ) and the standard deviation(σ). The distribution is symmetric around the mean, with the highest point of the curve located at the mean. As the name implies, it follows the mathematical form of the Gaussian function or the probability density function (PDF) of a normal distribution.

The probability density of the normal distribution is:

$$P(x) = \frac{1}{\sqrt{2\pi\sigma}} e^{\frac{\frac{1}{2}(X-\bar{X})^2}{\sigma^2}}$$

Where,

$\bar{X}$ is the mean of the distribution.

σ is the standard deviation of distribution

σ^2 is variance

3.4.2 Lognormal Distribution

This type of distribution is a commonly encountered probability distribution. When a variable X is normally distributed with a specific mean and variance, the corresponding random variable $Y = e^x$ is referred to as having a lognormal distribution. This can be expressed as Y being an exponential function of X:

$$Y = e^x \text{ and } X = \ln Y$$

Lognormal PDF:

$$P(X) = \frac{1}{\sigma_x\sqrt{2\pi}} \frac{1}{y} e^{-\frac{1}{2}\left(\frac{\ln y - m}{\sigma_x}\right)^2} \quad \text{For}(y>0)$$

3.4.3 Gamma Distribution

The aggregate consists of R independent exponential random variables, where each random variable is distributed independently and consistently takes positive values.

PDF and CDF function are as follows:

Gamma Distribution, PDF: $f_X(X) = \frac{\lambda}{\Gamma(R)}(\lambda X)^{R-1} e^{-\lambda X}$ if $(x \geq 0, \lambda \geq 0)$

Gamma Distribution, CDF: $1 - \sum_{k=o}^{R-1} \frac{1}{k!}(\lambda X)^k (\lambda X)^k$ if $(x \geq 0, \lambda \geq 0)$

In which Γ(.) represents the gamma function as defined:

Gamma function: $$\Gamma(x) = \int_0^{\infty} e^{-u} u^{(x-1)} du$$

3.4.4 Gumbel Distribution

The Gumbel distribution is commonly employed to represent the distribution of the maximum or minimum values observed in a set of samples from different distributions. It finds utility in assessing the likelihood of extreme events like earthquakes, floods, or other natural disasters occurring. Referred to as the extreme value type I distribution, the Gumbel distribution encompasses two forms: one for extreme maximum (extreme value largest I) and one for extreme minimum (extreme value smallest I). The respective definitions of these forms are as follows:

Gumbel (EV Largest-I) – $$f_x(x) = \alpha e^{-\alpha(x-\beta)} e^{-\exp(-\alpha(x-\beta))}$$

$$F_x(x) = e^{-\exp(-\alpha(x-\beta))} \text{ for } (-\infty < x < \infty)$$

Gumbel (EV Smallest-I) – $$f_x(x) = \alpha e^{-\alpha(x-\beta)} e^{-\exp(-\alpha(x-\beta))}$$

$$F_x(x) = 1 - e^{-\exp(-\alpha(x-\beta))} \text{ for } (-\infty < x < \infty)$$

The Gumbel distribution involves two parameters: β, representing the location, and α, representing the scale (with $\alpha > 0$). It encompasses the entire range of results for the random variable X, where X can take any value between -∞ and ∞.

To calculate the means and variances of both the distribution i.e. largest-I and smallest-I, the following formulas are utilized:

Mean: $$m_x = \beta + \frac{0.57722156649}{\alpha} \quad \text{(largest-I)}$$

$$m_x = \beta - \frac{0.57722156649}{\alpha} \quad \text{(smallest-I)}$$

Variance: $$\sigma_x^2 = \frac{\pi^2}{6\alpha^2} \quad \text{(largest-I and smallest-I)}$$

The value (0.57722156649) is called Euler's constant.

CHAPTER 4

STRUCTURAL RELIABILITY

4.1 Introduction

The evaluation of a structure's performance takes into account its safety, serviceability, and cost-effectiveness. However, the details on the input variables are never entirely definite, accurate, or comprehensive. The unpredictability in the design and analysis of structures come from a variety of factors, including physical randomness, incomplete knowledge, and egregious mistakes. Due to these variabilities, it is not possible to achieve absolute safety in structures, since it is unpredictable how the loads, material strengths, and human errors will affect the structure during its life. Additionally, the conventional deterministic analysis and design methods do not consider the probabilistic variations of the parameters. Moreover, the safety factors based on experience and judgment may not be adequate or economical.

Reliability is a concept applied in many fields and has various interpretations. The most commonly accepted definition of reliability is the probability that an item will perform its expected function during a given period under the operating conditions. In the context of structures, it can be defined as the probability that the structure will not exceed the specified limit states (such as flexure, shear, torsion, or deflection) during a reference period (the life of the structure). The concept of reliability is useful in many applications, such as calibrating codes and developing partial safety factors, assessing existing structures, establishing inspection criteria, and devising maintenance schedules.

To simplify matters, the failure in probability (P_f) can be considered as a basis for defining the reliability (R_o) in the following way,

$$R_0 = 1 - P_f$$

4.2 Levels of Reliability Methods

Depending on the significance of the structure that is being analysed, there are various stages or degrees that can be used to a design process in the subject of structural dependability analysis. The term "level" refers to the amount of information used and delivered in the analysis. Currently, there are four basic levels of safety analysis that can be employed to achieve a given limit state, with increasing levels of sophistication in the treatment of various problems. These levels are typically referred to as level I, II, III, and IV, and are chosen based on the degree of complexity required to accurately assess the safety and reliability of a structure.

4.2.1 Level I

By using the distinctive characteristics of random variables, Level I approaches are used in reliability analysis to take into account the unpredictable nature of numerical issues. These characteristic values are defined as fractiles corresponding to a specific order of the statistical distributions involved. The aforementioned values are linked to partial safety factors to guarantee the design is reliable to the acceptable levels. These factors are determined based on probabilistic considerations, aiming to minimize the disparity between the design's reliability and the target value. The Load & Resistance Factor Design (LRFD) approach is an illustration of a Level I method.

4.2.2 Level II

Level II methods in reliability analysis involve the consideration of mean and variance values for each uncertain parameter, along with their correlation with other parameters. These methods are also referred to as reliability methods and are more sophisticated than level I methods.

4.2.3 Level III

The topic is thoroughly examined using level III approaches, which also integrate the joint density function of probability of random variables throughout the safety domain. These techniques allow for a precise evaluation of dependability through the use of failure probability and reliability indices, such as the reliability index (β).

4.2.4 Level IV

Level IV design techniques should be used for structures that call for engineering monetary evaluation and development under unpredictability and are of major financial significance. These structures should take into account design, upkeep, repairs, the likelihood of possible failure, and the return on capital. This degree of study is best suited for delicate projects like nuclear power plants, towers for transmission, and bridges.

4.3 Calculating the reliability of a structure

Calculating structural dependability is the process of determining how likely it is that a structure will achieve given performance criteria or limit states in the presence of uncertainty. It comprises assessing the effects of uncertain input parameters on the structural response, including loads, material strengths, and other pertinent elements. The goal of this analysis, which often relies on probabilistic models and statistical techniques, is to calculate the likelihood that the structure will succeed or fail. Structural reliability simulations can be carried out using a variety of methods, which includes Monte Carlo simulation, the First-Order Reliability Method etc. The results of this study provide insightful information that may be used to improve the design, increase the structure's safety and serviceability, and make well-informed maintenance and operating decisions.

In the fundamental problem of structural reliability, the focus lies on a single load effect (S) and a single resistance (R) that possess known probability density functions, denoted as $f_S()$ and $f_R()$ respectively. It is crucial for the units of R and S to be consistent, and the safety of the structure is evaluated by comparing the values of R and S. In the event that R is lower than S, it signifies a failure occurrence. The probability of failure, pf, for the structural component can be stated in any of the following manners:

$$p_f = P(R \leq S)$$

$$= P(R - S \leq 0)$$

$$= P({}^{R}/_{S} \leq 1)$$

$$= P(lnR - lnS \leq 0)$$

or, in general

$$= P(G(R,S) \leq 0)$$

The chance of failure may be regarded as the likelihood of breaking the limit state, where G() stands for the limit state function. Figure 2 provides an overview of the density functions of $f_S()$ and $f_R()$ for variables S and R, respectively. It also shows the bivariate density function $f_{RS}(r,s)$ that describes the relationship between the two variables.

The joint density function $f_{RS}(r,s)$ represents the probability of R falling within the range of r to r+Δr and S falling within the range of s to s+Δs for an infinitesimally small element (ΔrΔs). In Figure 2, the shaded failure domain D represents the equations that define this probability. As Δr and Δs approach zero, the failure probability corresponds to the probability that both R and S fall within this domain.

$$p_f = P(R - S \leq 0) = \iint_D f_{RS}(r,s)drds$$

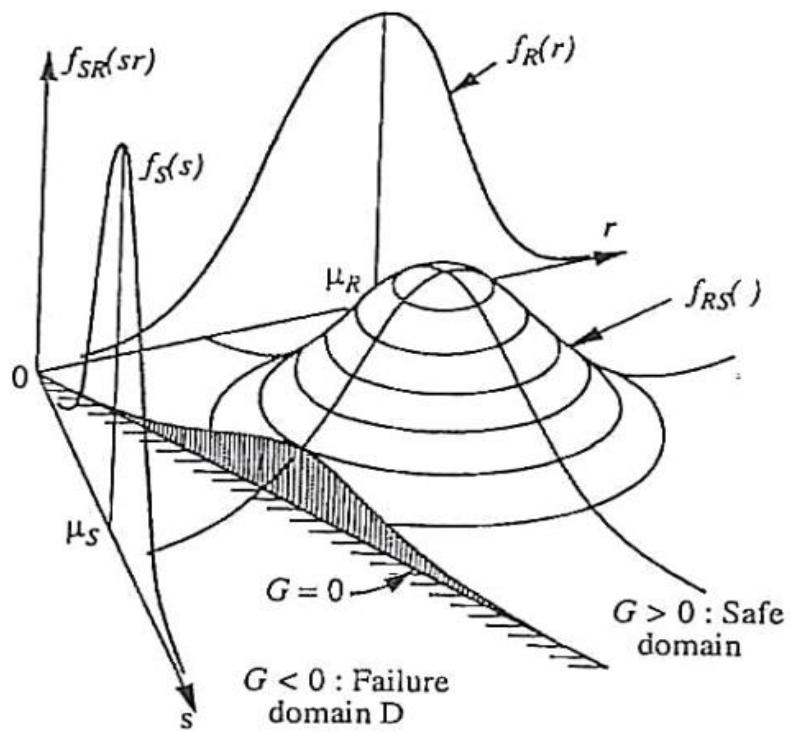


Fig. 4.1 : Joint density function $f_{RS}(r,s)$, marginal density functions $f_R(r)$ and $f_S(s)$ and failure domain *D*

When R and S are independent,

$$f_{RS}(rs) = f_R(r) f_S(s)$$

Moreover, equation for probability of faliures then becomes:

$$p_f = P(R - S \leq 0) = \int_{-\infty}^{\infty} \int_{-\infty}^{s \geq r} f_R(r) f_S(s) dr ds = \int_{-\infty}^{\infty} F_R(x) f_s(x) dx$$

The convolution integral's significance becomes evident when considering Figure 2. In this context, $F_R(x)$ represents the probability of the structure's resistance being less than or equal to x, indicating structural failure. Conversely, $f_s(x)$ shows the probability of the load effect S on the member falling within the range from x to x+Δx, with Δx approaching zero. To determine the overall probability of failure, integration across all possible values of x is required. Figure 3 illustrates this concept, displaying the density functions $F_R(x)$ and $f_s(x)$ on the same axis. Through this integration process, a comprehensive understanding of the probability of failure can be attained.

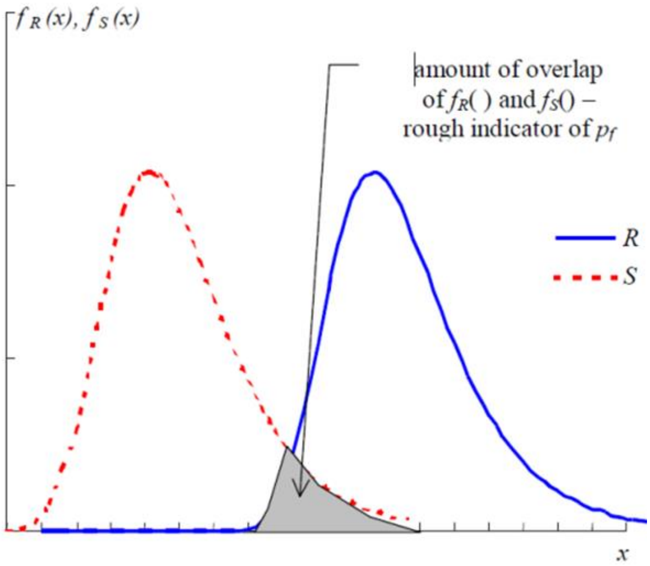


Fig. 4.2 : Basic *R*-*S* problem: $f_R(\,)\, f_S(\,)$ representation

4.3.1 Special case of normal random variable

Analytical integration of the convolution integral (2) can be accomplished for specific distributions of R and S. A notable example is when both variables follow normal distributions with means μ_R and μ_S, and variances σ_R^2 and σ_S^2, respectively. In this case, the safety margin Z, defined as Z=R-S, can be evaluated using established rules for adding normal random variables, yielding the mean and variance of Z.

$$\mu_Z = \mu_R - \mu_S$$
$$\sigma_Z^2 = \sigma_R^2 + \sigma_S^2$$

Equation for probability of failure then becomes :-

$$p_f = P(R - S \le 0) = P(Z \le 0) = \Phi\left(\frac{0 - \mu_Z}{\sigma_Z}\right)$$

Let $\Phi(\ )$ stand for the typical normal distribution function, which has a mean of 0 and a variance of 1. The random variable i.e. Z, defined as the difference between R and S, is depicted in the Figure, with the shaded region indicating the failure region $Z \le 0$. By utilizing the aforementioned equations, it can be deduced that

$$P_f = \Phi\left[\frac{-(\mu_R - \mu_S)}{(\sigma_R^2 + \sigma_S^2)^{\frac{1}{2}}}\right] = \Phi(-\beta)$$

where, $\beta = {}^{\mu_Z}/_{\sigma_Z}$ is defined as ***reliability (safety) index***.

