RELIABILITY ASSESSMENT OF RC CONTINUOUS BEAM DESIGNED TO BS 8110 (1985) CRITERIA

By

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DECLARATION

I hereby declare that myself composed this thesis and it is a record of my research. It has not been accepted in any previous application for a higher degree.

All quotations and their sources acknowledged by means of references are specifically acknowledged by means of references

……

Sign.

CERTIFICATION

This thesis entitled "RELIABILITY ASSESSMENT OF RC CONTINUOUS BEAM DESIGNED TO BS. 8110 CRITERIA" by TALIAT OLA YUSUF meets the regulations governing the award of Masters of Science degree in STRUCTURAL ENGINEERING of the Ahmadu hello University and is approved for its contribution to knowledge and Literary presentation.

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DEDICATION

This Thesis is Dedicated to my late parents, Alhaji Yusuf Atanda Balogun and Madam Ayishat Igbayilola Balogun who taught me virtues of humility, patience and perseverance among others. These have contributed to my ability to seek and impart knowledge. May Almighty Allah grant them with Jannat Firdaus. Amin

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ABSTRACT

The provision of a safe structural system is an object of structural design. Therefore the way safety is prescribed in design is concern for Engineers. It is generally agreed in codified designs that decisions are made under uncertainties and as such safety consideration is best done in a probabilistic format. It is only in this way that consistent and uniform safety level can be ensured. The limit state method assumes a probabilistic concept and it is on this basis that BS. 8110 (1985) for reinforced concrete design was developed. However the implicit reliability in designing to the code specification is not obvious in practical designs.

The BS. 8110 (1985) is assessed using a two span reinforced concrete beam to ascertain the consistency and uniformity of reliability level of designs to the code specifications. It is found out that reliability indices depend on the design input especially the load ratio, meaning that under variable load the reliability will be inconsistent. The results also show that the conventional method will produce designs with reliability index taking on a value among many possible ones. This implies that the designer does not have control over the choice of a required reliability level. Consequently, the work provides reliability plots that can be used as design charts to specify design variables for known reliability levels.

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CHAPTER ONE

INTRODUCTION

1.1 BACKGROUND TO THE STUDY

The field of Structural Engineering basically emanated from the need to provide safe and functional structures within the dictates of system and environmental constraints. These objectives require an intuitive understanding of the interaction of design variables and resultants necessary to keep the system in the safe mode. This is necessary so as to prevent an event of failure and its associated social economic problems. The effort of Structural Engineers thus primarily centres on providing rational analyses and discourses on theories and that in the final analysis border directly or indirectly on precision and safety consideration of structures.

The various theories on mechanical properties of structures and materials, of forms and shapes, and structural techniques, are tools employed to provide knowledge on the possible states of a structure with which the Engineer to makes design decision regarding safety and economy. It can be recognised that consideration for structural safety is a fundamental requirement in making provision of engineering facilities.

The way safety is prescribed is however dependent on the level of sophistication of prevalent engineering knowledge. Conwan (1971), observed that before the eighteenth century, structures were constructed based on empirical rules, established by experience and changing circumstances.

This was obviously due to the fact that at that time little was known about the mechanical properties of structures and materials. It was also obvious judging from massiveness of

the structural legacy of that period, that specification of large structural sections and sizes; and relative non—complexity of structural forms made adherence to empirical rules feasible.

Developments in modern structural engineering which started with the use of scientific methods of experiment prediction to analyse structural response in relation to actions led to the formulation of deterministic models to predict the state of the structure. The expressions assumed an exact state of the structure which is not possible considering random nature of input variables. Even though uncertainty in the state of the structure is acknowledged by the prescription of safety factors, these factors are themselves deterministic in the sense that it is perceived that once a factor of safety is applied to a deterministic value, safety is guaranteed. Deterministic safety factors derivation is not adequately rationalised. Freudenthal (1963)stated that:

> "The most careful and rigorous structural analysis is largely deprived of its merits if the accuracy of its results is limited by the use of empirical multipliers, so—called safety factors varying between 1.0 and 5.0 and selected rather arbitrarily on the basis of consideration that are not always relevant or even rational"

There is a consensus among researchers in structural safety that deterministic approach to matter concerning safety can best be described as inadequate considering the lack of categorical statement that can be made on the practical values of load, strength and response. According to Ang (1973), absolute safety of a structure cannot be guaranteed due to uncertainties. Surhman et al (1983), state that "recognising that load and resistance

are random variables, the rational approach to design would be through the use of probabilistic method." Ang and et al (1974), also noted that the scatter exhibited by the design variables introduces the need for probabilistic approach to reliability level determination.

Structural reliability methods are developed in response to the realisation of the need for probabilistic approach. The methods provide bases for a rational discuss of structural safety problems.

1.2 THE NEED FOR A RATIONAL APPROACH TO SAFE STRUCTURAL DESIGN

Structural Engineering is concerned with solving structural problems which are characterised by randomness and their solutions must therefore take into consideration inherent uncertainties. There exist uncertainties in the response of structures due to the randomness of load borne by the structures as well as variable material properties. Some of the uncertainties have been identified as (Ang (1973)):

i) unpredictability of the future loading conditions,

ii) inability to obtain and express the in—place material properties accurately,

iii) the use of simplified assumptions in predicting the behaviour of the structure due to the loading under consideration,

iv) the limitations in the numerical methods used, and human factor e.g. errors, omission e.t.c.

These are broadly classified as phenomenological, decision, modeling, physical and

statistical uncertainties as well as human factor error (Afolayan, 1992).

In design, it is assumed that state of the structure can either be in the safe or failure modes. Design input specification can therefore take on values in the safe set failure domain. Usually in practice, we cannot be sure that values in the failure will not occur. This brings about reliability problems. The safety requirement arises from the random nature of the design parameters. Design therefore must seek to achieve and acceptable probability that structures will perform satisfactorily or will not fail, within set criteria. Failure in this instance may be defined as a condition of total collapse of the structure or an element thereof, or show functional deficiency.

We need to have a measure of reliability according to which we design the system to have any specified degree of safety. In designing for safety, the principle involve assessing load effects that lead to the worst conditions in the life time of the structure and the capacity of materials to resist the loads. A substantial margin is then provided to take care of the inherent uncertainties. The way the margin is specified will vary widely from one Engineer to another if left to the judgement of individual Engineers. To standardise decision making with respect to analysis and design for safety, design codes are developed. Code specifications are assumed to have taken into consideration all matter relating to safety and of course economy of the designed structures.

Until recently code parameters were determined by judgement rather than by rational analysis (Madsen et al, (1986)) as in the case of allowable stress method where an empirical factor of safety is applied to ultimate stress to obtain allowable stress. A code parameter becomes acceptable if the code, having being in use for some period of years,

proved to have performed poorly. The need for precision and optimisation has brought about continual review of codes of practice and it is discovered that it can be done most rationally when probabilistic approach is considered (Ang, 1973; Surhman et al, 1983; Ang et al, 1974). According to Brown (1960), it is the obligation of a Structural Engineer to back up his safety consideration with as much logic and rational justification for safety margins as necessary. In pursuance of rational approach there is an international focus of researchers in applying probability concepts in analysing safety and uncertainties and it has been shown that in developing a consistent code, design criteria are most rationally formulated within a probabilistic framework (Ellingwood et al, 1974). Saul et al (1986), also remarked that implementation of probabilistic based codes procedures would provide harmonious philosophy to design and construction of structures.

Structural reliability method which evolved as a result of probabilistic consideration of safety provides a rational basis for analysis of safety. It is an analytical tool that can be utilised to arrive at either a point or an interval estimate of the probability that the structural system will perform its specified function without failure when subjected to loads and other environmental factors. The object of reliability analysis is to emphasise the systematic and consistent use of analytical tools with a view to providing bases for reducing the uncertainty involved in the reliability estimate (Afolayan, 1992). The performance of reliability analysis will among other things indicate

i) the effects of increasing or decreasing safety factors on the structural reliability

ii) the sensitivity of the structural reliability to other design parameters

iii) the additional information needed to improve accuracy of the reliability estimate

From this understanding consistent and systematic ways of making design decisions will emerge.

1.2.1 Limit State Design Method

A method of design is defined by the way the margin for safety is prescribed through a factor of safety either applied to the strength, the loading or combination of both. Mosley et al (1989), explained that the permissible stress method in which designs are obtained by applying a safety factor to the ultimate strength of the materials does posses some inconsistencies resulting from the arbitrary way the permissible stress is defined and that it is unsuitable for semi—plastic materials, non-linear structures and stability of structures subjected to overturning forces. Also the load factor method is deficient because it does not take variability of strength into consideration.

As earlier observed, the permissible stress and load factor methods though admit uncertain nature of structural design through the application of safety factors, do not consider rational reliability concept in their derivation. The introduction of probability based safety analyses allows for developments of formal methods to obtain probability of failure through systematic analysis of the uncertainties in all variables, although Ayyub et al (1984), remarked that this development does not eliminate the need for one's discretion, rather the ways in which judgement is exploited is presumably more effective.

The limit state design concept resulted from the probabilistic considerations and it is assumed to be more logical in its presentation of safety margin. The concept aims at achieving a consistent and acceptable probability that structures being designed will perform satisfactorily during their intended life (BS. 8110, Part 1, 1985). The philosophy

involves setting limits for various design criteria and minimising the chances of the limits being exceeded. Common among possible criteria are ultimate limit and serviceability limit states.

Although the limit state concept is probability based, the existing codes on limit state design especially with regard to reinforced concrete design, do not reveal the extent of reliability of a particular structure or its components, consequent upon which the proof of consistency or uniformity in reliability levels of structures designed to the existing limit state code specification are not apparent.

1.3 RESEARCH GOAL

The current thinking is that a rational safety analysis should provide a uniform and consistent reliability for effective optimisation of safety and economy. This should be the object of any design code. In normal situation, designs carried out using design codes to predict expected performance of structures subjected to varying design inputs given the same criteria should produce uniform reliability.

In their calibration, the existing limit state design codes on reinforced concrete provide partial safety factors which presumably cater for uncertainties inherent in design variables. Examples are British Standard code of practice (CP 110) of 1972 and the more recent British Standard Structural Use of Concrete (BS 8110) of 1985.

The method of partial factors of safety can best be described as ‘modified deterministic’ in the sense that a quantity displaying uncertainty is considered in terms of means and standard deviation and are subsequently employed in deterministic operations using a

constant characteristic value in place of the uncertain quantity. This raises some questions as to the consistency and uniformity in reliability of resultant designs. To make matter worse, adherence to design codes does not show the reliability to which designs are carried out leaving the Engineer to accept that his design is safe without information on the extent of reliability.

This research aims at investigating the implicit safety level provided by designing to B.S.8110 (1985) specifications using structural reliability methods.

1.4 SPECIFIC OBJECTIVES

The specific objectives of the research are:

i) To appraise the implicit reliability of BS. 8110 (1985) for uniformity and consistency in design of reinforced concrete beams

ii) To propose a basis for consistent design of rein forced concrete beams through the use of reliability charts.

1.5 METHODOLOGY

This study will be basically analytical in approach. The first task will be to form safety functions by obtaining the margin between strength functions and actions for the limit being considered. The basic variables in the limit state functions are considered as random variables. The means, standard deviations coefficients of variation and distributions of the basic variables will be obtained from literature on works done in the relevant areas of statistical test on loading and material strength.

Reliability calculation will then be performed using first order reliability method A

computer program, FORM5 (1988) will be used for the approximate computation of probability integrals. The program is capable of:

i) transforming non—guassian variables into independent standard normal variables locating the most likely failure point through optimisation procedure

ii) linearising the limit state function in that point, and

iii) estimating probability of failure using the standard normal integral.

The necessity for these will be realised in the course of the study.

Having determined the reliability indices for various conditions from the above procedures, the indices will be used to produce plots that will indicate implicit reliability levels. The plots will be used to produce design charts from which a known reliability can be prescribed for specific design in order to preserve uniformity in reliability level.

CHAPTER TWO

LITERATURE REVIEW

2.1 INTRODUCTION

Consideration for safety which requires additional strength to take care of uncertainty is as old as the evolution of design and construction of structures (Brown, 1960). However, probability-based safety analysis and Design was not recognized until 1950s when Freudenthal (1945), questioned the established thoughts on safety consideration which were mainly deterministic, and with the contribution of others like Carrelts et al, (1983), developed the fundamental concepts of random structural events.

In this chapter, important pioneering works in structural reliability leading to the development of limit state design philosophy are reviewed.

2.2 REVIEW OF DEVELOPMENTS IN STRUCTURAL RELIABILITY

The pioneering works on probability—based analysis were largely ignored because the attention Engineers then was mainly in the development of linear elastic and plastic analyses of structural systems (Madsen et al, 1986). Mayer (1926), and later Basler (1960), proposed safety measures similar to the Cornell (1967), safety index which was ignored and had no influence on design practices. Mayer (1926), suggested design based on the mean and variance of the random variables. Other pioneer researchers include Plum (1950), who established the apparent contradiction between economically optimum safety levels and much lower rates of failures observed in actual practice using reinforced concrete slabs in buildings. Johnson (1953), gave the first comprehensive presentation of

the theory of structural reliability and economic design. Among all the early researchers, the one whose work was the first to evoke a considerable acceptance among structural Engineers is Freudenthal(1947) who in his paper "The safety of structures" presented the fundamental problem of safety of a member under random variable load. However Madsen et al (1986) observed that structural engineers were reluctant to accept probability based structural design of early researchers professionally because deterministic design approach of that period apparently served well and as such no need for change. There were very few structural failures, which were simply attributed to human error when they occurred. Also the probabilistic design looked cumbersome and difficult mathematically and numerically, and there was also the problem of availability of adequate data to fully define tails of load and strength distribution.