The equation mentioned above shows that if the standard deviations of σ_R and σ_S are increased, when the gap between the resistance's mean and the load effect's mean is less, the value of Pf increases. This can be inferred from, where the overlap of f_R () and f_S() can be considered as an indicator of Pf.

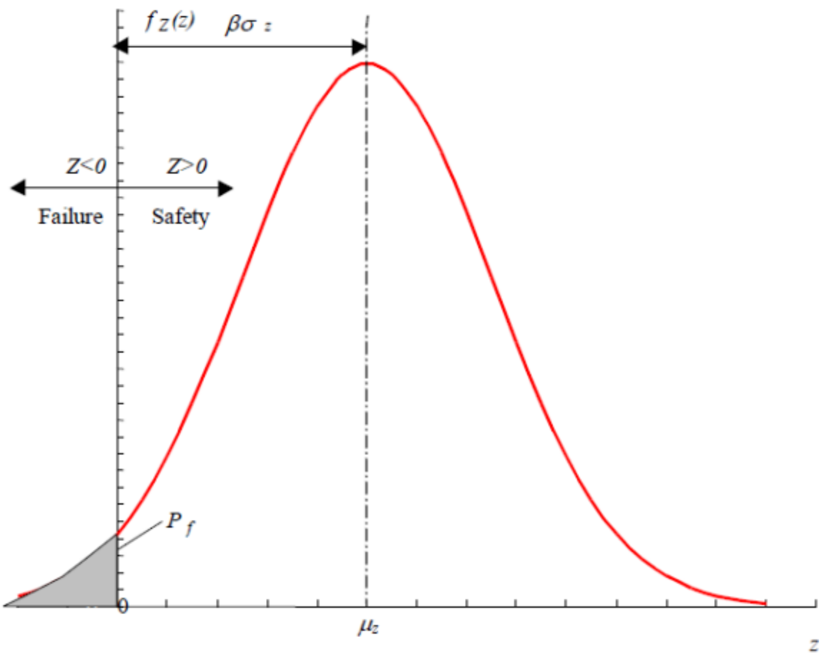


Fig. 4.3 : Distribution of margin safety Z = R – S

4.3.2 Reduced Variable

It is advised to express all random factors in their "established form," which is a non-dimensional form. The standard forms for the fundamental variables R and Q are given by:

$$Z_R = \frac{R - \mu_R}{\sigma_R}$$

$$Z_Q = \frac{R - \mu_Q}{\sigma_Q}$$

The transformed variables Z_R and Z_Q are derived by converting the random variables into a "standard form," which represents a dimensionless version of the variables. In terms of the reduced variables, the resistance, or R, and load, or Q, may be represented as follows:

$$R = \mu_R + Z_R\sigma_R$$

$$Q = \mu_Q + Z_Q\sigma_Q$$

$g(R, Q)$ = R Q is the limit state function. This may be expressed using equations based on the condensed variables i.e. eq. 2. The result is

$$g(Z_R, Z_Q) = \mu_R + Z_R\sigma_R - \mu_Q - Z_Q\sigma_Q = (\mu_R - \mu_Q) + Z_R\sigma_R - Z_Q\sigma_Q$$

The equation above represents a straight line in the space of reduced variables Z_R and Z_Q for any specific value of $g(Z_R, Z_Q)$. The line that corresponds to $g(Z_R, Z_Q) = 0$ separates the failure domain from the safe domain in the space of reduced variables. The literature sometimes expresses the loads Q and resistances R in terms of capacity C and demand D as well.

4.3.2 Reliability Index

The dependability index was given a fresh definition by Hasofer and Lind, who said that it was the inverse of the coefficient of variance. As shown in Figure 3 on the line with $g(Z_R, Z_Q) = 0$, the value of this index is denoted by the distance that is perpendicular between the source of reduced variables and the design point or breakdown point.

$$\beta = \frac{\mu_R - \mu_Q}{\sqrt{\sigma_R^2 + \sigma_Q^2}}$$

Let β represent the reciprocal of the coefficient of variance of the function $g(R,Q) = R - Q$. In the case where R and Q are uncorrelated and follow a normal distribution, the index of reliability can be connected to the failures probability as follows,

$$\beta = -\phi^{-1}(P_f) \ or \ P_f = \phi(-\beta)$$

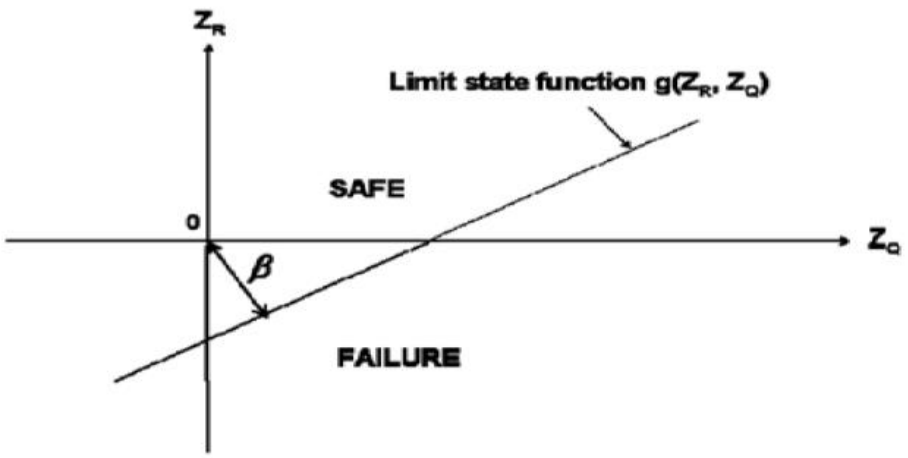


Fig. 4.4 : Reliability index defined as the least distance in the area of reduced variables

4.4 First Order Reliability Method

The first-order Taylor series approximations is used to linearize the performance function at the mean values of the random variables. In the mean value first-order second moment (MVFOSM) technique, the mean and variance of the random variables are used as second-moment statistics, is usually referred to as this. In his work, Cornell (1969) suggested a streamlined two-variable strategy and assumed that the final probability of Z would follow a standard distribution. The fraction of the predicted value of Z along with its standard deviation is how he developed the reliability index, abbreviated as β_c. The ordinate's absolute value at Z = 0 on the generalised normal probabilities plot, as shown in Figure 4, can be used to calculate β_c. This can be stated mathematically as follows:

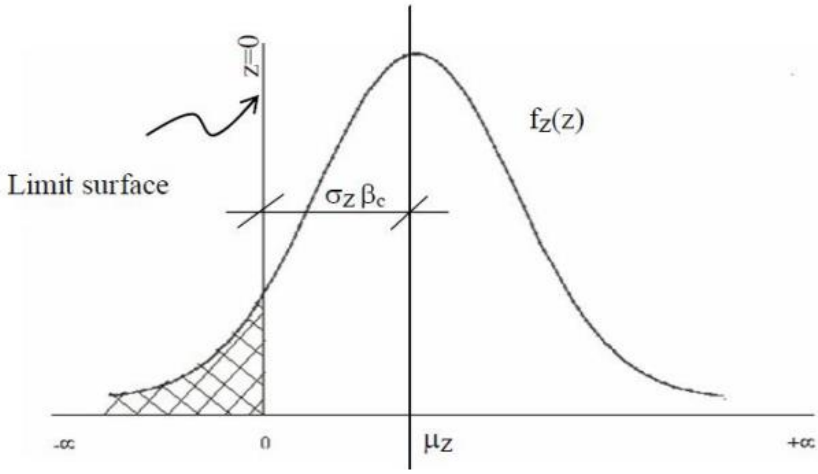


Fig. 4.5 : Definition of Reliability Index and Limit State

$$\beta_c = \frac{\mu_R - \mu_S}{\sqrt{\sigma_R^2 + \sigma_S^2}}$$

Alternatively, if joint probability density function $f_x(x)$ is identified for the multi variable case, then probability of failure p_f is given by

$$\mu_Z \approx g(\mu_{X_1}, \mu_{X_2}, \ldots\ldots\ldots\mu_{X_n})$$

Where it is understood that $g(X)<0$.

Since the integral mentioned previously typically lacks an analytical solution, the First-Order Reliability Method must be used to make an approximation. In order to use this strategy, the original scenario must be approximated to an idealised scenario in which g(X) is a linear function and X is a collection of uncorrelated Gaussian variables with a mean equal to zero and the standard deviation equal to one. A rough solution for the integral can be determined using the FORM technique, providing a workable answer to the issue. The probability of failure (p_f) is then understood as

$$p_f = P(g(X) < 0) = P\left(\sum_{i=1}^{n} \alpha_i X_i - \beta < 0\right) = \phi(-\beta)$$

The provided equation displays the chance of failure as determined by the FORM technique. The parameters α_i and in this equation denote the directional cosine of the random variable X_i and β the distances among the origin and the hyperplane g(X)=0, respectively. The term n is the number of fundamental random variables X, while the term Φ denotes the standard normal distribution function. It is important to understand that this approach is based on an erroneous assumption that X is a vector made up of distinct Gaussian parameters with a mean of 0 and a standard deviation of 1. The equation also suggests that the function g(X) is linear.

Let performance function is given as

$$Z = g(X) = g\,(X_1, X_2 \ldots X_n)$$

The performance function with respect to the mean value according to the Taylor series expansion is given by the equation,

$$Z = g(\mu_X) + \sum_{i=1}^{n} \frac{\partial g}{\partial X_i}(X_i - \mu_{X_i}) + \frac{1}{2}\sum_{i=1}^{n}\sum_{j=1}^{n} \frac{\partial^2 g}{\partial X_i\, \partial X_j}(X_i - \mu_{X_i})\left(X_j - \mu_{X_j}\right) + \cdots$$

The first-order Taylor series estimate of the performance function may be represented as a linear expression of the standardised random variables Z by computing the derivatives of the performance function at the averages of random variables $(X1, X2 \ldots Xn)$, where μx_i is the mean value of X_i. Converting the series in linear terms, the mean and variance of Z can be evaluated as:

$$\mu_Z \approx g\left(\mu_{X_1}, \mu_{X_2, \ldots\ldots\ldots\ldots} \mu_{X_n}\right)$$

And,

$$\sigma_Z^2 \approx \sum_{i=1}^{n}\sum_{j=1}^{n} \frac{\partial g}{\partial X_i}\frac{\partial g}{\partial X_j} var\left(X_i, X_j\right)$$

Where $var(X_i, X_j)$ is covariance of X_i and X_j. Since the variances are not correlated, then respectively the variance for z can be derived as

$$\sigma_Z^2 \approx \sum_{i=1}^{n} \left(\frac{\partial g}{\partial X_i}\right)^2 var(X_i)$$

Finding the ratio of mean(μ_z) to standard deviation of Z (σ_z) allows for the subsequent calculation of the dependability index as

$$\beta = \frac{\mu_Z}{\sigma_Z}$$

4.4.1 Hasofer-Lind's Reliability Index

Let the failure function be the function of independent stock variables $X_1, X_2, \ldots X_n$, $i.e.$ $g(X_1, X_2, \ldots X_n)$. The basic variables are then standardized using the relationship

$$Z_i = \frac{X_i - \mu_i}{\sigma_i} \quad i = 1,2,3, \ldots \ldots, n.$$

Where, μ_i=μx_i and σ_i= σx_i ,

As a function of Z_i, the failure of the system within the Z coordinate is represented. The failure surface equation is obtained in the normalised coordinate system by integrating this equation within the failure characteristic and setting it to zero. The failure surface separates the sample space into two areas namely safe and failure, due to the normalization of the basic variables with $\mu_{zi} = 0$ and $\sigma_{zi} = 1$.

It is significant to notice that the origin often lies inside the safe zone and that the z-coordinate system in issue displays rotational symmetry with respect to the standard deviation. The location of the failure surface in relation to the origin inside the normalised coordinate system determines the dependability measure. Reliability rises as the failure surface goes farther from the origin, whereas reliability declines as the failure surface gets nearer to the origin. The reliability index β was created by Hasofer and Lind as the shortest path through the normalised coordinate system between the origin O to the failure surface. The selection of failure function has no impact on this safety precaution since identical failure functions produce an identical failure surface. In the case of linear failure surfaces, the reliability index $\beta = \mu M/\sigma M$ can be utilized. The shortest path from the origin to a nonlinear failure surface is not, however, uniquely defined, and numerical integration is necessary to calculate the failure probability. An approximation of the reliability index can be obtained by utilizing the tangent plane to the design point, although the accuracy of the approximation relies on whether the failure surface is concave or convex towards the origin.

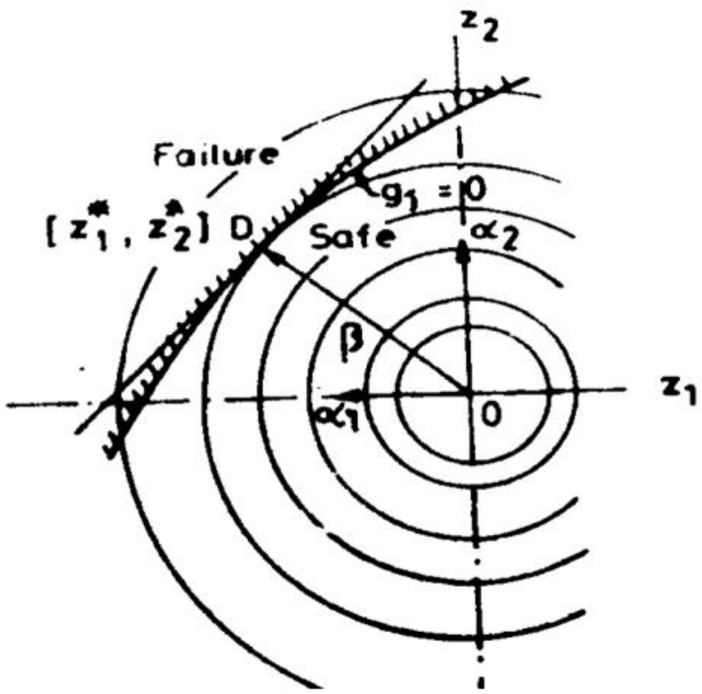


Fig. 4.6 : Formulation of safety analysis in normalized coordinates

4.5 SOFTWARE USED FOR ANALYSIS

4.5.1 COMREL

COMREL is a software program developed to conduct reliability analysis in engineering applications. It offers advanced methods to evaluate the probability's of failure and reliability of the systems subjected to various uncertainties.

COMREL incorporates first-order as well as second-order reliability methods to assess the reliability index (β) through iterations. It provides algorithms to determine the most likely failure point, considering both differentiable and non-differentiable failure criteria. These techniques allow for efficient and accurate analysis of reliability and failure probabilities.

COMREL supports the inclusion of arbitrary dependence structures in stochastic models, such as Nataf models, Hermite, also Rosenblatt. It can handle various stochastic models (44 models) from SYSREL, enabling the user to input parameters in different forms.

Additionally, COMREL allows the definition of multiple failure conditions within a single task, with state functions either called from external programs or expressed in mathematical representation. It offers a variety of built-in functions, such as hyperbolic, trigonometric, logarithmic, and other special functions like Bessel and Gamma functions. COMREL provides flexibility in differentiation, numerical integration, root finding, and includes test functions and comparative operators.

Overall, COMREL is a powerful tool that enables engineers to perform reliability analysis, assess failure probabilities, and evaluate the reliability of systems under uncertainties, contributing to more robust and dependable engineering designs.

4.5.2 ETABS

ETABS (Extended Three-Dimensional Analysis of Building Systems) is a comprehensive software program developed by CSI company. It is widely used in the engineering industry for the analysis and design of buildings and structures.

ETABS offers a range of powerful features and tools to facilitate the structural engineering process. Its 3D modeling capabilities allow users to create detailed and realistic models of structures, defining geometric elements, material properties, and structural components.

The software provides advanced analysis techniques, including linear and nonlinear analysis, to accurately predict the structural response to various loads and forces. It can handle static and dynamic analysis, accounting for factors such as material nonlinearity and seismic or wind loads.

ETABS also includes design modules that enable engineers to design various structural components, such as beams, columns, walls, and foundations. The software automatically generates design loads, performs code checks, and produces comprehensive reports and documentation.

Furthermore, ETABS seamlessly integrates with other CSI software programs like SAP2000 and SAFE, allowing for enhanced capabilities and a streamlined workflow in the structural design process.

With its user-friendly interface, powerful analysis capabilities, and efficient design tools, ETABS has become a trusted solution for structural engineers involved in the analysis and design of complex building systems. It ensures the compliance of structures with international building codes and standards, making it an indispensable tool in the field of structural engineering.

CHAPTER 5

METHODOLOGY

To derive the Moment of Resistance equation for a section, it is crucial to consider the probabilistic nature of the physical parameters involved, as they often exhibit statistical variations. Hasofer and Lind introduced a method that addresses these uncertainties by defining a theoretical definition of the reliability index (β). This method incorporates mean and standard deviation values to account for the statistical variations in the physical parameters.

STAAD PRO software is used to analyse the critical bending moment value and axial forces. These numbers are then transferred to COMREL for additional examination. The reliability index value and its inverse are evaluated using first-order and second-order reliability techniques by COMREL, which also performs numerous iterations to calculate the chance of failure.

5.1 Brief Review of Time History Analysis

Time history analysis is a method used in structural engineering to simulate and assess the structure's responsiveness to change subjected to time-varying loads. It involves analyzing the structure's behaviour over time, considering the actual recorded or synthetic load inputs as a function of time.

Time history analysis begins with obtaining the load inputs, which can be recorded from actual events or generated synthetically based on the expected dynamic forces. These loads can include seismic ground motions, wind loads, or any other time-dependent forces that act upon the structure.

The next step is to establish the mathematical model of the structure, which typically involves using finite element analysis or other numerical methods. The model represents the physical properties and behaviour of the structure, including its geometry, material properties, connections, and support conditions.

The load inputs are then applied to the structural model in a time-dependent manner. This involves incrementally applying the loads at each time step and calculating the responses of the structure at that specific time. The analysis proceeds step by step, considering the dynamic equilibrium of the structure at each time increment.

During the time history analysis, the response of the structure is computed in terms of various parameters such as displacements, accelerations, forces, and stresses. These response quantities can be obtained for specific locations or elements of interest within the structure.

The time history analysis considers the dynamic attributes of the structure, encompassing its inherent frequencies, mode shapes, and damping characteristics. These attributes significantly influence the structural response to external loads and the dissipation of energy during dynamic occurrences.

The analysis results provide valuable insights into the structural behaviour under dynamic loads, helping engineers assess its performance, identify potential failure modes, and ensure the structure's safety and reliability. It can also aid in the design of structural elements to withstand specific dynamic events, such as earthquakes or severe winds.

5.2 Methodological Verification and Rigor

5.2.1 Sample Problem for Time History Analysis

Question- The given diagram illustrates the plan dimensions of a 10-storey building with a storey height of 3.0 m. The floor area has a dead load (DL) of 4 kN/m^2, including the floor slab and finishes, while the weight of the partitions on each floor is 2 kN/m^2. Additionally, each floor is subjected to a live load intensity of 3 kN/m^2, and the roof has a live load intensity of 1.5 kN/m^2. The building is situated in Delhi on a hard soil foundation. The task at hand is to calculate the seismic forces at various floor along with shears.

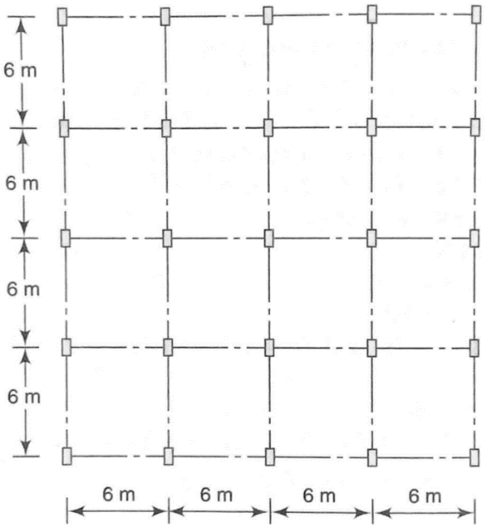


Fig. 5.1 : Structural Plan

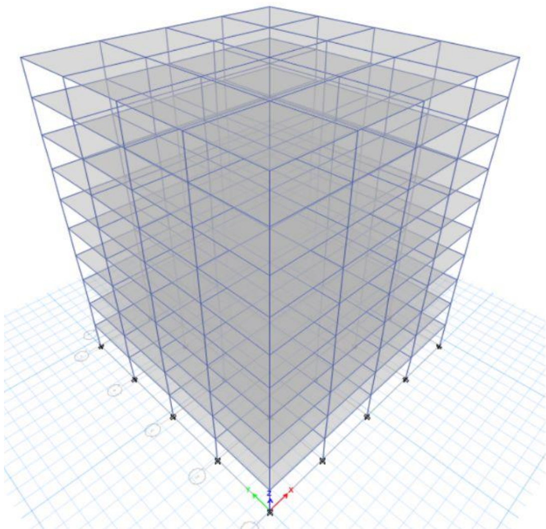


Fig. 5.2 : 3d view of structure

<u>Design Parameters</u>:

Considering Delhi (Zone IV), zone factor Z=0.24

Importance factor, I=1.0

Response reduction factor, R=3.0

<u>Seismic Weight</u>:

Floor area = 24×24 = 576 m^2

Dead weight = 4 kN/m^2

Weight of partitions = 2 kN/m^2

Percentage of live load to be considered is 25%

Fundamental natural period of vibration,

$$T_a = 0.075h^{0.75} = 0.075(30)^{0.75} = 0.96s$$

Damping coefficient = 5%

Average response acceleration coefficient = 1.04

Design horizontal seismic coefficient,

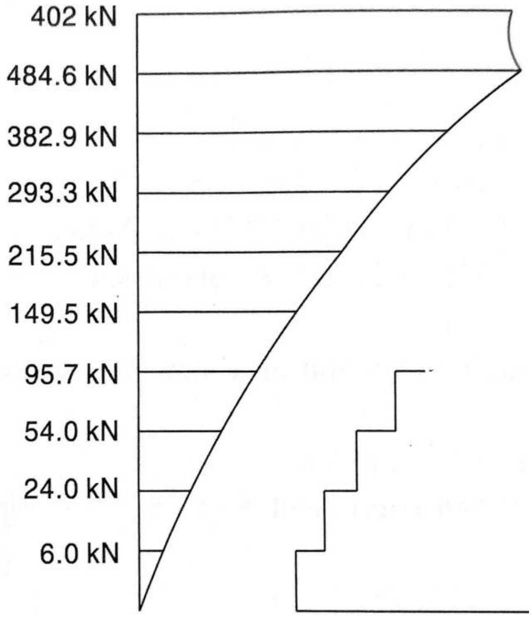
$$A = \frac{ZI(\frac{S_a}{g})}{2R} = \frac{0.24 \times 1.0 \times 1.04}{2 \times 3} = 0.04159$$

Base shear $V_B = A_h\, W = 0.04159 \times 50662 = 2107.5$ kN

Design Lateral Force at floor i, $Q_i = V_B \frac{W_i h_i^2}{\sum_1^n W_i h_i^2}$

Table 5.1 – Results based on Equivalent Static Lateral Load method

Mass No.	W_i (kN)	h_i (m)	$W_i h_i^2$ (kN-m^2)	$\frac{W_i h_i^2}{\sum_{i=1}^{n} W_i h_i^2}$	Q_i (kN)
1	3519	30.0	3167100	0.1907	402.0
2	5238	27.0	3818502	0.2299	484.6
3	5238	24.0	3017088	0.1817	382.9
4	5238	21.0	2309958	0.1391	293.3
5	5238	18.0	1697112	0.1022	215.5
6	5238	15.0	1178550	0.0709	149.5
7	5238	12.0	754272	0.0454	95.7
8	5238	9.0	424278	0.0255	54.0
9	5238	6.0	188568	0.0114	24.0
10	5238	3.0	47142	0.0028	6.0
$\Sigma W_i h_i^2$ = 16602570					



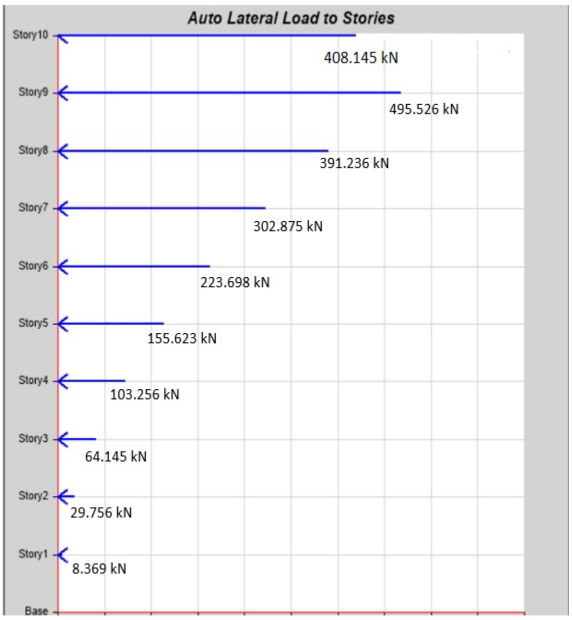


Fig. 5.3 : Story Shear Graphically (Manually)

Fig. 5.4 : Story Shear Graphically (Software)

Table 5.2– Results based on Non-Linear Time History Analysis

TABLE: Story Response				
Story	Elevation	Location	X-Dir	Y-Dir
	m		kN	kN
Story10	30	Top	408.145	0
Story9	27	Top	495.526	0
Story8	24	Top	391.236	0
Story7	21	Top	302.875	0
Story6	18	Top	223.698	0
Story5	15	Top	155.623	0
Story4	12	Top	103.256	0
Story3	9	Top	64.145	0
Story2	6	Top	29.756	0
Story1	3	Top	8.369	0
Base	0	Top	0	0

5.2.2 Sample Problem for Reliability Analysis

Question- Calculate the reliability index of an I-beam that is simply supported and experiencing a point load Q at mid-span, at the limit state of shear. Take into consideration that μ_Q= 5000 N, σ_Q= 1500 N, μ_{fs}= 100 N/mm^2, σ_{fs}= 15 N/mm^2, μ_d= 60 mm, σ_d= 4 mm, $\frac{d}{t_w}$ = 50.