With a view to circumvent these problems Lind (1964), and Tukstra (1970), established a design in an attempt to make decision under uncertainty and risk; the significance of which is that in the face of many alternative solutions to problems, the task will be to rationally determine the most suitable one. In order to avoid the sensitivity problem cause by inadequate data on populations, Cornell(1967), suggested the use of a second moment format. Second moment reliability theory expresses all uncertainties concerning the structure solely in terms of expected values (first moment) and covariance (second moment) of the input basic variables. Cornell's safety index requirement could be used to derive a set of safety factors on load and resistance.

Ravindra et al (1974), illustrated how to relate reliability analysis to existing design methods using the safety index method and discovered that "by code calibration, not only the design procedure but also the resulting designs are maintained identical to current

practice”

However some limitations to Cornell’s safety index were discovered with respect to definition of reliability index in cases of multiple random variables. Also, there were problems of invariance as discovered independently by Ditlevsen (1973) and Lind (1971). Hasofar et al (1978) solved the problem of invariance by proposing a non homogeneous linear mapping of the set of basic variables into a set of normalised and uncorrelated variables. The measure of reliability was defined as minimum distance from the origin to a point on the failure surface in the normalised space. At this stage, design codes were developed and guidelines contained in report of such bodies as Comite European du Beton (C.E.B. 1976) Construction Industry Research and Information (CIRIA, 1977) and Canadian Standard Association (CSA, 1981) reducing probabilistic base design into a routine. Having formulated the fundamental concept of reliability analysis, further researches focus on development of specific details. Ellingwood et al (1974), evaluated the risks associated with design procedures with reference to reinforced concrete. This is significant because these risks serve as the initial basis for the formulation of probability based design criteria. The determination of the minimum distance from the origin to the failure surface is an optimisation problem, which can be solved by algorithms. Rackwitz et al (1978), developed an iterative method to solve such an optimisation problem

2.3 THEORETICAL BASES FOR STRUCTURAL RELIABILITY

The fundamental concept of the classical theory of structural reliability involves identification of relevant load and resistance parameters known as basic variables and establishment of a functional relationship between these variables. In general the basic variables can include not only the physical quantities but also variables introduced to

allow for uncertainty in the models used to formulate the failure function and any other relevant uncertainty. Mathematically, the relationship is generally described as [31]

$g(x) = g(X1,X2,X3,\ldots\ldots Xn) <_ 0$ (2.1)

where

$g(x) = g(X1,X2,X3,\ldots\ldots\ldots\ldots Xn)=0$ (2.2)

is known as a limit state surface (or failure surface) and each X indicates the basic load or resistance variable. Failure occurs when

$g(X) < O$ (2.3)

The probability of failure Pf (or that a limit state will be reached) is given by [31]

$P_f = \int_D fx(t)dt$ (2.4)

in which $f_x(t) = fx_1, fx_2,\ldots\ldots\ldots xn(t_1,t_2,\ldots\ldots\ldots\ldots tn)$ (2.5)

this being the joint density function of $X_1, X_2, X_3\ldots\ldots\ldots Xn$, and D being the in the n-dimensional space and $dt=dt_l,dt_2\ldots.. dt_n$ (Shinozuka, 1983)

According to Shinozuka (1983) the problems associated with evaluation of eqn. (2.3) are:

i.) it is almost impracticable to construct the joint density function on the existing data base which is usually scarce and insufficient, particularly for the crucial range of large variable deviations, and

ii.) practically for all cases of engineering interest, the multidimensional integration over the generally irregular domain D is impossible to carry out analytically and quite costly to perform numerically if not impossible.

A way to circumvent these computational difficulties in the evaluation of risk is to confine safety statements to only variances. This way of expressing all uncertainties concerning structural reliability in terms of expected values and variances is known as the

First order second moment reliability method. This method is used to evaluate various reliability indices.

The failure surface separating the safe set and the failure set is defined by the limit state function [31].

$$g(xi) = 0 \quad (2.6)$$

where Xi are elements of the failure surface Lx.

The safety margin M is given as:

$$M = g(X) \quad (2.7)$$

Where Xi is the random variable

Cornell (1969), defined a reliability index $ß_C$ as

$$\beta_C = \frac{E(M)}{D(M)} \quad (2.8)$$

where, $ß_C$=Cornell's reliability index

E(M) = mean value of M

D(M) = standard deviation of M

The probability of failure is evaluated as

$$P_f = \Phi(-ß_C) \quad (2.9)$$

where, Φ(.) is the cumulative probability of standardised normal variate.

Cornell s safety function was simply given as the difference between a resistance r and load effect s; with the corresponding safety margin given as

$$M = R — S \quad (2.10)$$

The reliability index for uncorrelated random variables R and S from eqn. (2.8) becomes

$$\beta_c = \frac{E(R) - E(S)}{\sqrt{(VAR(R) + VAR(S))}} \quad (2.11)$$

$$= \frac{\mu_R - \mu_S}{\sqrt{\delta^2{}_R + \delta^2{}_S}} \tag{2.12}$$

For a hyper plane, the safety margin can be defined as

$$M = a_0 + \sum_{i=1}^{n} a x_i \tag{2.13}$$

$$= a_0 + [A]^T [x]$$

where

$[A]^T$ = a row vector with elements a_i and

$[x]$ = a column vector with elements x_1

The reliability index of the hyper plane is given as

$$\beta_C = \frac{a_0 + [A]^T [x]}{\sqrt{[A]^T C_x [A]}} \tag{2.14}$$

where E([x]) is the vector of expected values and

C_X is the matrix of covariances of [x]

2.3.1. First Order Second Moment Reliability Index

The limitation of Cornell's reliability index formulation was that once the failure surface is not a hyper plane, the expected values and variance of safety margin cannot be calculated just from the second—moment representation of the basic variables.

This problem can be solved in non—linear function of the basic variables by linearising the safety margin M using Taylor series expansion around the mean value point.

This is of the form [31]

$$M_{FO} = g([x]) + \sum_{i=n}^{n} \frac{\delta_g}{\delta_{xi}} [x](X_i - x_i) \tag{2.15}$$

and reliability index is

$$\beta_{FO} = \frac{g([x]) + \sum_{i=n}^{n} \frac{\delta_g}{\delta_{xi}} [x](X_i - x_i)}{\sqrt{\sum_{i=1}^{n} \sum_{j=1}^{n} \frac{\delta_g([x])}{\delta_{xi}} \frac{\delta_g([x])}{\delta_{xj}} \mathrm{cov}(x_i x_j)}} \tag{2.16}$$

This reliability index is called a first-order second moment reliability index and faced with problem of arbitrariness. An improved index estimate was proposed by Hasofar et al (1978) thus solving the problem of arbitrariness.

2.3.2 Hasofar and Lind safety index

Hasofar et al (1978), proposed a linear transformation of the set of basic variables into a set of normalised and uncorrelated variables U with vector of expected values

$$E[U] = 0 \tag{2.17}$$

and covariance matrix

$$C_U = \mathrm{cov}[U, U^T] = I \tag{2.18}$$

where I is the unit matrix.

The failure surface Lx in X—space is mapped onto the corresponding failure surface Lu in U—space . The geometrical distance from the origin in u—space to any point on L_U is the number of standard deviation from the mean—value point in x—space to the corresponding point on L_X. Thus,

$$\beta_U = \sqrt{(U^T U)}, \qquad U \in L_U \tag{2.19}$$

or

$$\beta_x = \sqrt{\left([C_x - E([X])]^T C_x^{-1}([X] - E([X]))\right)}, \quad X \in L_X \tag{2.20}$$

Hasofar et al (1978), proposed the smallest distance from the origin to a point on the failure surface as the definition of a reliability index

$$\beta_{HL} = \min \sqrt{\left([C_x - E([X])]^T C_x^{-1}([X] - E([X]))\right)}, \quad X \in L_X \tag{2.21}$$

The solution point X is known as the design point or point of maximum likelihood.

Hasofar et al (1978), definition thus provides a generalisation of the definition by Cornell with non linear failure surfaces. The ambiguity in the value of the first—order reliability index is thus resolved when the design point is taken as the linearisation point.

2.3.3 Generalised First order Reliability Index

In general, the basic variables that are non guassian in practical situations and the non linear limit state functions are transformed, one to one, into independent standardised normal variables

$$T : X = (X_1, X_2, X_3 \ldots\ldots\ldots\ldots X_n) \rightarrow U = (U_1, U_2, U_3 \ldots\ldots\ldots U_n) \tag{2.22}$$

where the random variables, $U_1, U_2, U_3 \ldots\ldots$ Un are uncorrelated and standardised normally distributed given as

$$U_i = \frac{X_i - E(X_i)}{\sigma_i} \tag{2.23}$$

The limit state surface in X-space is mapped onto the corresponding limit state surface in U-space. The next step in an attempt to evaluate reliability will be to determine the design point which is the minimum distance β from origin to a point on the failure surface. The failure probability is then approximated to a first order by Φ(-β) corresponding to a

linearisation of the failure surface and the linearisation point is the design point U*.

A reliability method based on this procedure is known as generalised First order reliability method (FORM) and ß is the first order reliability index.

Determination of design point can be done by iteration algorithms. A faster and efficient algorithm was proposed by Rackwitz et al (1978) and it is suitable for complex problem of load combination. The iteration algorithm in U-space is given by [31]

$$U^{m+1} = \left(U^{(m)T}\delta^{m}\right)\delta^{m} + \frac{g_u\left(U^{(m)}\right)}{\nabla g_u\left(U^{(m)}\right)}\delta^{m} \tag{2.24}$$

where $\boldsymbol{\delta^{(m)}}$ is a unit vector normal to the limit state curve and it is given as

$$\delta^{m} = -\frac{\nabla g_u\left(U^{(m)}\right)}{\left|\nabla g_u\left(U^{(m)}\right)\right|} \tag{2.25}$$

The failure function in U—space is given in terms of the failure function g in X—space as

$$g(X^{(m)}) = g(T^{-l}(u)^{m})\, g_u(U^{(m)}) \tag{2.26}$$

$g(U^{(m)})$ is represented by the co—ordinates $U^{(\mathbf{m})}$ where $U^{(\mathbf{m})}$ is a number in sequence $U^{(0)}, U^{(1)}, U^{(2)}, \ldots\ldots\ldots\ldots U^{(m)}$, the limit of which is the design point U*.

The gradient vector $\nabla g_u(U^{(m)})$ is given as

$$\nabla g_u(U^{m}) = \frac{\delta g_u(U^{m})}{\delta_{11}}, \frac{\delta g_u(U^{m})}{\delta_{12}} \ldots\ldots\ldots \frac{\delta g_u(U^{m})}{\delta_{1n}} \tag{2.27}$$

The partial derivatives in eq. (2.27) are given as

$$\frac{\delta g_u}{\delta_{1i}}(U^{m}) = \sigma_i \frac{\delta g}{\delta_{xi}}(X^{m}) = \frac{\delta g_u(X^{m})}{\delta_{ni}} \tag{2.28}$$

The modulus of the gradient vector is also given as

$$\left|\nabla g_u(U^m)\right| = \sqrt{\left(\frac{\delta g_u(U^m)}{\delta_{11}}\right)^2 + \left(\frac{\delta g_u(U^m)}{\delta_{12}}\right)^2 + \ldots\ldots\left(\frac{\delta g_u(U^m)}{\delta_{1n}}\right)^2} \tag{2.29}$$

The algorithm in eq. (2.24) is expected to converge to U* at convergence

$$U^* = \beta\delta^m \tag{2.30}$$

And

$$\beta = \left(\delta^m\right)^T U^* \tag{2.31}$$

2.4 CLASSIFICATION OF RELIABILITY METHODS

Reliability analyses are classified into various levels of sophistication namely level I, level II, level III and level IV (Madsen et al. (1986)). It is necessary to identify these because the factor of safety method being adopted by most codes on limit state design is more of level I than level II since they employ one characteristic value of strength and load variables. For instance, if failure function in respect of a limit state is expressed in the form

$$g(X_i) \leq 0 \tag{2.32}$$

safety is ascertained by the inequality

$$g(\gamma X_i^*) \geq 0 \tag{2.33}$$

where X_i = basic variable,

X_i * = characteristic value of basic variable

γ = partial safety factor

In the Level I method, the characteristic values and partial safety factors are expected to provide approximate reliability level without an indication of actual reliability. Afolayan

(1992), observed that level I scheme is simple but the implied reliability varies from one case to case.

Level II method employs two values of each basic variable namely mean and variance . The method provides a measure of reliability for structures. All methods of reliability indices discussed in the previous sections are examples of level II scheme. This forms a basis for the current study.

Level III scheme requires the actual computation of probability of failure using our knowledge of the joint distributions of all uncertain variables. The method apparently is the most complete but equally the most difficult to apply to practical safety checking and design because of the analytical and numerical difficulties involved (Afolayan, 1992). However, they are best suited for checking approximate methods of levels I and II

Level IV method which is still undergoing developmental process employs the principles of engineering economic analysis and design under uncertainty, considering costs and benefits of construction , maintenance ,repair , consequences of failure and interest on capital (Madsen et al 1986

2.5 REVIEW OF LIMIT STATE DESIGN CODE

With the adoption of limit state design (LSD) philosophy , many design codes and manuals have been fashioned in line with this philosophy. Some of these are the British Standard code of practice (CP110, 1972), British Standard Code of Practice (BS. 8110, 1985), National Building Code of Canada (1980). There also exist Code Committee reports on LSD by bodies such as Nordic Committee on Building Regulations (NKB,

1978), and Comite European du Beton (CEB, 1976).