Where, d is the depth of the beam,

t_w is the thickness of the web,

fs is the shear strength of the material.

Solution-

$$\text{Maximum shear force} = \frac{Q}{2}$$

For shear failure in beam,

$$f_s t_w d - \frac{Q}{2} \leq 0$$

Hence, the failure surface equation is

$$g(X) = f_s t_w d - \frac{Q}{2} = 0$$

Variation in t_w being negligible, t_w can be considered as deterministic.

Let,

$$z_1 = \frac{(f_s - \mu f_s)}{\sigma f_s}$$

$$z_2 = \frac{(d - \mu d)}{\sigma d}$$

$$z_3 = \frac{(Q - \mu Q)}{\sigma Q}$$

Substituting the values in equation given for $g(X) = 0$,

$$g_1(z) = t_w \ (\sigma f_s z_1 + \mu f_s)(\sigma d z_2 + \mu d) - \frac{\sigma Q}{2} z_3 - \frac{\mu Q}{2} = 0$$

On further substitution,

$$g_1(z) = 1080z_1 + 480z_2 + 72z_1z_2 - 750z_3 + 4700 = 0$$

At design point we know that, $z_i = \beta\alpha_i$

$$g_1(z) = 1080\,\beta\alpha_1 + 480\,\beta\alpha_2 + 72\,\beta^2\alpha_1\alpha_2 - 750\,\beta\alpha_3 + 4700 = 0$$

$$\beta = \frac{-4700}{1080\alpha_1 + 480\alpha_2 + 72\beta\alpha_1\alpha_2 - 750\alpha_3}$$

Taking partial derivative of $g_1(z)$,

$$\left(\frac{\partial g_1}{\partial z_1}\right) = (1080 + 72z_2)$$

$$\left(\frac{\partial g_1}{\partial z_2}\right) = (480 + 72z_1)$$

$$\left(\frac{\partial g_1}{\partial z_3}\right) = (-750)$$

Start with,

$$\beta = 6,\ \alpha_1 = -0.5,\ \alpha_2 = -0.5,\ \alpha_3 = 0.707$$

Substituting these values in equation above,

$$\beta = \frac{-4700}{1080(-0.5) + 480(-0.5) + 72(6)(-0.5)(-0.5) - 750(0.707)} = 3.909$$

Now using equation,

$$\alpha_i = -\frac{1}{K}\left(\frac{\partial g_1}{\partial z_i}\right)$$

$$\alpha_1 = -\frac{1}{K}[1080 + 72\ (3.909)\ (-0.5)\ ] = -\frac{939.276}{K}$$

$$\alpha_2 = -\frac{1}{K}[480 + 72\ (3.909)\ (-0.5)\ ] = -\frac{339.276}{K}$$

$$\alpha_3 = -\frac{1}{K}[\ -750\ ] = \frac{750}{K}$$

$$K^2 = (-939.276)^2 + (-339.276) + (750)^2$$

$$= 1559847.608$$

$$K = 1248.938$$

Hence,

$$\alpha_1 = -\frac{939.276}{1248.938} = -0.75$$

$$\alpha_2 = -\frac{339.276}{1248.938} = -0.276$$

$$\alpha_3 = \frac{750}{1248.938} = 0.6005$$

The cycle is repeated using the updated values of β, α_1, α_2, and α_3 until β reaches the minimum convergence. Summary for the data is given in the table below,

Table 5.3- Computation of β

Variable	Iteration			
	Start	I	II	III
β	6	3.909	3.524	3.526
α_1	-0.5	-0.75	-0.7209	-0.7165
α_2	-0.5	-0.276	-0.333	-0.341
α_3	0.707	0.6005	0.607	0.6095

Result:

$$\beta = 3.526,\ \alpha_1 = -0.7165,\ \alpha_2 = -0.341,\ \alpha_3 = 0.6095$$

Also, the design point is $z^* = (\beta\alpha_1, \beta\alpha_2, \beta\alpha_3)$

5.2.2.1 Solved using COMREL

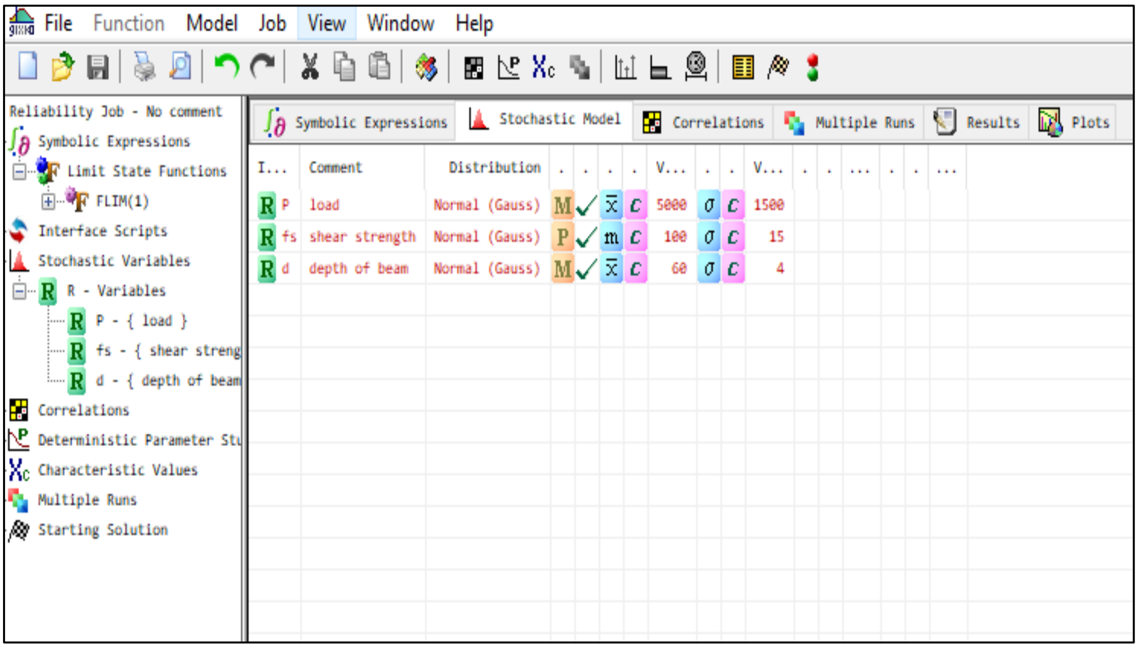


Fig. 5.5 : Numerical Representation for assigning the variables with corresponding values of mean and standard deviation

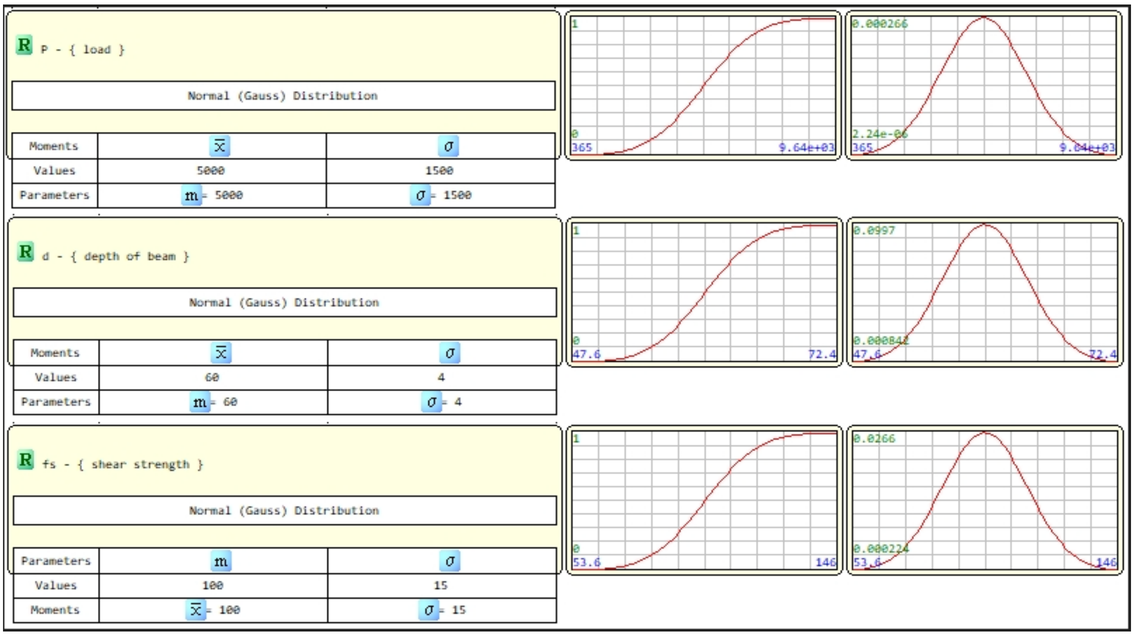


Fig. 5.6 : Graphical Representation for assigning the variables with corresponding values of mean and standard deviation

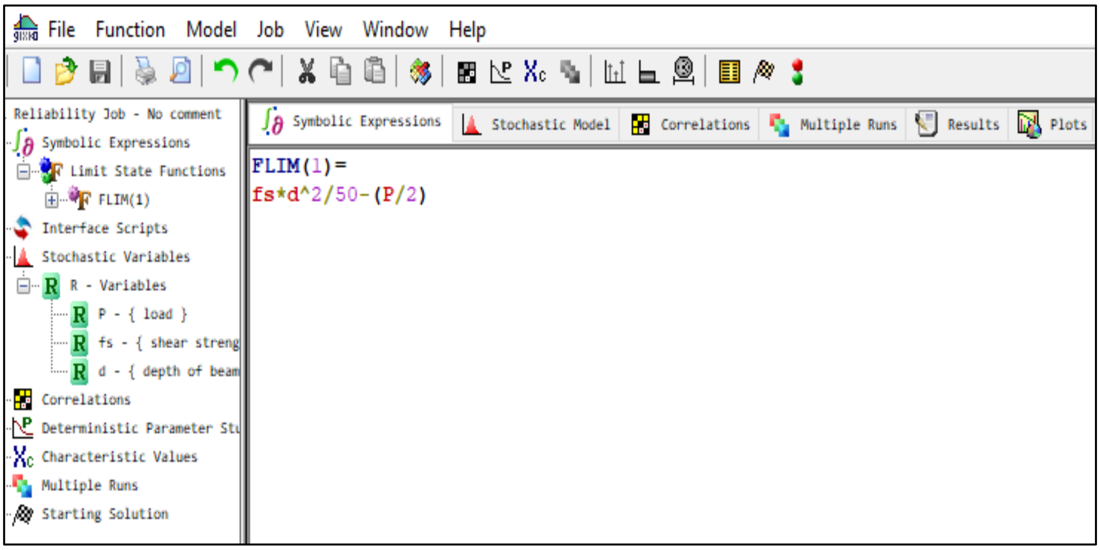


Fig. 5.7 : Defining the limit state function for the program

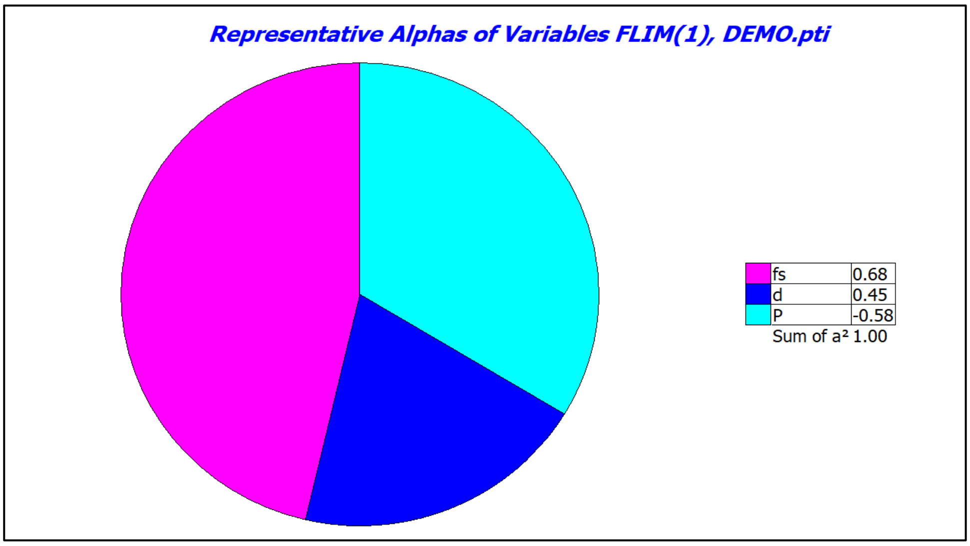


Fig. 5.8 : Representative alphas of variables in the function

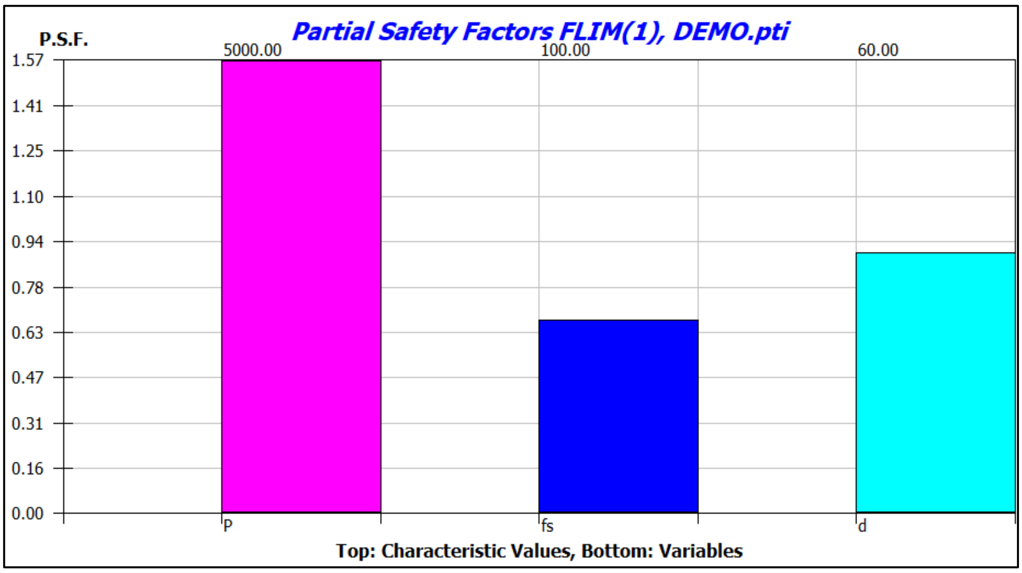


Fig. 5.9 : Partial Safety Factors

Numerical Result

```
 Job name ............ : DEMO
 Failure criterion no. :      1
 Comment : No comment
 Transformation type   : Rosenblatt
 Optimization algorithm: RFLS
 Date(dd.mm.yyyy) .... : 13.05.2023
 Time(hh:mm) ........  : 16:18
 Comrel-TI, (Version 2023), Copyright: RCP GmbH (1989-2023)
------------------------------------------------------------------------------

Block beginning with Keyword $CHARVAL not found in Input-file !
Characteristic Values default to mean values.

Create buffer for $LIMFUNC with      2852 Bytes (Ier from Fstat =    0)

   ----------------------------------------
                COMREL-TI
   Iteration monitoring of YBETAU algorithm
   ----------------------------------------
      General enviroment information
      Maximal number of iterations in RFLS1 =     50
      Maximal number of iterations in RFLS2 =     50
      Type of numerical derivatives         =      0
      Precision for convergence criteria    =   1.0000E-03
      Line search precision                 =   1.0000E-01

   Initial U-space solution
         0.0000      0.0000      0.0000

   Algorithm 1 : Iteration history
------------------------------------------------------------------------------
Iteration No.  1; CPU-seconds(cumulative):     0.012
 Scaled St.F(U) =  0.1139    ; BETA =      0.0000; BETA/||U||=      0.0000
------------------------------------------------------------------------------
Iteration No.  2; CPU-seconds(cumulative):     0.020
 Scaled St.F(U) = -0.9358E-03; BETA =      2.8451; BETA/||U||=      0.8722
------------------------------------------------------------------------------
Iteration No.  3; CPU-seconds(cumulative):     0.020
 Scaled St.F(U) = -0.6096E-04; BETA =      3.2609; BETA/||U||=      1.0010
------------------------------------------------------------------------------
Iteration No.  4; CPU-seconds(cumulative):     0.027
 Scaled St.F(U) = -0.4052E-05; BETA =      3.2575; BETA/||U||=      1.0001
------------------------------------------------------------------------------
Iteration No.  5; CPU-seconds(cumulative):     0.031
 Scaled St.F(U) = -0.2644E-06; BETA =      3.2572; BETA/||U||=      1.0000
------------------------------------------------------------------------------
Iteration No.  6; CPU-seconds(cumulative):     0.031
 Scaled St.F(U) = -0.1721E-07; BETA =      3.2572; BETA/||U||=      1.0000
   Statistics after RFLS-algorithm #1 (YRFLS1)
   Cumulative seconds used         :       0.0312
   Number of iterations            :      6
   Cumulative gradient calls       :      6
   Cumulative state function calls:     25
   State function scaling          :   4.7000E+03

   Statistics after beta-point search :
   Cumulative seconds used         :       0.0312
   Number of iterations(RFLS-1+-2):      6
   Cumulative gradient calls       :      6
   Cumulative state function calls:     25

   Transfer to GUI: NBV=   3;  NPVEC=   0

   -----Vector U-mem plus FU at solution                     -----
     -2.217         -1.457         1.889        -0.1721E-07

   -----Vector X at solution transfered to GUI              -----
      66.74          54.17          7834.

   -----Vector of constant Parameters transfered to GUI   -----


   beta-value to GUI:      3.2572;  Pf-value to GUI:  5.6258E-04
```

CHAPTER 6

RELIABILITY ANALYSIS

6.1 Reliability assessment of a column in a multi-storey asymmetric RCC building

To analyze earthquake loading on a six-storey RCC building, a column was considered and time history analysis was conducted in ETABS to determine both axial load as well as biaxial bending moments on the column, using data from the well-known El Centro earthquake. The resulting table was then exported to MS Excel to calculate the mean and standard deviations, which were obtained using the software's built-in formulae. The column failure criteria is taken from the RCC code IS:456-2000 prevalent in India for biaxial bending as well as axial loading which is given as:

$$\left[\frac{M_{ux}}{M_{ux1}}\right]^{\alpha_n} + \left[\frac{M_{uy}}{M_{uy1}}\right]^{\alpha_n} \leq 1.0$$

Where,

M_{ux}, M_{uy} = moments about x and y axes due to design loads,

M_{ux1}, M_{uy1} = maximum uniaxial moment capacity for an axial load of P_u, bending about x and y axes respectively, and

α_n is related to P_u/P_{uz}

Where $P_u = 0.45\ f_{ck} A_c + 0.67\ f_y A_{sc}$

$P_{uz} = 0.45\ f_{ck} A_c + 0.75\ f_y A_{sc}$

Where,

P_u = axial load on the member,

f_{ck} = characteristic compressive strength of the concrete,

A_c = Area of concrete,

f_y = characteristic strength of the compression reinforcement,

A_{sc} = area of longitudinal reinforcement for columns.

For values of P_u/P_{uz} = 0.2 to 0.8, the values of α_n vary linearly from 1.0 to 2.0. For values less than 0.2, α_n is 1.0; for values greater than 0.8, α_n is 2.0.

The equations mentioned above are utilized to construct the failure's limiting state equation in COMREL. Various probability density functions (PDFs) including normal, lognormal, and Gumbel max have been considered during the analysis. To achieve optimized reliability, certain PDFs have been assigned to specific input variables. FORM analyses have been conducted, and the reliability and probability of failure system have been computed using this approach.

The figures below display the values given, middle steps, and results obtained from various software applications.

6.1.1 Structural Modelling

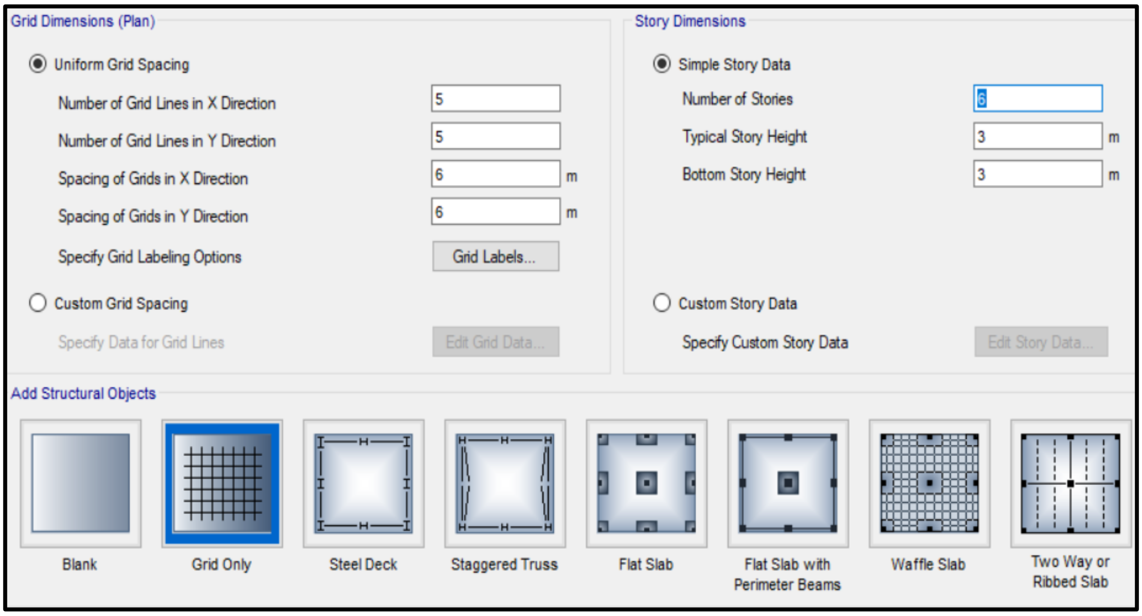


Fig. 6.1 : Defining the grid points for the structural plan

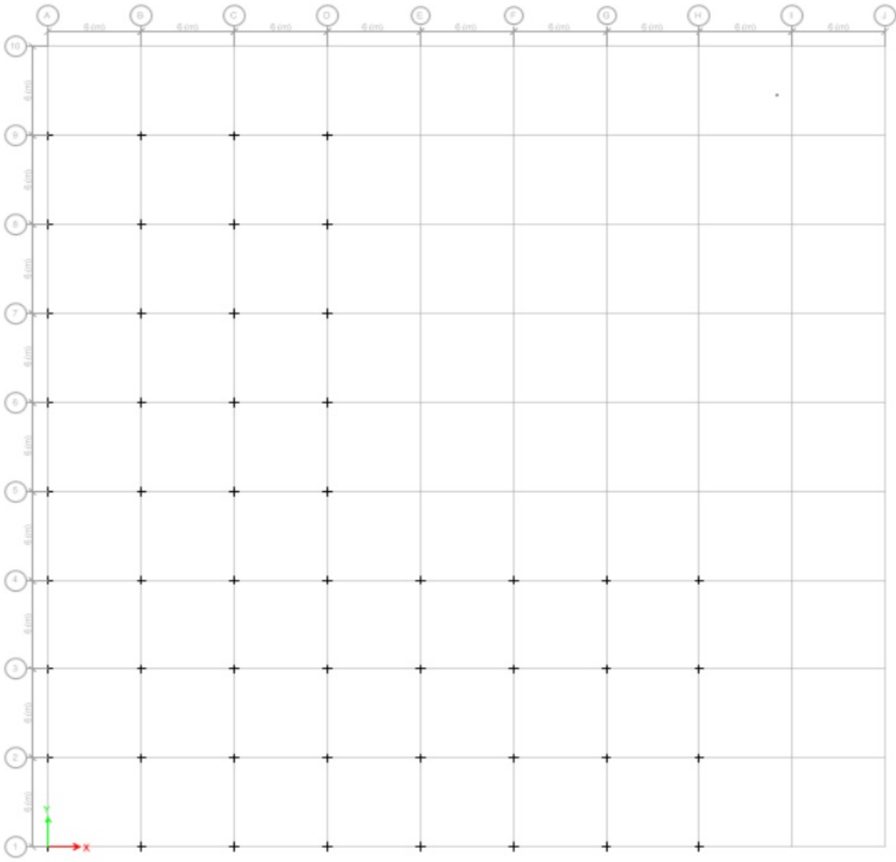


Fig. 6.2 : Structural Plan

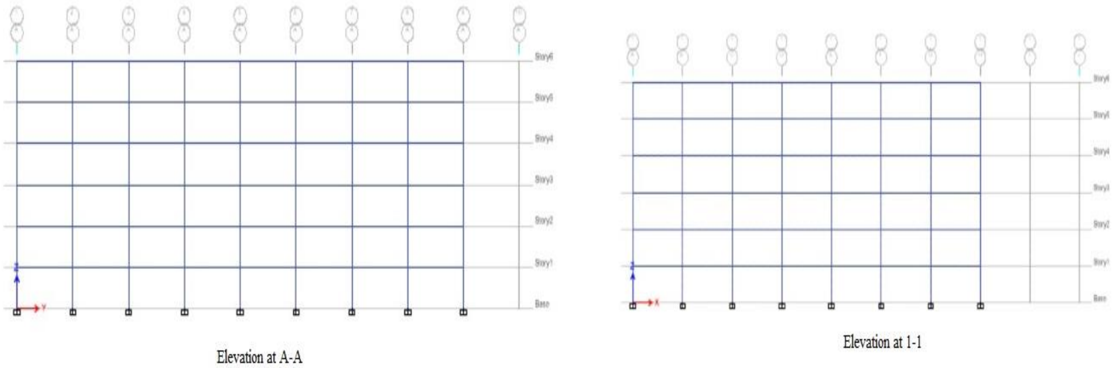


Fig. 6.3 : Structural Elevation

Table 6.1– Structural Data

Storeys	**G+5**
Plan Dimension	for longer leg- 48m by 18m for shorter leg- 42m by 18m
Slab Thickness	0.15m
Beam Dimension	0.3m × 0.43m
Column Dimension on regular structure	0.5m × 0.5m
Live load on floor	3 KN/m2
Live load on roof	1.5 KN/m2
Concrete grade	M30
Rebar Grade	Fe415