The basic philosophy involves classification of possible state of structural malperformance and setting limits for an acceptable probability of performance. In order to guarantee this level of acceptance, partial coefficients are applied to characteristic values of variables in the partial factor of safety method which is currently being adopted by virtually all the codes of practices mentioned.

Nodic Committee on Building Regulations report (1978) classified limit states as ultimate limit state which corresponds to the maximum load carrying capacity of the structure or its components; conditional limit state (a special case of the ultimate limit state) corresponding to the load—carrying capacity of a structure in which a local failure caused by accidental actions or fire has occurred. Serviceability limit states are related to the criteria governing normal use and durability and other limit states involving design for some special or unusual function.

According to NKB report (1978), the ultimate limit may be reached due to

i.) Transformation of the structure into a mechanism

ii.) Rupture or excessive plasticity, of critical sections of the structure to excedance of the material strength

iii.) rupture due to fatigue

iv.) instability (buckling, lateral buckling and turnover)

v.) loss of equilibrium of a part or the whole of the structure considered as rigid body

According to the same report, serviceability limit state may be reached due to

i.) excessive deflection affecting the appearance or efficiency of the structure

ii.) local damages which entail excessive maintenance or lead to corrosion

iii.) excessive vibrations producing discomfort.

Conditional limit states being a special case of the ultimate limit states have similar failure modes.

2.5.1 Loading and Safety factors

BS. 8110 (1985), specifies load combination methods and partial coefficient of safety for various types of loads and characteristic strength of materials. Tables 2.1 and 2.2 show the code specification of partial coefficients for various loading types and characteristic material strength.

For load combinations 1 and 2 in Table 2.1, the 'adverse' partial factor is applied to any loads that tend to produce a more critical design condition while the 'beneficial' partial factor is applied to any load that tends to produce a less critical design condition at the section being considered.

TABLE 2.1 Load Combinations and values of partial factors safety for loads (source: BS. 8110 1985, part 1)

LOAD COMBINATION	DEAD LOAD		IMPOSED LOAD		EARTH WIND	WIND
	Adverse	Beneficial	Adverse	Beneficial		
1.0-Dead + Impose (Earth + Water pressure)	1.4	1.0	1.6	0	1.4	-
2.0-Dead + Wind (Earth + Water pressure)	1.4	1.0	-	-	1.4	1.4
3.0-Dead + Wind + imposed (Earth + Water pressure)	1.2	1.2	1.2	1.2	1.2	1.2

TABLE 2.2 Values of Partial Safety Factors for Strength of Materials. (source: BS. 8110 part l 1985)

MATERIAL	PARTIAL COEFFICIENTS
1.0 Reinforcement	1.15
2.0 Concrete in flexure or axial load	1.5
3.0 Shear strength without shear reinforcement	1.25
4.0 Bond strength	1.4
5.0 Others (e.g. bearing stress)	>1.5

2.5.2 Method of Partial Factor of Safety

The method of partial factor of safety as applied to limit state design is based on characteristic values and partial coefficients for load and material strength. The assumption is that uncertainties in load and resistance are taken care of by the application

of partial factors of safety to their respective characteristic values.

The characteristic material strength is taken to be the value , below which results of standard tests are not likely to fall. For a standardised strength distribution it is taken as the value which it is not likely that more than 5.0 percent of them will fall (BS. 8110 1985) This is given by the expression

$$f_k = \mu + n\sigma \qquad (2.34)$$

where $n = \Phi^{-1}(\alpha)$

α =1, 2, 5 and 10 percent

n = -2.33, -2.05, -1.64, and -1.28

f_k = characteristic strength

μ = Mean Strength

σ = Standard Deviation

In the case of load, the characteristic value is the one within 90 percent of confidence level, that is not more than 5 percent of cases will exceed the upper limit and not more than 5 percent will fall below the lower limit.

NKB (1978), report characterised the partial coefficient for load (γ_f) as a function of two factors γ_{f1} and γ_{f2} with the former accounting for the likelihood of unfavourable deviation of the load from the characteristic values, modeling uncertainty and error of inaccurate assessment of the action effect; and the latter accounting for the reduced probability of combination of loads.

The report also classified partial coefficient of material properties Ym as a function of five factors. The first takes account of the unfavourable deviations of the material strength

and other properties from the characteristic values. The second takes care of possible inaccurate assessment of the resistance, randomness of geometrical parameters, as well as that part of the action effect which may be dependent upon the structural material.

Thirdly, a factor also takes account of possible deviation from the strength of material properties in the structure as compared to that derived from control test specimens.

Another factor account for the consequences and type of failure. Lastly, account is taken of the degree of control on site. All these are lumped into a single partial factor in the form y_f and Y_m for load and material strength respectively, giving design load and strength respectively as

$$W_d = \gamma(f_k) \tag{2.35}$$

and

$$f_d = \frac{f_k}{\gamma_m} \tag{2.36}$$

BS 8110 (1985) specifies partial factors as indicated in Tables 2.1 and 2.2

The NKB (1978), recommends reliability index according to failure type and consequences of failure as shown in Table 2.3 . The failure consequences are of three classes namely less serious ,serious and very serious.

In the first class **(**less serious**)**, risk to lives is negligible, and small social consequences are associated. Examples are Sheds, stock buildings, some agricultural buildings, non-load bearing internal walls and roofs.

In the serious class, there exist risk to lives and considerable social consequences. The very serious class portends great risk to lives and great social consequences.

Table 2.3 : Classification of Failure Types and consequences
Source :NKB (1978) Report

FAILURE CONSEQUENCES (Safety Classes)	FAILURE TYPES		
	I	II	III
Less Serious	**β** =3.1	**β**=3.7	**β**=4.2
Serious	**β** =3.7	**B**=4.2	**β**=4.7
Very Serious	**β** =4.2	**β**=4.7	**β**=5.2

Failure Type I is assumed to deal with ductile failures without any extra carrying capacity beyond the defined resistance is available.

Failure Type II includes ductile failure without any extra carrying capacity

Type III failure includes states of brittle failure and instability failures and similar failures.

CHAPTER THREE

DETERMINATION OF IMPLIED RELIABILITY LEVELS

3.1 INTRODUCTION

Analyses of a continuous beam using BS 8110 (1985) specifications will be performed with a view to determine the inherent reliability level. The first step in this direction involves idealisation of load and strength functions in accordance with the code, and the limit state functions established.

Computer programs for calculating the reliability indices of the various limit states are developed and run using FORM5 (1988)

3.2 IDEALISATION OF FUNCTIONS OF LOADS AND LOAD EFFECTS

The continuous beam considered in this study is of the type in Fig. 3.1 with two equal spans carrying uniformly distributed load. It is assumed that the external supports are not restrained in rotation.

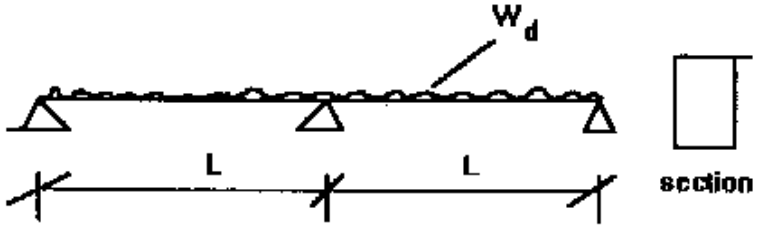


Fig 3.1 Uniformly distributed loaded continuous Beam

To this end, variation in imposed and dead load combination was done in accordance with the code requirements.

The design load W_d is given as

$$Wd = 1.4g_k + 1.6q_k \quad (3.1)$$

where g_k = characteristic dead load per unit length

q_k = characteristic live load per unit length

The support moment given by equation (3.2)

$$M_B = \frac{W_d l^2}{8} = (0.175gk + 0.2qk)l^2 \quad (3.2)$$

Where l = the effective span

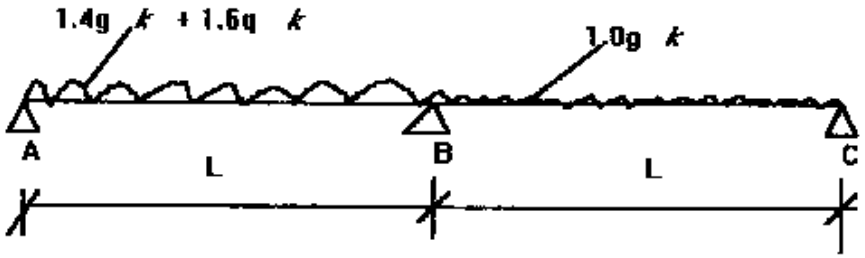


Fig 3.2 Loading to yield maximum moment at the span

From the idealisation of Fig. 3.2, maximum span moment is evaluated to be:

$$M_{max} = (0.l08g_k + 0.152q_k)l^2 \quad (3.4)$$

The shear forces are gives as

$$V_{AB} = -V_{CB}$$

$$= (0.525g_k + 0.225q_k)1 \quad (3.5)$$

$$V_{BA} = -V_{BC}$$

$= (0.875g_k + 1.0q_k)1$ (3.6)

Fig. 2.1 of BS 8110 (1985) gives the ultimate design stress as:

$$\frac{0.67f_{cu}}{\gamma_{mc}} = \frac{.067f_{cu}}{1.5} \approx 0.45f_{cu} \quad (3.7)$$

in which the coefficient 0.67 accounts for the difference between the bending strength and the cube test..

γ_{mc}=l.5 is the partial factor of safety for concrete strength as in Table *2.2* of section 2.4.

f_{cu} is the characteristic strength of concrete. The yield stress of steel is given as $\frac{f_y}{\gamma_{ms}}$

where f_y is the characteristic strength of steel and γ_{ms}=1.15 is the partial factor of safety for stee1 strength (Table 2.2 section 2.4)

In the derivation of moment of resistance for various sections and conditions, simplified equivalent rectangular stress block is made use of, bearing in mind the need for maintaining equilibrium at section under consideration.

3.3 LIMIT STATE EQUATION FOR FLEXURAL CRITERIA

Failure function is given as

$$g(x) = M_U - M_{app} = 0 \quad (3.9)$$

where M_u is the moment of resistance of the section and M_{app} is the applied moment.

3.4 SINGLY REINFORCED SECTIONS

3.4.1 Interior support

For a rectangular section at the interior support, the moment of resistance M_u of the section for concrete in compression is given as (BS 8110, 1985)

$M_u = 0.156 f_{cu} bd^2$ (3.10)

and for the tensile section, it is given as

$M_u = O.87 f_y Z A_s.$ (3.11)

From eqns. (3.10) and (3.11), moment of resistance M_u is a function of breadth b, effective depth d of the concrete section and the characteristic strength of concrete f_{cu} (in compression) lever arm z, area of steel A_s, and characteristic strength of steel f_y (in tension).

The limit state function for the compression section is given as

$g(x) = 0.1\ 56\ f_{cu} bd^2 — (0.175 g_k + 0.2 q_k) 1^2$ (3.12)

From equations (3.10) and (3.2)

$= 0.1\ 56\ f_{cu} bd^2 — (0.lO8a + 0.2) q_k l^2$ (3.13)

where α is load ratio given as $\alpha = \frac{g_k}{q_k}$ and the limit state function for the tensile section is

$g(x) = 0.87 f_y Z A_s - (0.175\alpha + 0.2) q_k l^2$ (3.14)

where

$Z = d\left[0.5 + \sqrt{0.25 - K/0.97}\right]$ n.e. 0.95d (3.15)

and

K = 0.156

The mid span is considered for rectangular and flanged sections. For a rectangular section, the limit state functions are derived to be

$g(x) = 0.156 f_{cu} bd^2 - (0.108\alpha + 0.152) q_k l^2$ (3.16)

for compression section, and

g(x) 0.87fyZAs— (0.108α+ 0.152) (3.17)

for tensile section. The lever arm Z is the same as in equal. (3.15)

For a flanged beam with stress block outside the flange and neutral axis depth (x)=0.5d, the limit state function g(x) is derived to be

$$g(x) = 0.87f_yA_s(d — 0.5hf)-0.1f_{cu}b_wd(0.45- h_f)- (1.08\alpha \pm 0.152)q_kl^2 \quad (3.\ 1\ 8)$$

3.4.2 Doubly reinforced section

The moment of resistance of the compression section is the sum of that due to concrete section in compression and that due compression reinforcement.

$$M_u= 0.156f_{cu}bd^2 + 0.87\ f_yA_s'(d-d') \quad (3.19)$$

Thus,

$$g(x) = 0.156f_{cu}bd^2 \pm 0.\ 87f_yA_s'(d — d') — qkl^2\ (0.\ 175\alpha + 0.2) \quad (3.20)$$

The moment of resistance due to reinforcement is derived by taking moment of forces about the line of action of concrete compressive force in a doubly reinforced section

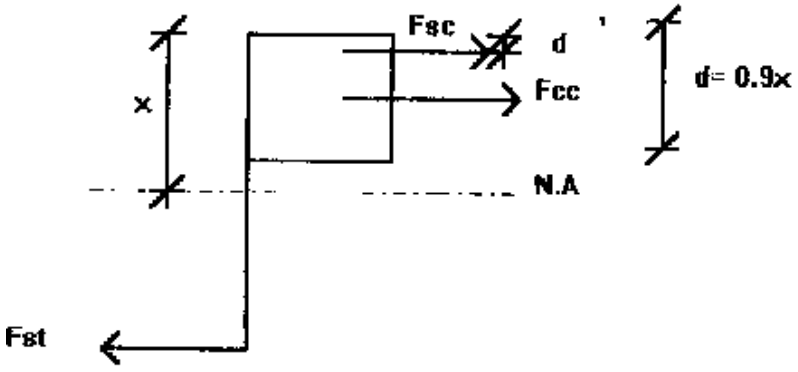


Fig 3.4 Equivalent rectangular stress block for a doubly reinforced section

Taking moment about concrete compressive force F_{cc} the moment of resistance is given as