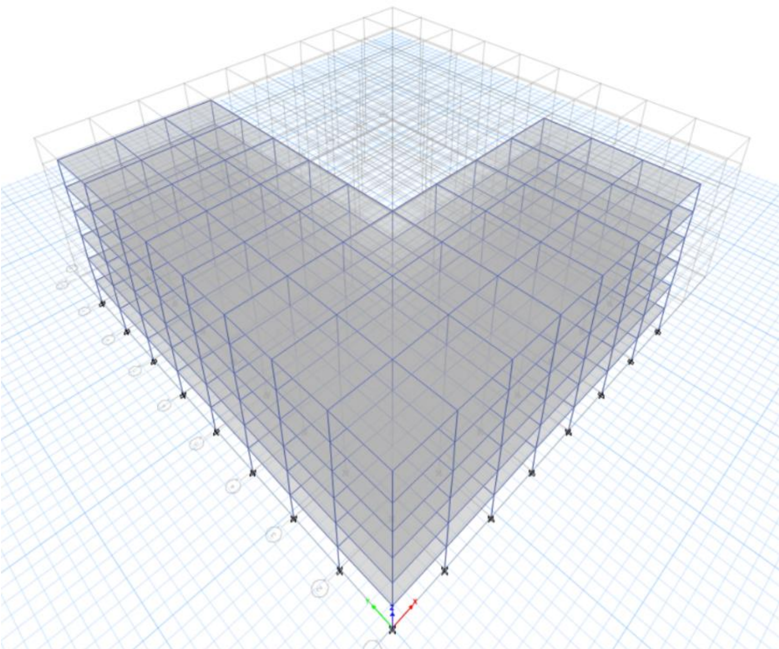


Fig. 6.4 : 3d view of structure

Table 6.2– Earthquake Data

Earthquake	Magnitude	Year	Station Name	Damping Ratio
Denali, Alaska	7.9	2002	Anchorage-K2-05	5%

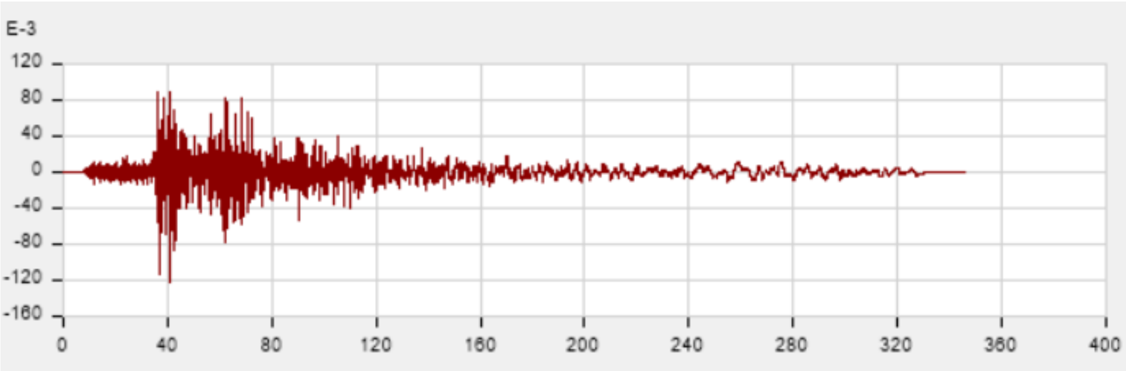


Fig. 6.5 : Earthquake in X-Direction

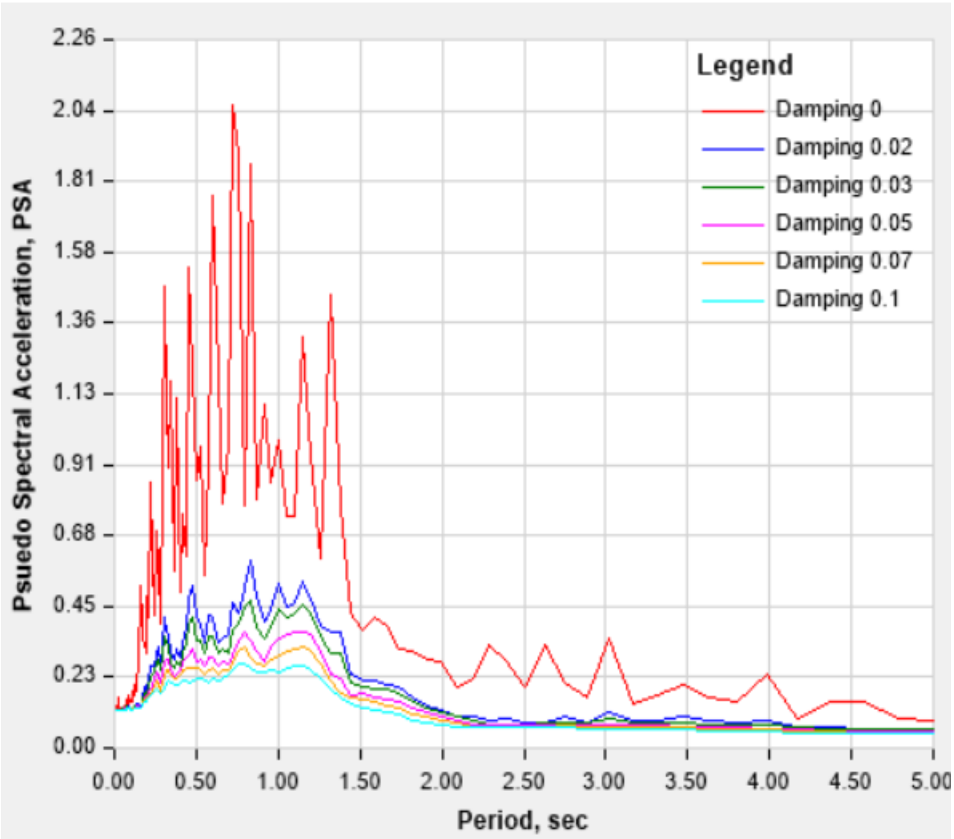


Fig. 6.6 : Target Response Spectrum in X-Direction

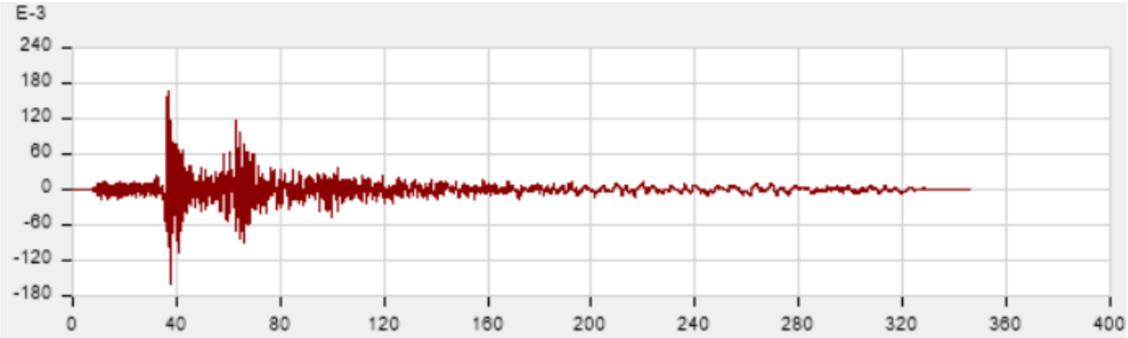


Fig. 6.7 : Earthquake in Y-Direction

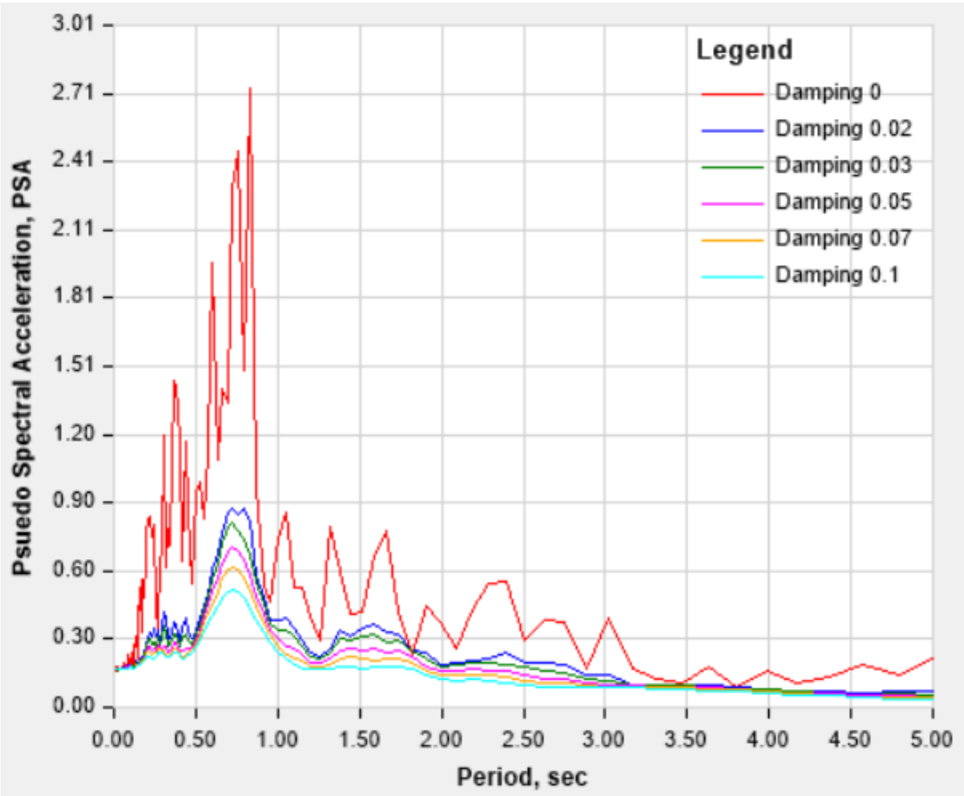


Fig. 6.8 : Target Response Spectrum in Y-Direction

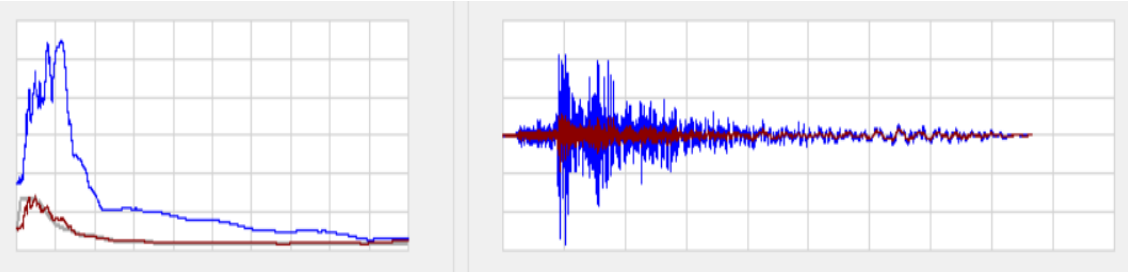


Fig. 6.9 : Spectral Matching of Earthquake in X-Direction

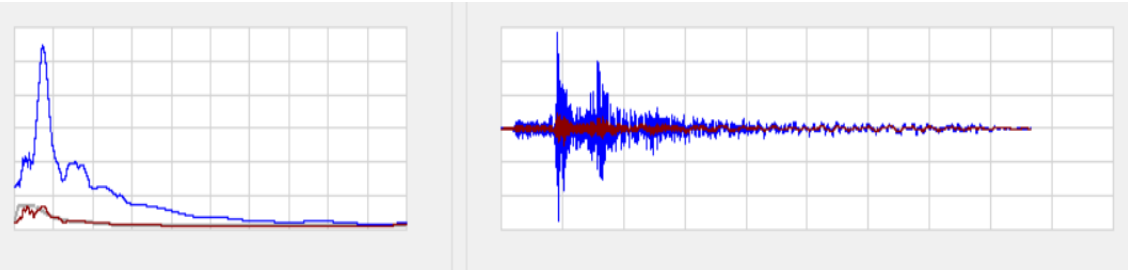


Fig. 6.10 : Spectral Matching of Earthquake in Y-Direction

NOTE- Spectral matching for earthquakes is a technique used to compare observed seismic waveforms with synthetic waveforms to determine key characteristics of an earthquake. By analyzing the frequency content, amplitude, and arrival times of the seismic waves, scientists can estimate parameters like the earthquake's location, magnitude, and focal mechanism. This method plays a crucial role in earthquake source characterization, seismic hazard assessment, and the development of early warning systems. Spectral matching provides valuable insights into earthquake behaviour, helps improve seismic monitoring and prediction, and enhances our understanding of earthquake processes. Its accurate analysis of seismic waveforms contributes to better preparedness and mitigation strategies for seismic events.