$M_u = F_{cc}Z — F_{cc}(0.45d-d')$ (3.21)

The maximum value of the lever arm Z allowed to ensure a tensile failure with a ductile section is given as (BS.8110,1985)

$Z = 0.775d$ (3.22)

Hence,

$M_u = 0.87f_yA_sx\ 0.775d+0.87f_yA'_s(0.45d — d')$ (3. 23)

and the failure function is given as

$$g(x) = 0.67f_y\rho bd^2 + 0.87f_y\rho' bd^2\left(0.45 - \frac{d'}{d}\right) - q_k l^2(0.175\alpha + 0.2) \quad (3.24)$$

where $\rho = A_s/(bd)$

$\rho' = A'_s/(bd)$

At the mid span considering a flanged section with compression stress block extending beyond the flange depth and neutral axis depth of 0.5d for a conservative design is herein considered. Area of compression steel is given as

and the area of tensile steel is derived to be

$$A_s = \frac{0.2f_{cu}b_wd + f_{cu}h_f(b_f - b_w) + A'_s}{0.87f_y} \quad (3.26)$$

The limit function for tensile failure is derived to be

$$g(x) = 0.87f_yA_s\rho bd(d-d') - 0.044b_wd^2\left(1 + 4.5\frac{d'}{d}\right) + 0.45f_{cu}(b_f - b_w)(0.5h_f - d') - ql^2(0.108\alpha + 0.152) \quad (3.27)$$

Similarly for compression failure, limit state function becomes

$$g(x) = 0.156f_{cu}bd^2 + 0.45f_{cu}(b_f - b_w)(d - 0.5h_f) + 0.87f_y\rho' bd(d-d') - ql^2(0.108\alpha + 0.152) \quad (3.28)$$

3.5 LIMIT STATE EQUATIONS FOR SHEAR CRITERIA

The B.S.8110 (1985) specifies two design conditions for shear capacity. In the first case, nominal reinforcement are recommended in beams with shear stress (v) at a cross section satisfying the following condition

$0.5v(v(v_c + 0.4))$

where

$v = \frac{V}{b_w d}$

V = design shear force due to ultimate load

b_w = *breadthof section*

d = effective depth of beam

v_c = *design concrete shear stress of Table 3.9 of BS 8110 (1985)*

The limit state equation for this condition is derived as

$$g(x) = 0.87f_{yv}A_{sv} — 0.4b_vS_v \qquad (3.29)$$

It should be noted that the code prescribed a constant design shear stress of O.4N/mm^2 appearing as a coefficient in the equation. The Implication is that the limit state function is independent of load.

In the second case the shear stress V satisfies the condition

$(v_c + 0.4) < v < 0.8\sqrt{f_{cu}}$ 50N/mm^2

Shear links are provided using

$$A_{sv} \geq \frac{b_v S_v (v - v_c)}{0.0.87 f_{yv}} \qquad (3.30)$$

and the state function becomes

$$g(x) = 0.87 f_{yv} A_{sv} + \frac{b_v S_v V}{b_v d} \tag{3.31}$$

The shear ***force, V=q_kl(0.875a + 1.0)*** and substituting in eq.(3.31) become

$$g(x) = 0.87 f_{yv} A_{sv} + b_v S_v v_c - Sv \frac{q_k l(0.875\alpha + 1.0)}{d} \tag{3.32}$$

Design concrete shear stress V_c of Table 3.9 of BS 8110 (1985) is derived from the expression

$$vc = \frac{0.79}{\gamma_m} \sqrt[3]{\frac{100 A_s}{b_v d}} \sqrt[4]{\frac{400}{d}} \tag{3.33}$$

3.6 PARAMETERS IDENTIFICATION FOR LIMIT STATE FUNCTION

The basic variables for the various limit state functions and their statistics are identified as indicated in Tables 3.1 and 3.2. The Parameters are employed by FORMS in the determination of reliability indices.

Table 3.1: Parameters for Flexural Requirement

Parameter no.	Basic Variable	Distribution type	Mean (EX)	Standard deviation (SX)	Co-efficient of Variation COV
X1	Fcu	NORMAL	23.8N/mm^2	3.75N/m^2	0.15
X2	Qk	GUMBEL	5.0N/mm	1.85N/mm	0.037
X3	B	LOGNORMAL	300.0mm	13.5mm	0.045
X4	D	LOGNOMAL	400.0mm	34.4mm	0.086
X5	Fy	LOGNOMAL	250N/mm^2	3.5N/mm^2	0.014
X6	P	NORMAL	Varying	Varying	0.036
X7	P'	NORMAL	Varying	Varying	0.036
X8	D'	NORMAL	35.0mm	3.01mm	0.086
X9	Hf	LOGNOMAL	200.0mm	17.2mm	0.08

Table 3.2: Parameters for shear Requirement

Parameter no.	Basic Variable	Distribution Type	Mean (EX)	Standard deviation (SX)	Coefficient of Variation COV
X1	F_{yv}	LOGNORMAL	250N/mm^2	3.5N/mm^2	0.14
X2	ϕ	NORMAL	Varying	Varying	0.036
X3	B_v	LOGNORMAL	300.0mm	13.5mm	0.045
X4	D	LOGNORMAL	400.0mm	34.4mm	0.086
X5	Qk	GUMBEL	2.5N/mm	0.925N/mm	0.37
X6	Fcu	NORMAL	23.8N/mm^2	3.75N/mm^2	0.15

3.7 EVALUATION OF LIMIT STATE FUNCTIONS

Fig 4.1, 4.2 and 4.3 show the graphs of the tension reliability of singly reinforced section at support. Generally the curves follow similar trends, that is reliability decreases with increasing load ratio for a given reinforcement ratio (P=0.005), has a reliability indices varying from 3.5 to 1.4 for load ratios 0.5 to 3.0. Comparing this with values in Table 2.3 (NKB,1978), this curve will give an equivalent load ratio of about 0.75 considering minimum recommended index of 3.1 for less serious safety consequences. This means that the curve cannot be used for indices greater than this value. For p- values of 0.01 and above, the requirements of Table 2.3 are satisfied, with P=0.01, having an index varying from 5.0 to 2.9 and P=0.025 index varying from 6.8 to 4.65, for the range of load ratios considered.

It is also observed that reliability indices decrease with increase in the span of the beam.

Computer programs in FORTRAN 77 are written for use by FORM5 in evaluation of reliability indices.

CHAPTER FOUR

DISCUSSION OF RESULTS

4.0 EVALUATION OF LIMIT STATE FUNCTIONS

Computer programs in FORTRAN 77 are written for use by FORM5 in evaluation of reliability indices. Using the Data generated from the reliability evaluation of the various limit state conditions (Appendix I), graphs are plotted to allow for proper analysis of the reliability levels in the BS 8110 (1985), requirements for reinforced concrete continuous beam. The reliability indices observed for the various failure conditions using varying parameters are plotted against varying load Ratio. It is generally observed that reliability decreases with increasing load ratio.

4.1 SINGLY REINFORCED SECTION IN FLEXURE

4.1.1 Tensile Requirement at Support

Figures 4.1, 4.2 and 4.3 show the graphs of the tension reliability of singly reinforced section at support. Generally the curves follow similar trends, that is, reliability decreases with increasing load ratio for a given reinforcement ratio (rho=0.005), has a reliability indices varying from 3.5 to 1.4 for load ratios 0.5 to 3.0. Comparing this with values in Table 2.3 (NKB, 1978), this curve will give an equivalent load ratio of about 0.75 considering minimum recommended index of 3.1 for less serious safety consequences. This means that the curve cannot be used for indices greater than this value. For rho values of 0.01 and above, the requirements of Table 2.3 are satisfied, with rho =0.01, having an index varying from 5.0 to 2.9 and rho =0.025 index varying from 6.8 to 4.65, for the range of load ratios considered.

It is also observed that reliability indices decrease with increase in the span of the beam.

Generally, the curves follow similar trends for varying beam length. This is also the case with varying reinforcement ratios as can be inferred from the graphs. For a given reinforcement ratio, reliability index decreases with increase in load ratio. Thus, the general trend as noticed in reliability graphs in Figs. 4.1, 4.2, 4.3 and others as shall be seen later are indicating the fact that reliability arising from design to the code is sensitive to design variables such as load and the geometry as observed by the varying beta level in the graphs; meaning that uniform reliability level which is an object of code (BS 8110, 1985) specification can not be guaranteed.

From Fig. 4.1, the least reinforcement ratio (0.005) has reliability indices varying from 3.5 to 1.4 for load ratio from 0.5 to 3.0. Judging from NKB (1978) classification in Table 2.3 of chapter Two, this curve will be adequate for just load ratio up to about 0.75 considering the minimum recommended index of 3.1 for less serious safety consequences meaning that the curve will be generally inadequate for higher recommended indices. The 0.01 and above reinforcement curves satisfy the requirements of Table 2.3. The 0.01 reinforcement ratio has maximum index of 5.0 and a minimum of *2.9*. The maximum reinforcement ratio considered in this instance is 0.025 having indices varying between 6.8 and 4.65 for the range of load ratio considered.

It is also observed that reliability indices , decrease with increase in beam length. This feature can be explained from the fact that increase in length increases the external load effect component of the limit state function.

Furthermore, it can be observed in Fig. 4.2 that the curve for 0.025 reinforcement ratio

stopped at load ratio = 1.5 because at that point, it was observed that K exceeded K', a condition that necessitates compression reinforcement going by the code under consideration, where $K = M_{app}/f_{cu}bd^2$ and K'=0.156. This condition is reached as a result of reduction in effective depth of beam due to increased reinforcement in the tensile section of the beam, meaning that tensile steel reinforcement cannot be indefinitely increased in an attempt to ensure reliability without consideration for compression section. Similar trend is observed for the 6m beam as indicated in Fig.4.3 with reduced reliability indices for corresponding reinforcement and load ratios.

The curve of 0.005 reinforcement ratio has lowest reliability indices with probability of failure of 0.0048 (beta = 1.66) corresponding to load ratio of 0.5 and failure probability of 0.5 (beta = 0) occurring at 2.25 load ratio. Similarly in Fig 4.3 the 0.025 reinforcement ratio curve has the least probability of failure of $\mathbf{4.7918x10^{6}}$.

Table 4.1: Load and Reinforcement ratios for specified reliability index of a singly reinforced section at support.

Beta =3.1 ($P_f = 9.68x10^{-4}$)	Rho	0.01	0.015	-
	Alpha	1.3	2.4	-
Beta =3.7 ($P_f = 1.044x10^{-4}$)	Rho	0.01	0.015	0.02
	Alpha	0.4	1.5	2.2
Beta =4.2 ($P_f = 1.32x10^{-5}$)	Rho	0.015	0.02	-
	Alpha	1.0	1.6	-
Beta =4.8 ($P_f = 7.93x10^{-6}$)	Rho	0.015	0.05	0.025
	Alpha	0.6	1.1	1.5
Beta =5.2 ($P_f = 9.69x10^{-7}$)	Rho	0.02	0.025	-
	Alpha	0.7	1.0	-

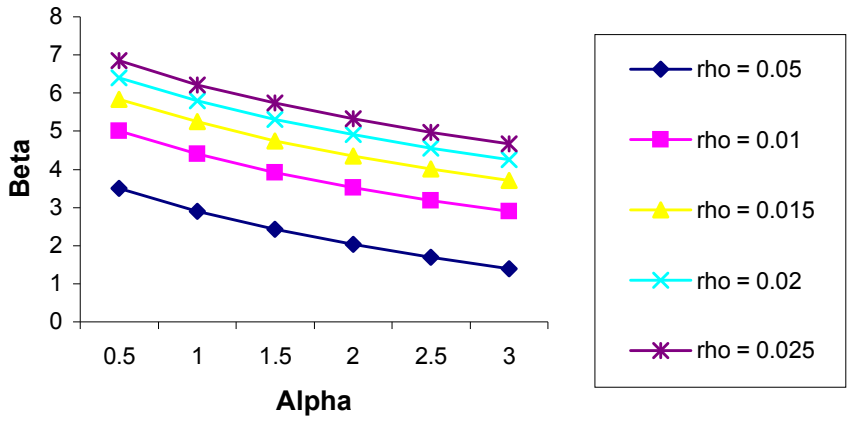


Fig. 4.1: Tension Reliability of a Singly Reinforced Rectangular Section at interior support

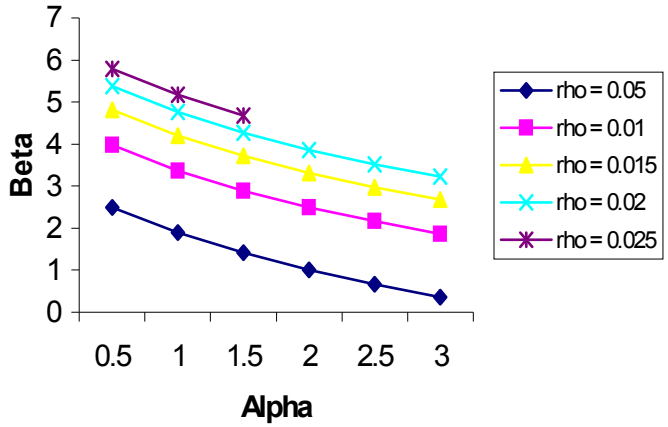


Fig. 4.2: Tension Reliability of a Singly Reinforced Section at interior support of 5m spanned beam

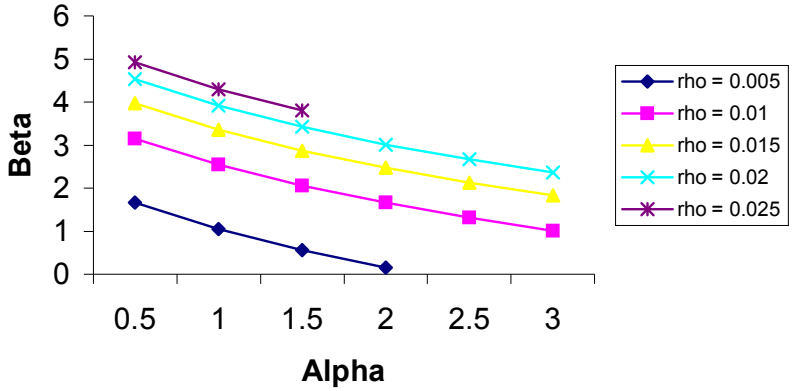


Fig. 4.3: Tensile reliability of singly reinforced section at interior for 6m spanned beam

4.1.2 Compression Requirement at interior support

Fig.4.4 shows the graph of the compression reliability of the concrete section at support. From Fig. 4.4, the compression reliability indices range between 2.77 and 5.47 for load ratios 3.0 to 0.5 respectively for the 4m spanned beam. Similar results are obtained for 5m and 6m beams.

If the graphs of compressive indices are superimposed on that of the tensile indices, it is observed that the reliability of the tensile section exceeds that of the compression section for reinforcement ratios 0.01 and above (Figs 4.5 and 4.6). For the same reliability to apply, the reinforcement ratio will be 0.0096 got by interpolation from Fig. 4.5. Using an index of 4.2 (Failure type II, Table 2.3) on figure 4.5, means that load ratio not exceeding 1.10 must be considered for compression reinforcement in the 4m span beam lengths. This shows that design to BS 8110(1985), may not ensure the same reliability for compression and tensile requirement. To ensure adequate performance of beam section, the reliability of both the tensile and compression section must be considered.

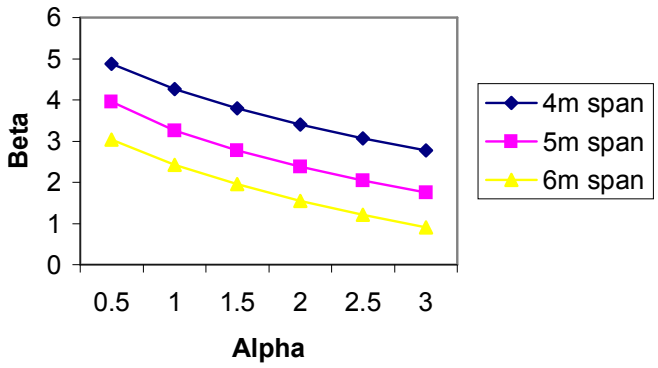


Fig. 4.4: Reliability of compression section at support

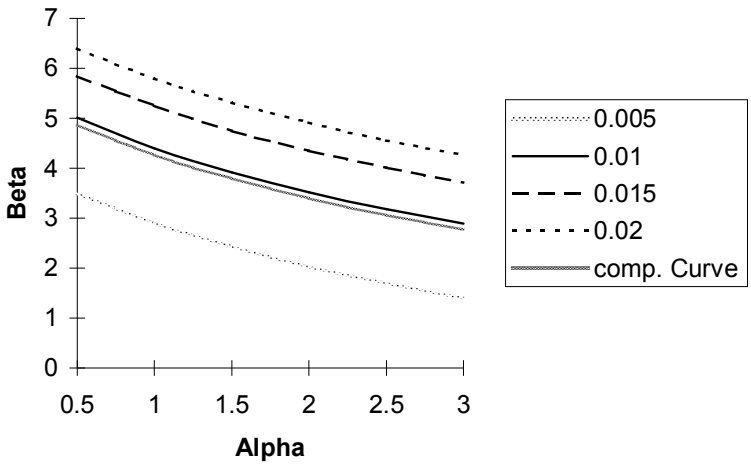


Fig. 4.5: Superimposed Tensile and compressive reliability of 4m spanned beam at interior support

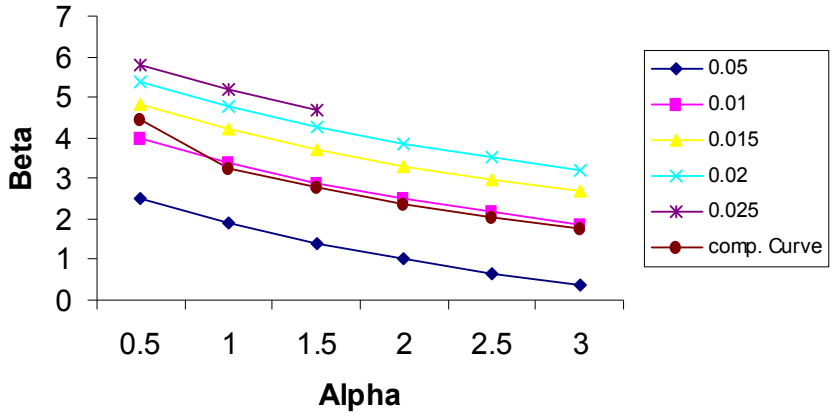


Fig. 4.6: Superimposed Tensile and compressive reliability of 5m spanned beam at interior support

4.1.3 Tensile Requirement at Mid Span

Figures 4.7, 4.8 and 4.9, show the graph of reliability of a slightly reinforced T-section with varied reinforcement ratio (p) for span 4m, 5m and 6m respectively. The graphs show reliability indices decreasing with increasing load ratio for a given reinforcement ratio. Comparing these graphs (Figs.4.7, 4.8 and 4.9) with Figs. 4.1, 4.2 and 4.3, for the interior support tensile reliability indices, it is very evident that the mid span has higher indices for the same values of reinforcement ratios. This is due to the bending moment at the mid span being less than that on the support.

Reliability indices resulting from variation of effective depth to breadth of web (d/b_W) are indicated by graphs of Figs. 4.10 to 4.12. The plots show the sensitivity of reliability levels to d/b_w.

However, the depth of flange to effective depth ratio h_f/d is not sensitive to reliability as shown in Figs. 4.13 and 4.14.

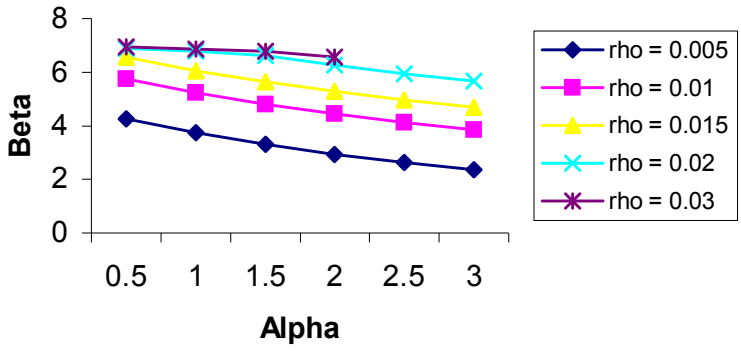


Fig. 4.7: Reliability of a singly reinforced T-section with varied reinforcement ratio (Rho) for 4m spanned beam

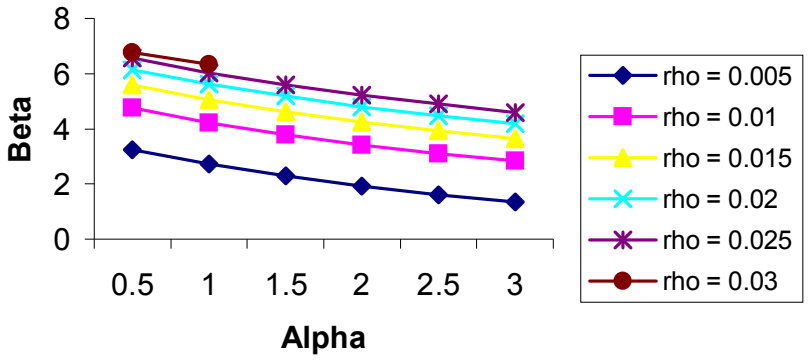


Fig. 4.8: Reliability of a singly reinforced T-section with varied reinforcement ratio (rho) for 5m spanned beam

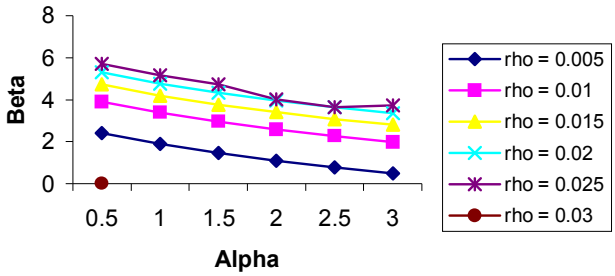


Fig. 4.9: Reliability of a singly reinforced T-section with varied reinforcement ratio (rho) for 6 spanned beam.

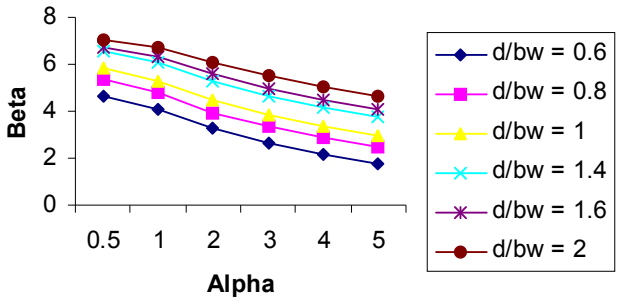


Fig. 4.10: Reliability of a singly reinforced T-section at mid span with varied d/b_W for 4m spanned beam

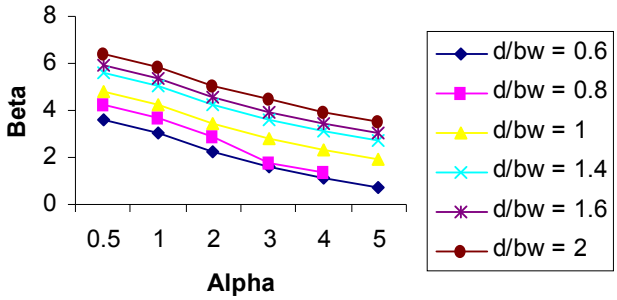


Fig. 4.11: Reliability of a singly reinforced T-section at mid span with varied d/b_W for 5m spanned beam

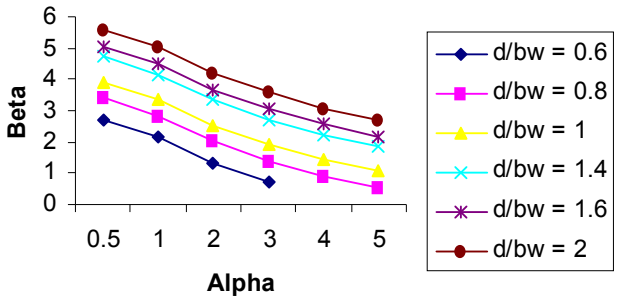


Fig. 4.12: Reliability of a singly reinforced T-section at mid span with varied d/b_W for 6m spanned beam

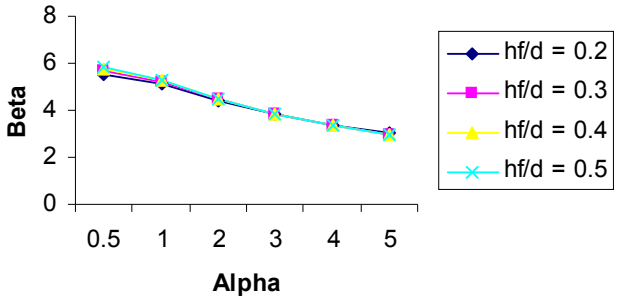


Fig. 4.13: Reliability of a singly reinforced T-section at mid span with varied h_F/d for 4m spanned beam

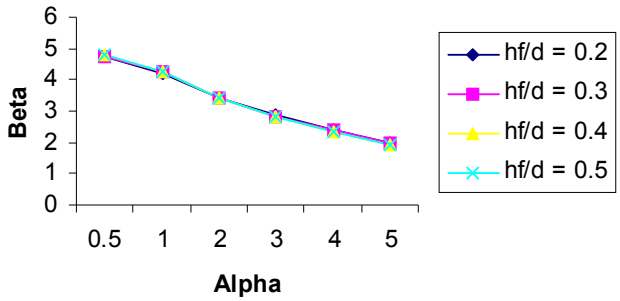


Fig. 4.14: Reliability of a singly reinforced T-section at mid span with varied h_F/d for 5m beam

Table 4.2: Reliability indices of Mid span and support of 4m Beam

Alpha	RHO = 0.005		RHO = 0.01		RHO = 0.015	
	Mid span	Support	Mid span	Support	Mid span	Support
0.5	4.25	3.5	5.76	5.01	6.28	5.84
1.0	3.73	2.9	5.23	4.4	5.76	5.23
1.5	3.3	2.43	4.8	3.92	5.23	4.75
2.0	2.94	2.03	4.44	3.52	4.80	4.35
2.5	2.63	1.7	4.13	3.18	4.44	4.01
3.0	2.36	1.4	3.85	2.89	4.13	3.71

4.1.4 Compression Requirement for Mid Span

Figure 4.15 shows the reliability of singly reinforced concrete flanged sections in compression at mid span. This shows reliability indices decreasing with increasing load ratio for a given reinforcement ratio. Unlike the case of tensile requirement at mid span (Fig. 4.7 to 4.9) the reliability of the mid span is less than the values obtained for the interior support (Fig.4.1 to 4.3). This may be due to the flanged action with reduction in concrete cross-section in compression.