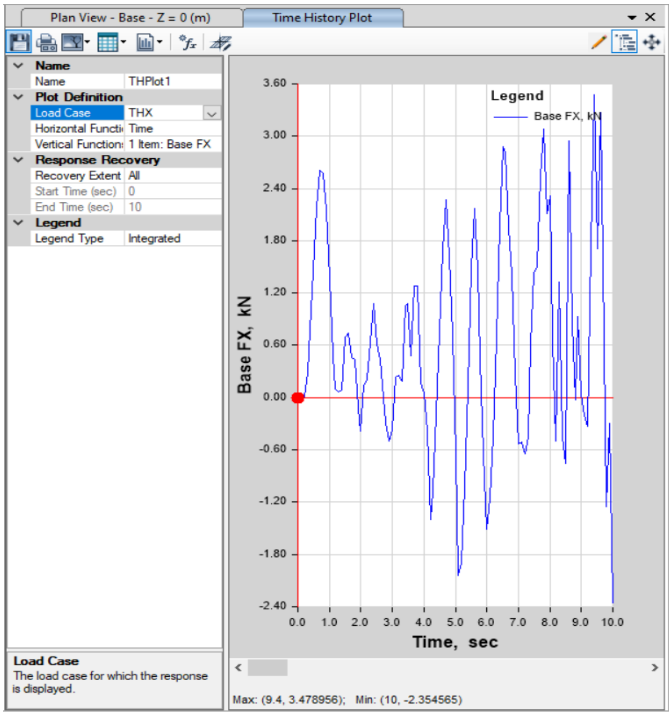


Fig. 6.11 : Plot of Time History Function for X-Direction

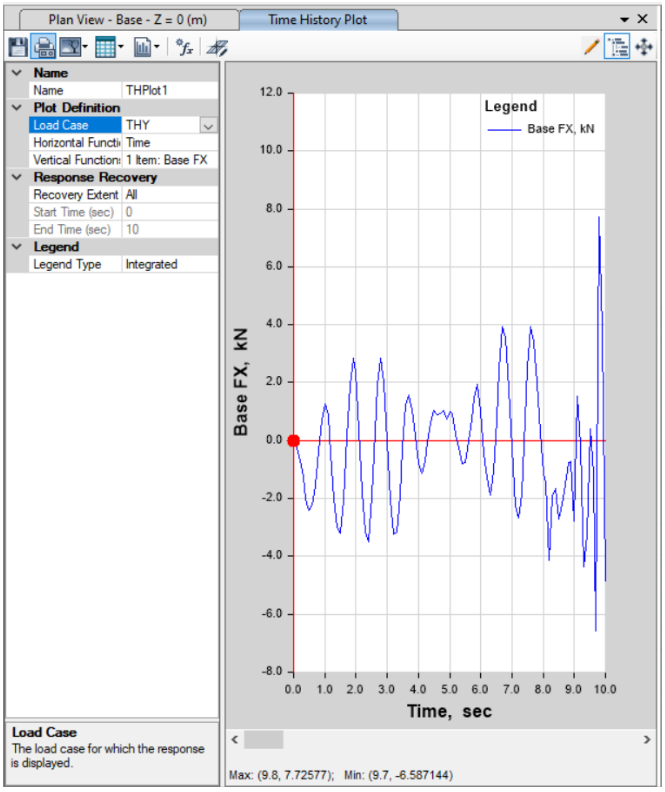


Fig. 6.12 : Plot of Time History Function for Y-Direction

6.2 COMREL Analysis

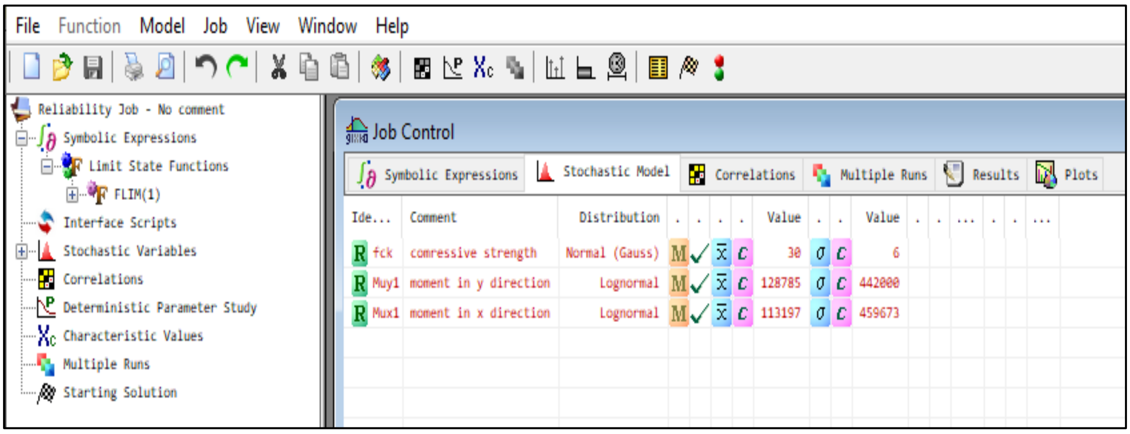


Fig. 6.13 : Input Variables for COMREL in Numerical form

Fig. 6.14 : Input Variables for COMREL in Graphical form

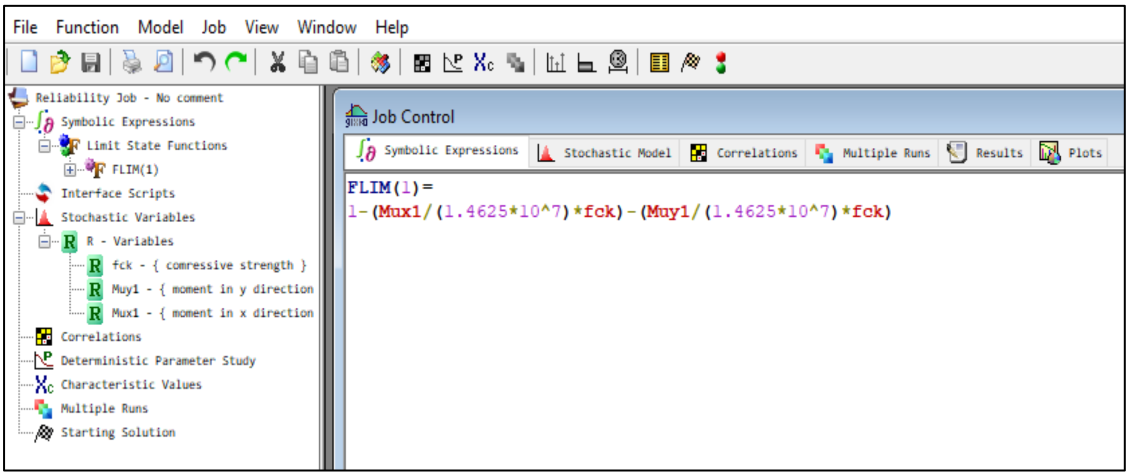


Fig. 6.15 : Defining the limit state function

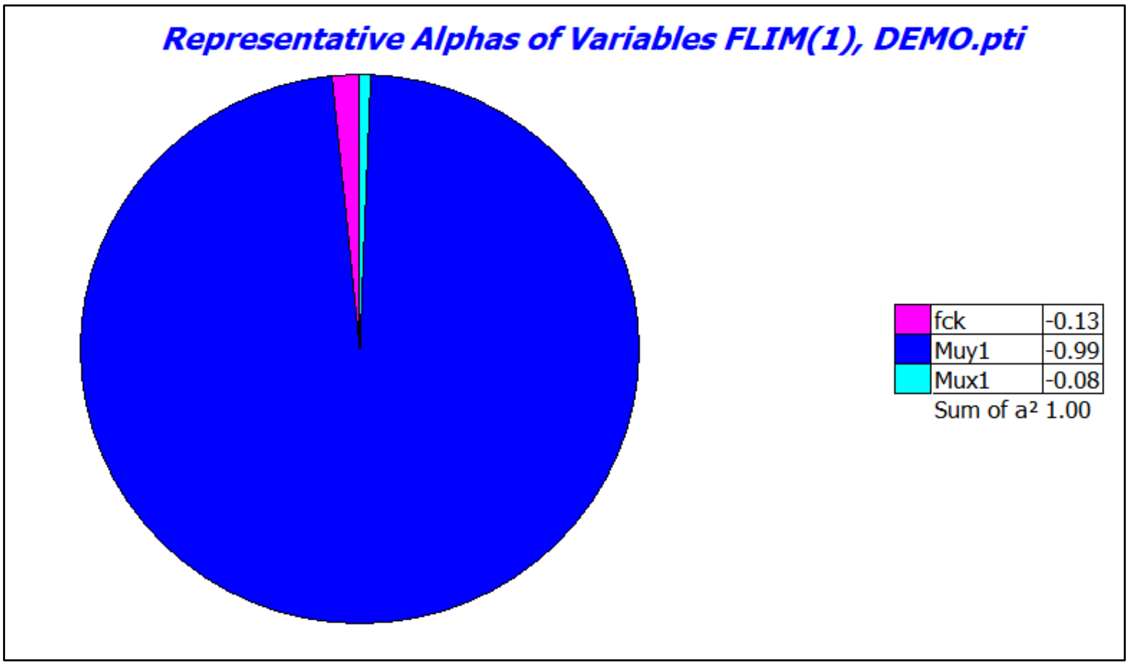


Fig. 6.16 : Representative alphas of variables

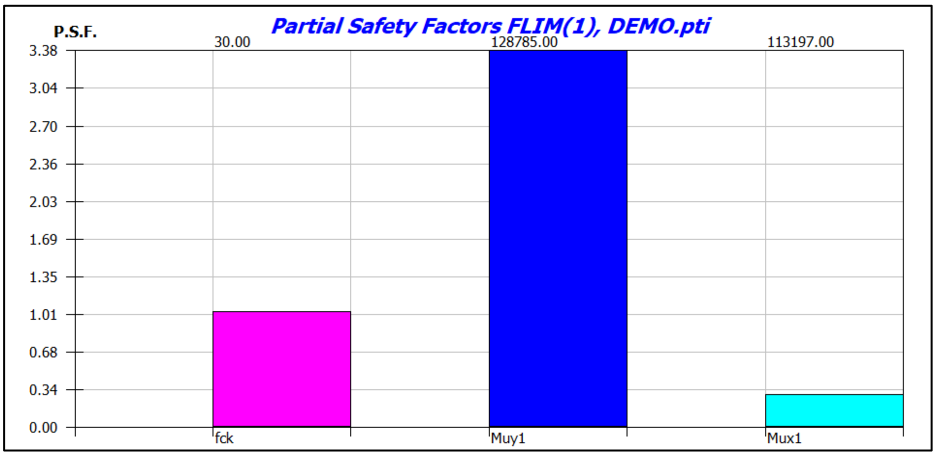


Fig. 6.17 : Partial Safety of Factors

<u>Numerical Result</u>

```
 Job name ............ : DEMO
 Failure criterion no. :      1
 Comment : No comment
 Transformation type   : Rosenblatt
 Optimization algorithm: RFLS
 Date(dd.mm.yyyy) .... : 22.05.2023
 Time(hh:mm) ........  : 21:13
 Comrel-TI, (Version 2023), Copyright: RCP GmbH (1989-2023)
------------------------------------------------------------------------------

Block beginning with Keyword $CHARVAL not found in Input-file !
Characteristic Values default to mean values.

Create buffer for $LIMFUNC with      2888 Bytes (Ier from Fstat =   0)

   ----------------------------------------
                 COMREL-TI
   Iteration monitoring of YBETAU algorithm
   ----------------------------------------
      General enviroment information
      Maximal number of iterations in RFLS1 =    100
      Maximal number of iterations in RFLS2 =    100
      Type of numerical derivatives         =      0
      Precision for convergence criteria    =   1.0000E-03
      Line search precision                 =   1.0000E-01

   Initial U-space solution
         0.0000      0.0000      0.0000

   Algorithm 1 : Iteration history
------------------------------------------------------------------------------
Iteration No.  1; CPU-seconds(cumulative):     0.000
 Scaled St.F(U) =  0.9508    ; BETA =      0.0000; BETA/||U||=      0.0000
------------------------------------------------------------------------------
Iteration No.  2; CPU-seconds(cumulative):     0.008
 Scaled St.F(U) =  0.9059    ; BETA =      0.2417; BETA/||U||=      0.5837
------------------------------------------------------------------------------
Iteration No.  3; CPU-seconds(cumulative):     0.008
 Scaled St.F(U) =  0.8642    ; BETA =      0.4138; BETA/||U||=      0.7558
------------------------------------------------------------------------------
Iteration No.  4; CPU-seconds(cumulative):     0.008
 Scaled St.F(U) =  0.8136    ; BETA =      0.5471; BETA/||U||=      0.7990
------------------------------------------------------------------------------
Iteration No.  5; CPU-seconds(cumulative):     0.008
 Scaled St.F(U) =  0.6318    ; BETA =      0.6837; BETA/||U||=      0.6543
```

```
--------------------------------------------------------------------------------
 Iteration No.  6; CPU-seconds(cumulative):     0.008
  Scaled St.F(U) = -0.7178    ; BETA =      1.0400; BETA/||U||=      0.5072
--------------------------------------------------------------------------------
 Iteration No.  7; CPU-seconds(cumulative):     0.023
  Scaled St.F(U) = -0.1726    ; BETA =      1.9819; BETA/||U||=      1.1700
--------------------------------------------------------------------------------
 Iteration No.  8; CPU-seconds(cumulative):     0.023
  Scaled St.F(U) = -0.1539E-01; BETA =      1.6787; BETA/||U||=      1.0566
--------------------------------------------------------------------------------
 Iteration No.  9; CPU-seconds(cumulative):     0.023
  Scaled St.F(U) = -0.3625E-03; BETA =      1.5880; BETA/||U||=      1.0056
--------------------------------------------------------------------------------
 Iteration No. 10; CPU-seconds(cumulative):     0.023
  Scaled St.F(U) = -0.1503E-04; BETA =      1.5790; BETA/||U||=      1.0001
--------------------------------------------------------------------------------
 Iteration No. 11; CPU-seconds(cumulative):     0.023
  Scaled St.F(U) = -0.8270E-06; BETA =      1.5788; BETA/||U||=      1.0000
--------------------------------------------------------------------------------
 Iteration No. 12; CPU-seconds(cumulative):     0.023
  Scaled St.F(U) = -0.4518E-07; BETA =      1.5788; BETA/||U||=      1.0000

    Statistics after RFLS-algorithm #1 (YRFLS1)
    Cumulative seconds used         :       0.0234
    Number of iterations            :     12
    Cumulative gradient calls       :     12
    Cumulative state function calls:     54
    State function scaling          :   8.7058E-01

    Statistics after beta-point search :
    Cumulative seconds used         :       0.0234
    Number of iterations(RFLS-1+-2):     12
    Cumulative gradient calls       :     12
    Cumulative state function calls:     54

    Transfer to GUI: NBV=   3;  NPVEC=   0

    -----Vector U-mem plus FU at solution                  -----
      0.2025          1.561          0.1279        -0.4518E-07

    -----Vector X at solution transfered to GUI            -----
       31.21          0.4349E+06     0.3361E+05

    -----Vector of constant Parameters transfered to GUI   -----


    beta-value to GUI:      1.5788;  Pf-value to GUI:  5.7189E-02
```

Result-

FORM-beta	1.579
FORM-Pf	5.72e-02

CHAPTER 7

RESULTS AND CONCLUSIONS

A reliability index value of 1.58 suggests a moderate level of reliability for a structure. The reliability index, often denoted as β, quantifies the margin of safety between the applied loads and the capacity of the system. It is typically calculated based on probabilistic analysis considering the variability and uncertainties in the relevant parameters.

In general, a higher reliability index value indicates a greater level of reliability and a larger safety margin. A value of 1.58 suggests that the structure has some margin of safety, but it may not be as robust as structures with higher reliability index values.

The interpretation of the reliability index value also depends on the specific industry and design standards. Different industries and jurisdictions may have different acceptable limits for the reliability index based on the consequences of failure, risk tolerances, and specific design codes.