4.2 DOUBLY REINFORCED SECTION IN FLEXURE

Whenever applied moment exceeds the moment of resistance of the concrete section, the section would have to be provided with compression reinforcement. The reliability graphs of compression sections raise some questions on the accuracy of the conventional method of checking the need to provide compression reinforcement. And provide better insight to reliability evaluation of doubly reinforced concrete. This section considers the mid span and interior support for compression reinforcement.

4.2.1 Compression Requirement for Interior support

Figs. 4.16 to 4.18 are plotted for the interior support requiring compression reinforcement for a particular tensile reinforcement ratio (rho = 0.03). The need for compression reinforcement is satisfied at points beyond a superimposed compression curves of Fig. 4.5 and 4.6. Taking Figure 4.5 for instance, a required reliability index of 4.2 for a load ratio of 2.0 will coincide with a point above the compression reliability curve on the superimposed plot and thus will require comprehension reinforcement.

Figs 4.24 to 4.29 are plotted for varying ratio of effective depth to breadth at interior support keeping other design parameters of Tables 3.1 and 3.2 constant in limit state equation 3.20. The graphs show that the higher the effective depth to breadth ratio, the higher the reliability. They also indicate that the higher the load ratio, the less the reliability index for a particular effective depth to breadth ratio. The significance of these plots will be seen in the choice of design parameters when they are used to as design charts.

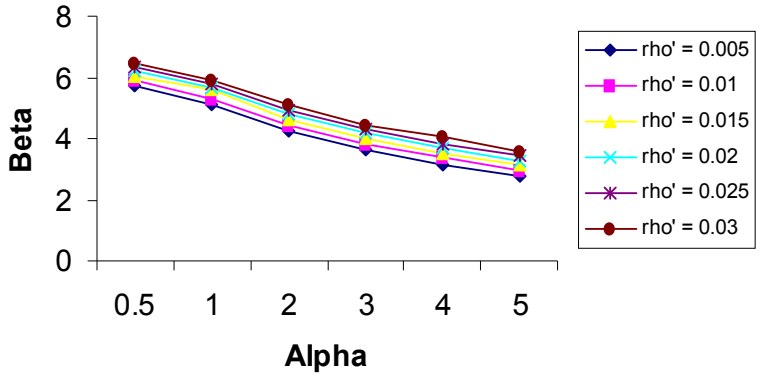


Fig. 4.15: Reliability requirement for compression in a doubly reinforced section at support of 4m spanned beam.

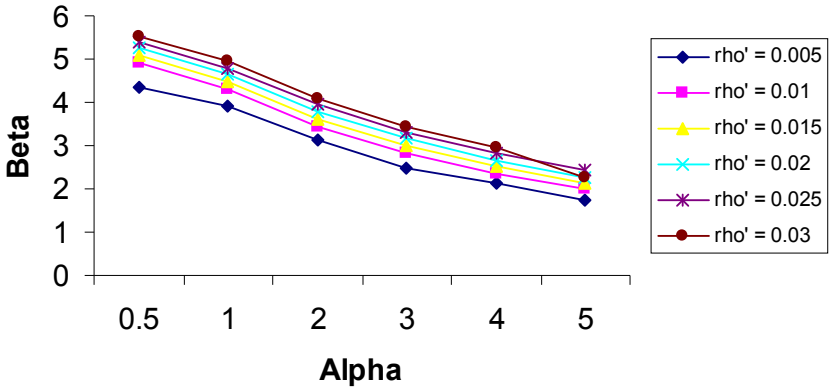


Fig. 4.16: Reliability requirement for compression in a doubly reinforced section support of 5m spanned beam

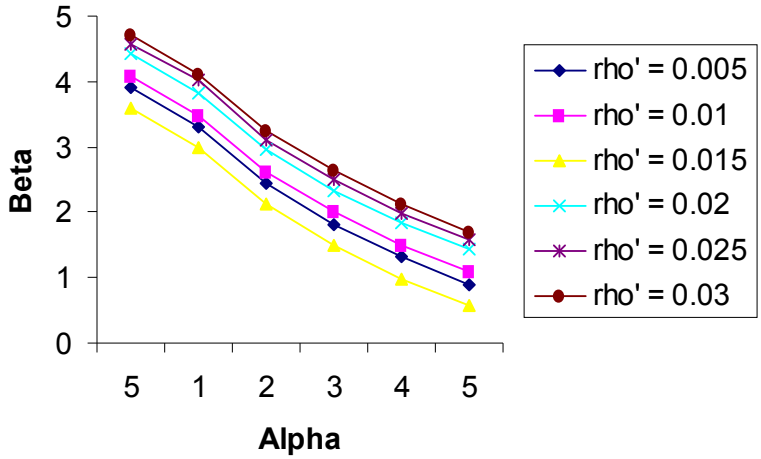


Fig. 4.17: Reliability requirement for compression in a doubly reinforced section at support of 6m spanned beam

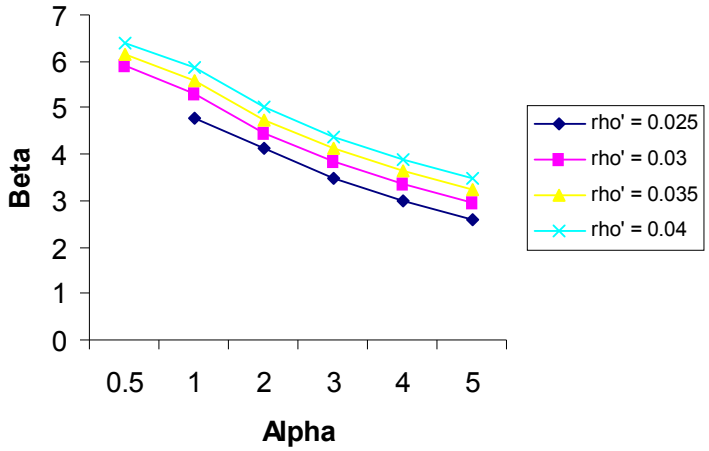


Fig. 4.18: Reliability of Tensile section in a doubly reinforced beam (4m span) at support for varying rho at rho' = 0.01

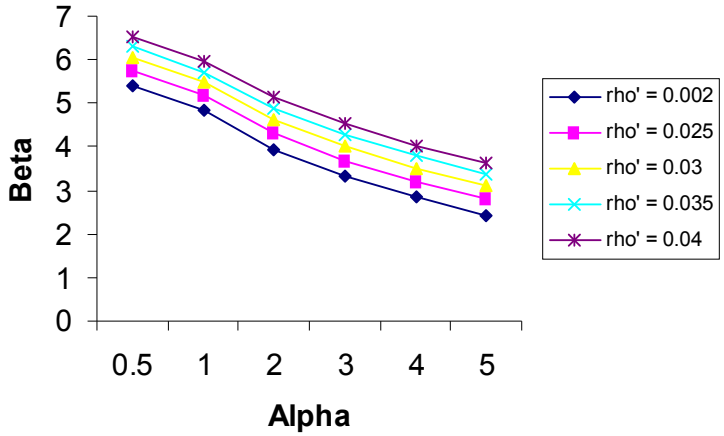


Fig. 4.19: Reliability of Tensile section in a doubly reinforced beam (4m span) at support for varying rho at rho' = 0.05

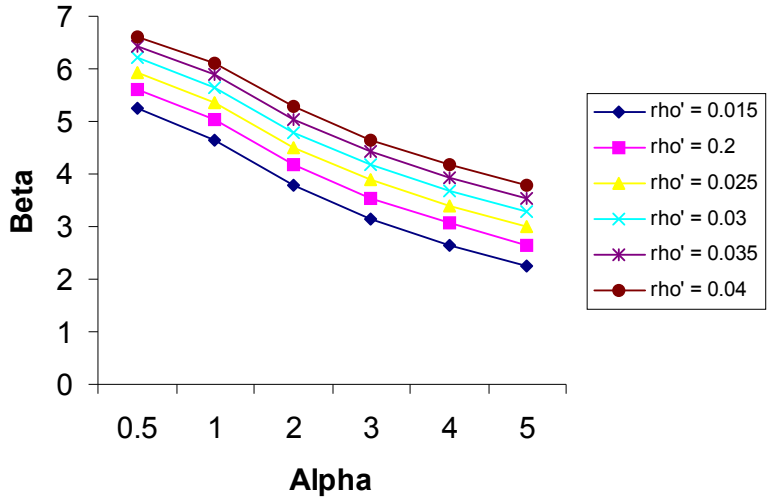


Fig. 4.20: Reliability of Tensile section in a doubly reinforced beam (4m span) at support for varying rho at rho' = 0.020

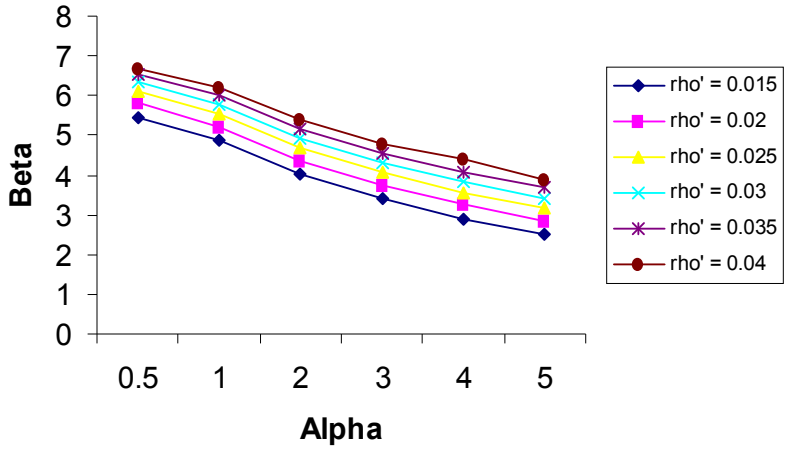


Fig. 4.21: Reliability of Tensile section in a doubly reinforced beam (4m span) at support for varying rho at rho' = 0.025

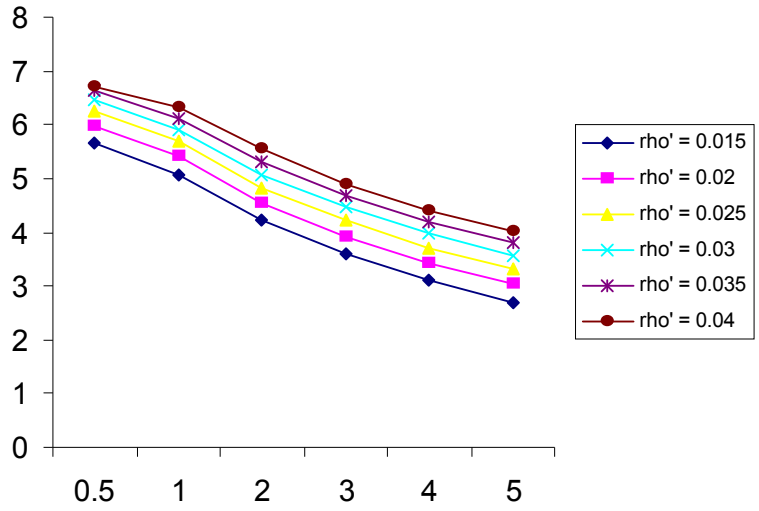


Fig. 4.22: Reliability of Tensile section in a doubly reinforced 4m spanned beam at support for varying rho at rho' = 0.03

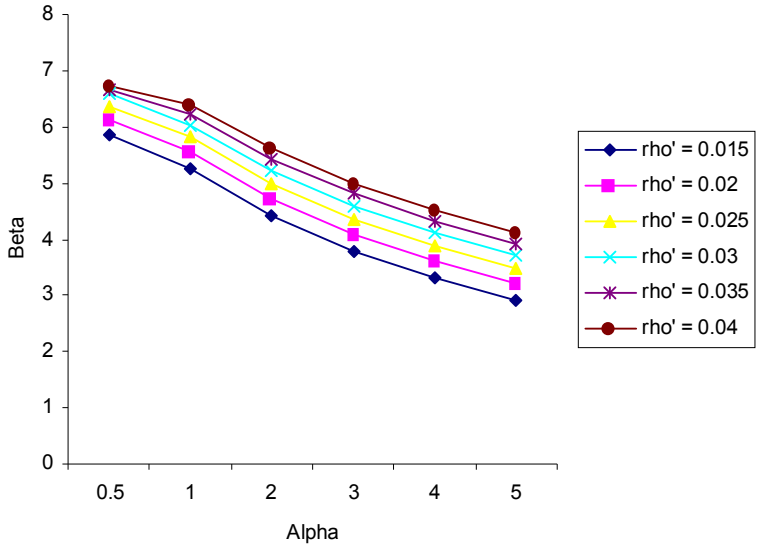


Fig. 4.23: Reliability of Tensile section in a doubly reinforced 4m spanned beam at support for varying rho at rho' = 0.035

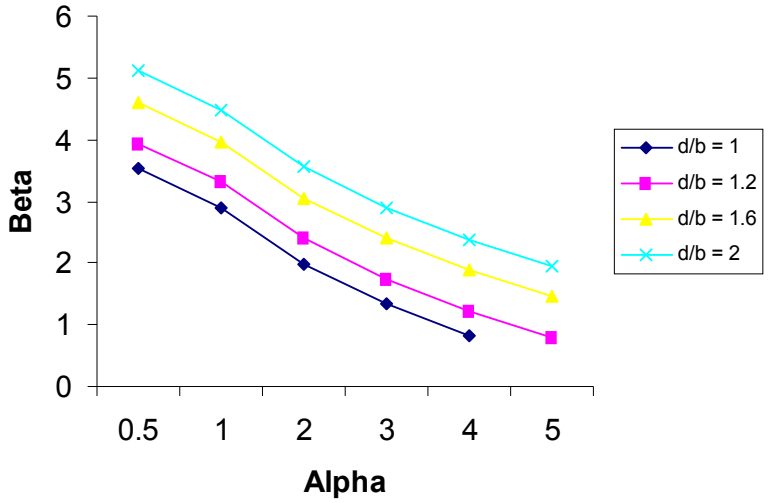


Fig.4.24: Reliability of a singly reinforced section with varied d/b at interior support of 4m spanned beam for rho = 0.01

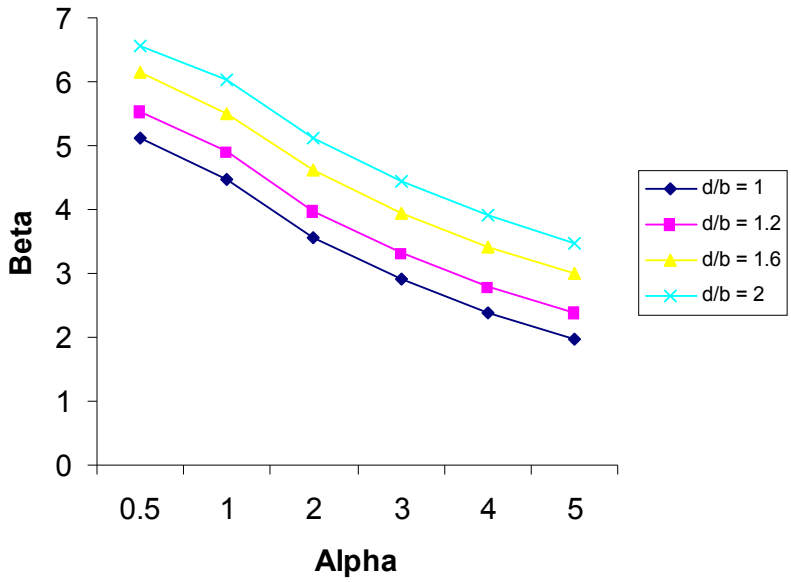


Fig.4.25: Reliability of a singly reinforced section with varied d/b at interior support of 4m spanned beam for rho = 0.015

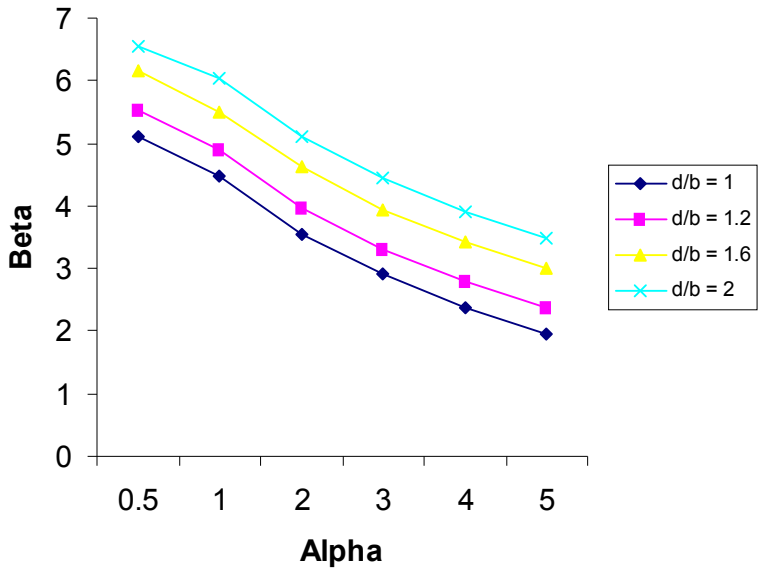


Fig.4.26: Reliability of a singly reinforced section with varied d/b at interior support of 4m spanned beam for rho = 0.02

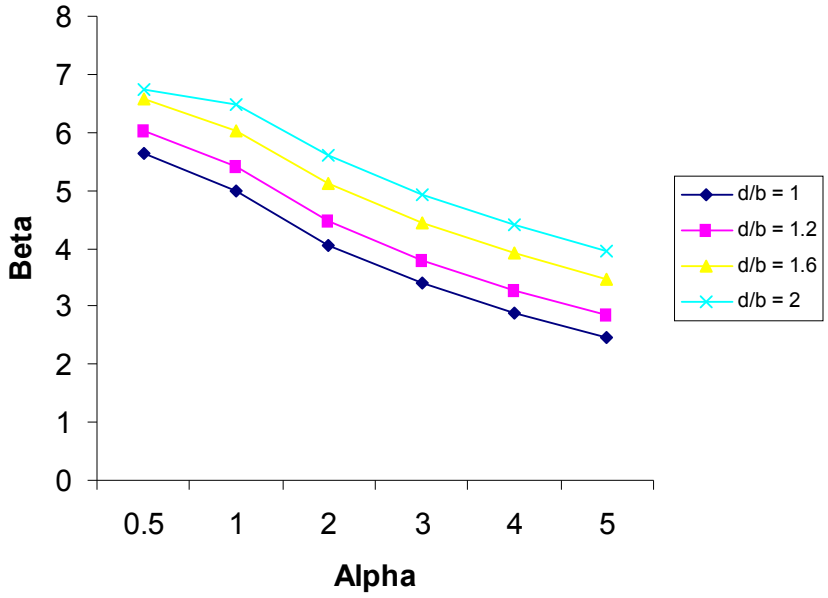


Fig. 4.27: Reliability of a singly reinforced section with varied d/b at interior support of 4m spanned beam for rho = 0.025

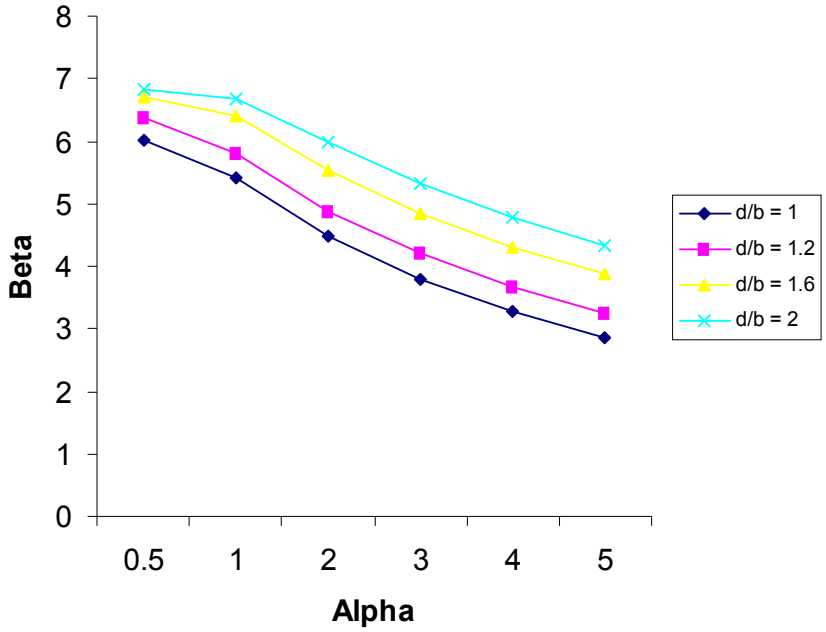


Fig. 4.28: Reliability of a singly reinforced section with varied d/b at interior support of 4m spanned beam for rho = 0.03

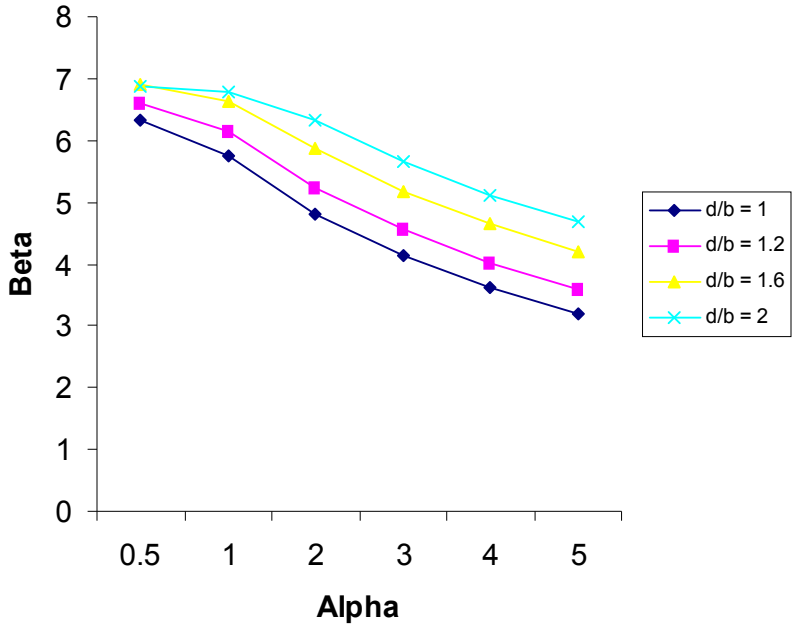


Fig.4.29: Reliability of a singly reinforced section with varied d/b at interior support of 4m spanned beam for rho = 0.035

4.2.2 Compression Requirement for Mid Span

The reliability graphs of the compression section using compression reinforcement are as shown in Figs. 4.30 to 4.32.

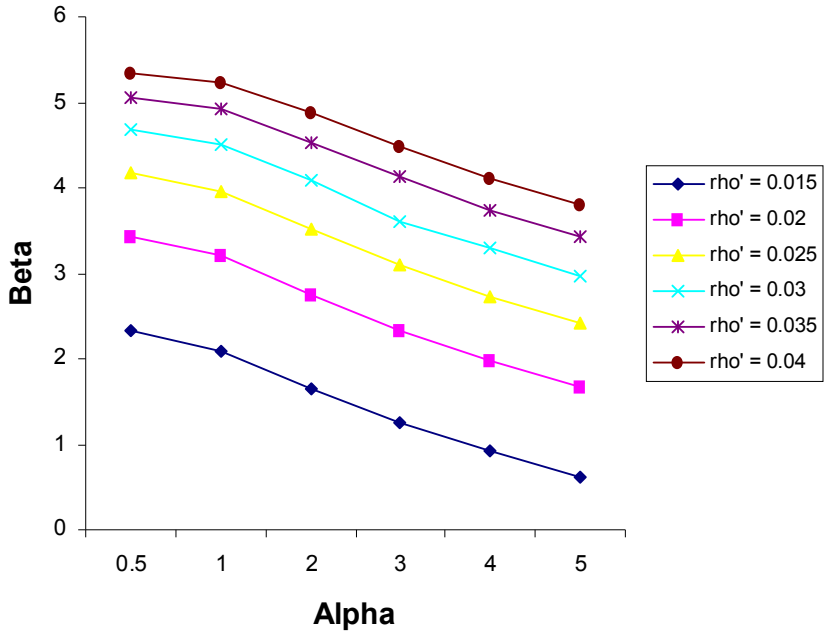


Fig.4.30: Reliability of a doubly reinforced Flanged section of 4m beam

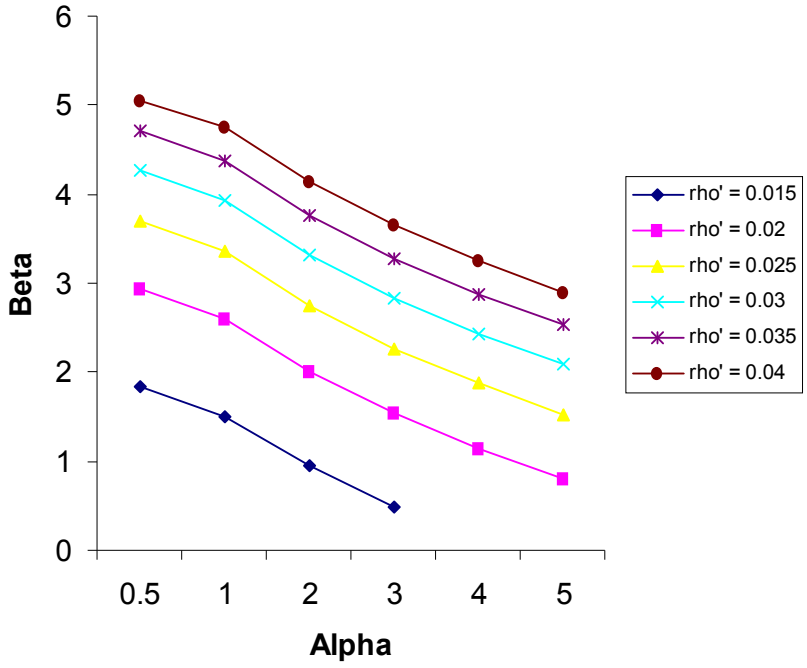


Fig.4.31: Reliability of a doubly reinforced Flanged section of 5m beam

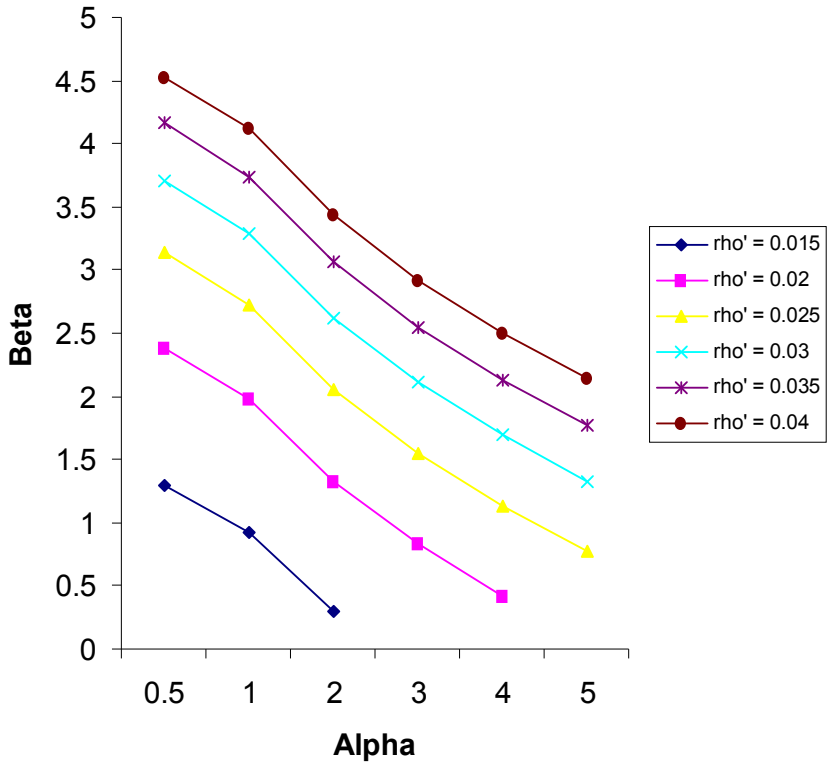


Fig.4.32: Reliability of a doubly reinforced Flanged section of 6m beam

4.3 SHEAR CRITERIA

It is observed from Fig. 4.33 that irrespective of the load ratio and beam length, the reliability index is constant for a give A_{SV}/S_V. This shows that both the load ratio and beam length are not having any influence on the limit state equation. Recall that BS. 8110 (1985) suggests a design shear stress of $0.4N/mm^2$. However, reliability level is dependent on the ratio of area of shear reinforcement to spacing as in Fig. 4.33.