It is observed that the reliability index value should be evaluated in conjunction with other factors, such as the consequences of failure, the criticality of the structure, and the specific design requirements, to determine if the structure meets the desired level of reliability and safety.

It should be noted that with varying cross-section of structural elements i.e. beams and columns, and also the compressive strength of concrete, we can see a change in the probability of failure of the structure which shows a change in the reliability index.

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APPENDIX

Element Output Data for Column No.1 of Story No.1(Base):

					m	kN	kN-m	kN-m	kN
Story	**Column**	**Output Case**	**Case Type**	**Step Type**	**Station**	**P**	**M2**	**M3**	**P ABS**
Story1	C1	UDConS1	Combination		0	-596.1778	10.8104	13.459	596.18
Story1	C1	UDConS1	Combination		1.285	-584.1345	13.3298	15.2388	584.13
Story1	C1	UDConS1	Combination		2.57	-572.0912	15.8492	17.0186	572.09
Story1	C1	UDConS2	Combination		0	-841.8375	5.6317	8.3246	841.84
Story1	C1	UDConS2	Combination		1.285	-829.7942	14.9137	16.8305	829.79
Story1	C1	UDConS2	Combination		2.57	-817.7509	24.1958	25.3364	817.75
Story1	C1	UDConS3	Combination	Max	0	-673.3684	4.5147	6.7971	673.37
Story1	C1	UDConS3	Combination	Max	1.285	-663.7337	11.9362	13.5472	663.73
Story1	C1	UDConS3	Combination	Max	2.57	-654.0991	19.358	20.2974	654.10
Story1	C1	UDConS3	Combination	Min	0	-673.642	4.4945	6.4712	673.64
Story1	C1	UDConS3	Combination	Min	1.285	-664.0074	11.9248	13.3628	664.01
Story1	C1	UDConS3	Combination	Min	2.57	-654.3727	19.3552	20.234	654.37
Story1	C1	UDConS4	Combination	Max	0	-673.298	4.5162	6.8481	673.30
Story1	C1	UDConS4	Combination	Max	1.285	-663.6634	11.9371	13.566	663.66
Story1	C1	UDConS4	Combination	Max	2.57	-654.0287	19.3581	20.3043	654.03
Story1	C1	UDConS4	Combination	Min	0	-673.5717	4.496	6.5223	673.57
Story1	C1	UDConS4	Combination	Min	1.285	-663.937	11.9257	13.3816	663.94
Story1	C1	UDConS4	Combination	Min	2.57	-654.3024	19.3552	20.2409	654.30
Story1	C1	UDConS5	Combination	Max	0	-673.1832	4.5252	6.9575	673.18
Story1	C1	UDConS5	Combination	Max	1.285	-663.5486	11.942	13.5866	663.55
Story1	C1	UDConS5	Combination	Max	2.57	-653.9139	19.359	20.3482	653.91
Story1	C1	UDConS5	Combination	Min	0	-673.6831	4.4848	6.2664	673.68
Story1	C1	UDConS5	Combination	Min	1.285	-664.0484	11.9196	13.2861	664.05
Story1	C1	UDConS5	Combination	Min	2.57	-654.4138	19.3545	20.1894	654.41
Story1	C1	UDConS6	Combination	Max	0	-673.257	4.5259	7.0529	673.26
Story1	C1	UDConS6	Combination	Max	1.285	-663.6223	11.9423	13.6427	663.62
Story1	C1	UDConS6	Combination	Max	2.57	-653.9877	19.3587	20.3489	653.99
Story1	C1	UDConS6	Combination	Min	0	-673.7568	4.4855	6.3618	673.76
Story1	C1	UDConS6	Combination	Min	1.285	-664.1221	11.9199	13.3423	664.12
Story1	C1	UDConS6	Combination	Min	2.57	-654.4875	19.3542	20.1901	654.49
Story1	C1	UDConS7	Combination	Max	0	-596.0508	10.8221	13.6307	596.05
Story1	C1	UDConS7	Combination	Max	1.285	-584.0075	13.3364	15.3423	584.01
Story1	C1	UDConS7	Combination	Max	2.57	-571.9642	15.851	17.0539	571.96
Story1	C1	UDConS7	Combination	Min	0	-596.3929	10.7968	13.2234	596.39
Story1	C1	UDConS7	Combination	Min	1.285	-584.3495	13.3221	15.1118	584.35
Story1	C1	UDConS7	Combination	Min	2.57	-572.3062	15.8474	16.9746	572.31

Story1	C1	UDConS8	Combination	Max	0	-595.9628	10.824	13.6945	595.96
Story1	C1	UDConS8	Combination	Max	1.285	-583.9195	13.3375	15.3658	583.92
Story1	C1	UDConS8	Combination	Max	2.57	-571.8762	15.8511	17.0626	571.88
Story1	C1	UDConS8	Combination	Min	0	-596.3049	10.7987	13.2872	596.30
Story1	C1	UDConS8	Combination	Min	1.285	-584.2616	13.3233	15.1353	584.26
Story1	C1	UDConS8	Combination	Min	2.57	-572.2183	15.8475	16.9833	572.22
Story1	C1	UDConS9	Combination	Max	0	-595.8194	10.8352	13.8313	595.82
Story1	C1	UDConS9	Combination	Max	1.285	-583.776	13.3436	15.3915	583.78
Story1	C1	UDConS9	Combination	Max	2.57	-571.7327	15.8523	17.1174	571.73
Story1	C1	UDConS9	Combination	Min	0	-596.4442	10.7847	12.9674	596.44
Story1	C1	UDConS9	Combination	Min	1.285	-584.4008	13.3156	15.0159	584.40
Story1	C1	UDConS9	Combination	Min	2.57	-572.3575	15.8466	16.9189	572.36
Story1	C1	UDConS10	Combination	Max	0	-595.9115	10.8361	13.9505	595.91
Story1	C1	UDConS10	Combination	Max	1.285	-583.8682	13.344	15.4616	583.87
Story1	C1	UDConS10	Combination	Max	2.57	-571.8249	15.8519	17.1183	571.82
Story1	C1	UDConS10	Combination	Min	0	-596.5363	10.7856	13.0866	596.54
Story1	C1	UDConS10	Combination	Min	1.285	-584.493	13.316	15.0861	584.49
Story1	C1	UDConS10	Combination	Min	2.57	-572.4497	15.8462	16.9198	572.45
Story1	C1	UDConS11	Combination	Max	0	-357.5797	6.4979	8.2471	357.58
Story1	C1	UDConS11	Combination	Max	1.285	-350.3537	8.0044	9.2468	350.35
Story1	C1	UDConS11	Combination	Max	2.57	-343.1277	9.5113	10.2465	343.13
Story1	C1	UDConS11	Combination	Min	0	-357.9217	6.4727	7.8398	357.92
Story1	C1	UDConS11	Combination	Min	1.285	-350.6957	7.9902	9.0163	350.70
Story1	C1	UDConS11	Combination	Min	2.57	-343.4697	9.5077	10.1672	343.47
Story1	C1	UDConS12	Combination	Max	0	-357.4917	6.4998	8.3109	357.49
Story1	C1	UDConS12	Combination	Max	1.285	-350.2657	8.0056	9.2703	350.27
Story1	C1	UDConS12	Combination	Max	2.57	-343.0397	9.5114	10.2551	343.04
Story1	C1	UDConS12	Combination	Min	0	-357.8338	6.4745	7.9036	357.83
Story1	C1	UDConS12	Combination	Min	1.285	-350.6078	7.9913	9.0398	350.61
Story1	C1	UDConS12	Combination	Min	2.57	-343.3818	9.5078	10.1759	343.38
Story1	C1	UDConS13	Combination	Max	0	-357.3482	6.5111	8.4477	357.35
Story1	C1	UDConS13	Combination	Max	1.285	-350.1222	8.0117	9.296	350.12
Story1	C1	UDConS13	Combination	Max	2.57	-342.8962	9.5126	10.3099	342.90
Story1	C1	UDConS13	Combination	Min	0	-357.973	6.4605	7.5838	357.97
Story1	C1	UDConS13	Combination	Min	1.285	-350.747	7.9837	8.9204	350.75
Story1	C1	UDConS13	Combination	Min	2.57	-343.521	9.5069	10.1114	343.52
Story1	C1	UDConS14	Combination	Max	0	-357.4404	6.512	8.5669	357.44
Story1	C1	UDConS14	Combination	Max	1.285	-350.2144	8.0121	9.3661	350.21
Story1	C1	UDConS14	Combination	Max	2.57	-342.9884	9.5122	10.3109	342.99
Story1	C1	UDConS14	Combination	Min	0	-358.0652	6.4614	7.7031	358.07
Story1	C1	UDConS14	Combination	Min	1.285	-350.8392	7.9841	8.9906	350.84
Story1	C1	UDConS14	Combination	Min	2.57	-343.6132	9.5065	10.1124	343.61

The respective mean and standard deviation of the moments:

M2 MEAN	M2 STD	M3 MEAN	M3 STD
11.31973	4.596753	12.8785	4.419995