In the second shear condition where design shear stress is in the range:
$V_C + 0.4 < V < (0.8F_{cu}$ or $5.0N/mm^2)$ reliability calculation yields indices shown in Figs. 4.34 and 4.36. It is observed that for a given A_{SV}/S_V, the reliability decreases with load ratio as in other reliability curves. Reliability also increases with reinforcement spacing ratio. This is obvious because variations in these parameters influence resistance as well as action on the beam. In this instance for $A_{SV}/S_V = 0.335$, the reliability indices range from about 1.0 to 4.7 corresponding respectively to a load ratio of 5.0 and 0.5.

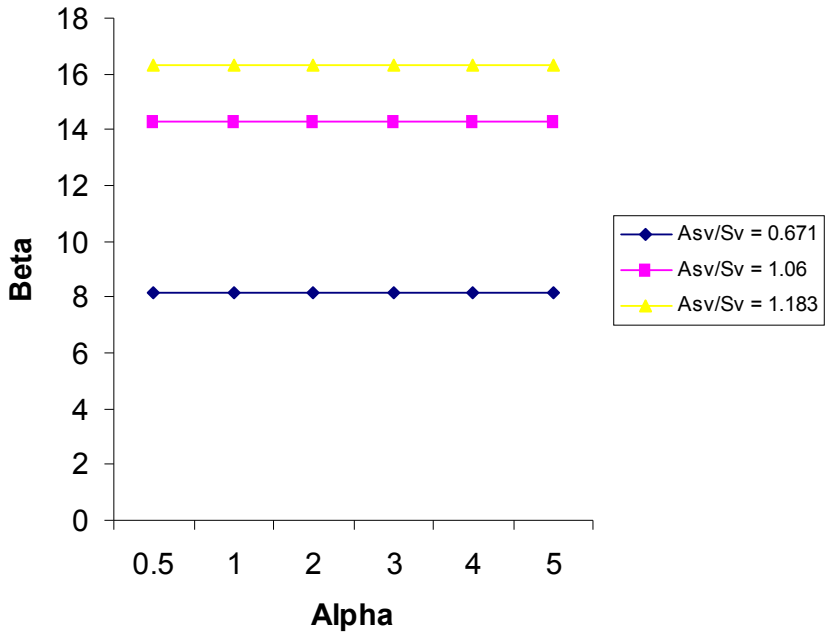


Fig. 4.33: Nominal shear reliability for varied Asv/Sv for 4m,5m and 6m beam spans

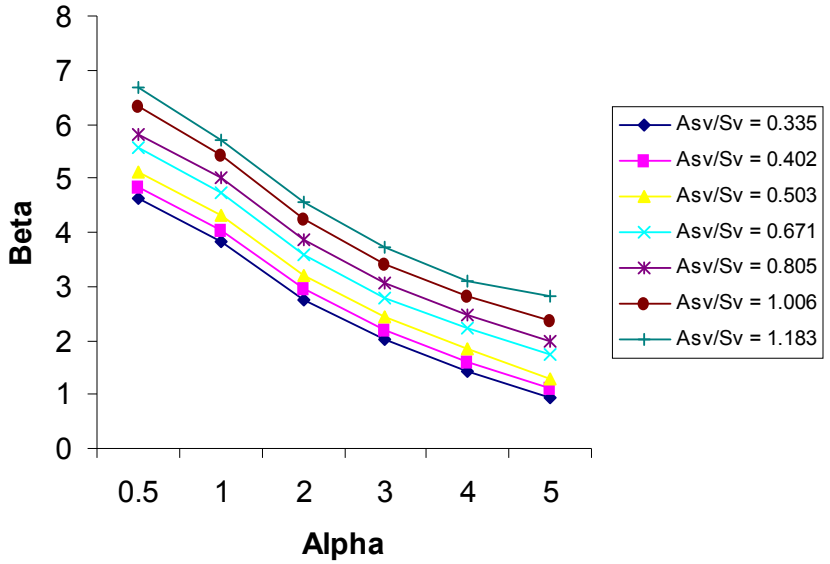


Fig. 4.34: Design shear reliability for varied Asv/Sv for 4m beam

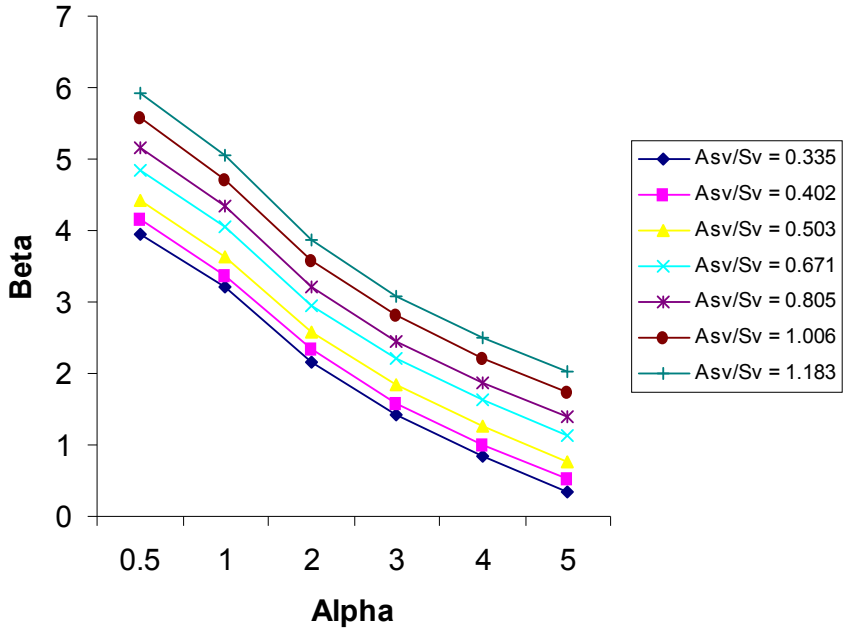


Fig.4.35: Design shear reliability for varied Asv/Sv for 5m beam

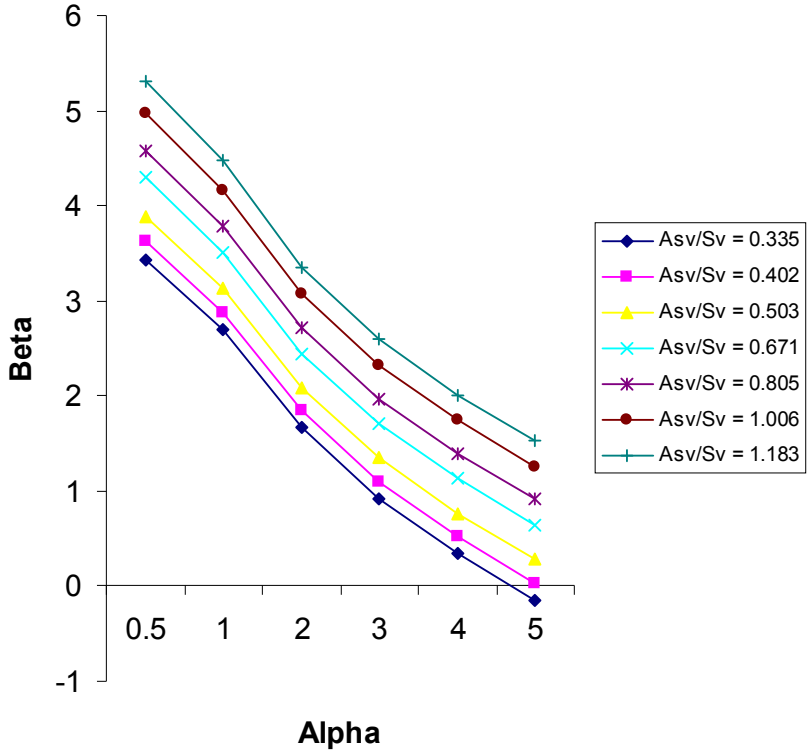


Fig.4.36: Design shear reliability for varied Asv/Sv for 6m beam

4.4 RELIABILITY DESIGN CHARTS

It can be seen from the previous reliability plots that design to BS 8110 (1985) gives varying reliability levels depending on design parameters used. Thus, beams under similar load ratio but varying geometry or vice versa will not have uniform reliability level using the code requirements.

Going by the current method of design of a singly reinforced section for instance, the moment of resistance of the tensile section is given by the expression:

$M_u = 0.87 f_y AZ$

So that

$$A_s = \frac{M_u}{0.87 f_y Z}$$

But usually in practice, applied moment M_{app} is used in place of moment of resistance M_u in the determination of area of enforcement steel.

The graphs resulting from this study can be used to carry out designs to specified reliability level and more importantly, the use of these reliability graphs allow for flexibility in choice of reliability level.

4.4.1 Uniform reliability level (ISOREL)

A specific reliability index can be prescribed for varying load ratio on reliability charts. The specific reliability index satisfying varying load ratio on different reinforcement ratio curves is herein referred to as "ISOREL". A typical graph of Figure 4.37 shows plot for constant reliability indices for tensile requirement of 4m beam.

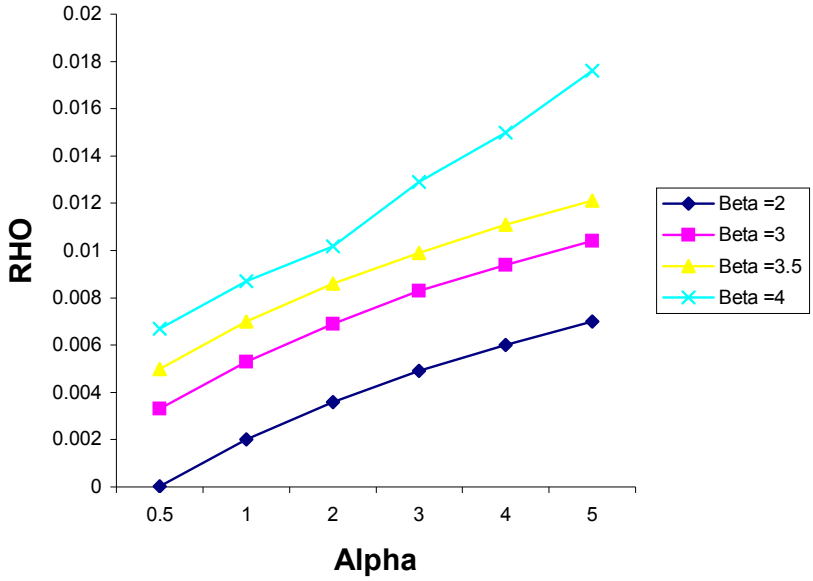


Fig. 4.37: Data for constant reliability index for single reinforcement at support of 4m beam (ISOREL).

CHAPTER FIVE

SUMMARY, CONCLUSION AND RECOMMENDATION

5.1 SUMMARY AND CONCLUSION

This study has considered a two span continuous beam designed to BS8110 (1985) using second moment first order reliability method to evaluate various performance criteria. The study also shows that the present method of design does not guarantee the same reliability when different section of the structural elements are considered. These show that reliability indices change with load and reinforcement of a span for instance, is not the same for the support of a beam section. The same is true for the reliability of the tensile portion of the beam when compared with the compression section and thus, the same failure criteria is not ensured.

For flexibility in our design, the study proposes the use of the reliability charts for the designs of reinforced concrete beam, The ideas and principles embodied in BS8110 (1985), have been used for the development of the charts.

The procedure for their use involves the determination of the ratio of dead to live loads and choosing a reinforcement ratio for a required reliability for sections under consideration. Some of the advantages of using these charts are that constant reliability indices for all design criteria are produced and any changes in the design values are appreciated.

5.2 RECOMMENDATION

For rational and optimal designs there is the need to review the present status of the code of practice BS8110 (1985), for reinforced concrete beams. This will encourage the use of more uniform reliability levels in our designs in our designs.

More research works are encouraged in the following areas:

(i) development of more charts for different loading systems and dimensions

(ii) the suitability of the present method in checking serviceability requirements

(iii) structural reliability studies be incorporated into structural engineering curriculum.

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