PROBABILISTIC ANALYSIS OF REINFORCED CONCRETE COLUMNS SUBJECTED TO CORROSION

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DECLARATION

I declare that the work in this dissertation entitled PROBABILISTIC ANALYSIS OF REINFORCED CONCRETE COLUMNS SUBJECTED TO CORROSION has been carried out by me in the Department of Civil Engineering. The information derived from the literature has been duly acknowledged in the text and a list of references provided. No part of the project thesis was presented for another degree or diploma at this or any other institution.

_____ _____ _____

Name of Student Signature Date

CERTIFICATION

This dissertation entitled ‘’**Probabilistic Analysis of Reinforced Concrete Columns Subjected to Corrosion**’’ meets the regulation governing the award of the degree of a Master of Science in Civil Engineering of Ahmadu Bello University and is approved for its contribution to knowledge and literary presentation.

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ABSTRACT

This dissertation is a report of the investigation carried out on the Probabilistic analysis of reinforced concrete columns subjected to corrosion. Corrosion of reinforcement bars is one of the most significant causes of deterioration of reinforced concrete columns. The deterioration leads to a loss of serviceability, functionality and ultimately affects the structural safety. The probabilistic analysis was carried out using MATLAB. The reliability index, β, was computed for various points on the interaction curve using First Order Reliability Method (FORM). The variations of the safety index with respect to the basic variables (Load ratio, yield strength and concrete grade) were examined to analyzed the behavior of columns subjected to corrosion, given the values of the design parameters. Graphs were developed for each mode of the columns. The results indicated that the safety index of the reinforced concrete column decreases as the load ratio of the column is increasing while an increase in yield strength and concrete grade leads to corresponding increase in the safety indices. For reinforced concrete columns and other aggressive environment prone to corrosion attack, a minimum of 0.5 load ratio, 500 N/mm2 yield strength and 30 N/mm2 of concrete grade at corrosion rate of 0.4mm/year with good anti-corrosion properties (epoxy resins) be used in the construction of columns as load carrying components is therefore recommended.

DEDICATION

This dissertation is dedicated to Almighty Allah

and my late Dad.

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CHAPTER ONE

1.0 INTRODUCTION

1.1 Introduction

Corrosion of reinforcement is recognized as the predominant factor that limits the service life of reinforced concrete (RC) structures exposed to aggressive environments. This corrosion deterioration can lead to damage resulting in capacity loss or even failure. For structures exposed to coastal marine environments or deicing or anti-icing applications, this deterioration is often accelerated. The deterioration is often attributable to the corrosion of steel, which can significantly reduce the strength of both steel and reinforced concrete structures. Robert *et al* (1997) Corrosion rates are highly variable and dependent on the local environment, quality of the material strengths, geometry and applied loads vary during the life of a structure. There is therefore a high degree of uncertainty associated with the assessment of deteriorating structures.

The corrosion deterioration in RC structures has raised significant attention from researchers and analytical studies and experimental tests have been performed worldwide. Durability and serviceability of corroded RC structures have been investigated to determine the relationship between the corrosion process and the service life of these structures and these relationships have been used to develop service life models. In addition the Predominantly, with the extensive use of de-icing salt in cold weather regions, bridge decks and bridge piers are vulnerable to corrosion of steel reinforcement (ACI 1999).

The concept of the application of reliability theory to the assessment of bridges and other structures has been accepted and adopted in North America. In the UK, the Highways Agency has recently commissioned a number of projects to develop a whole-life performance assessment method based on reliability theory.

The main corrosion-induced damage mechanisms in RC are:

(i) the decrease in the reinforcement cross-sectional area,

(ii) the possible loss of steel ductility,

(iii) the cracking and spalling of the concrete cover, and

(iv) the loss of bond along the steel/concrete interface.

There has been an increasing effort in recent years to quantify the damage progress in RC due to the advance of steel reinforcement corrosion. Particularly, quantifying concrete cracking propagation with respect to reinforcement corrosion propagation is of great relevance in the serviceability assessment of deteriorated RC structures with corroded reinforcement, and it must be included in estimations of residual service life (Andrade and Alonso, 1996; Liu and Weyers, 1998).

There have been limited studies on the effect of reinforcement corrosion on the serviceability and structural performance of RC columns (Lee *et al*, 2000; Rodriguez *et al*, 1996; Saito *et al*, 2007). A major observation from these studies is that the load carrying capacity of corroded columns is lower than that of non-corroded columns. This reduction in resistance capacity is attributed to: (a) the increase in load eccentricity due to uneven

corrosion of the longitudinal reinforcement; (b) buckling of the longitudinal reinforcing bars when column ties are corroded; and,

(c) the actual deterioration of the concrete section due to cracking, spalling and/or delamination of the concrete cover.

However, none of the above studies were conducted on RC columns that were subjected to sustained axial loads and reinforcement corrosion simultaneously.

The high alkaline environment of good quality concrete forms a passive film on the surface of the embedded steel that normally prevents the steel from further corroding. However, under the influence of chloride and carbonation, the passive film is disrupted or destroyed and the steel corrodes. The corrosion products occupy a larger volume and these induce stresses in the cover concrete resulting in cracking, delamination and spalling. In addition to loss of cover concrete, a RC member may undergo structural damage due to loss of bond between steel and concrete and loss of rebar cross-sectional area as shown in Figure 1.1

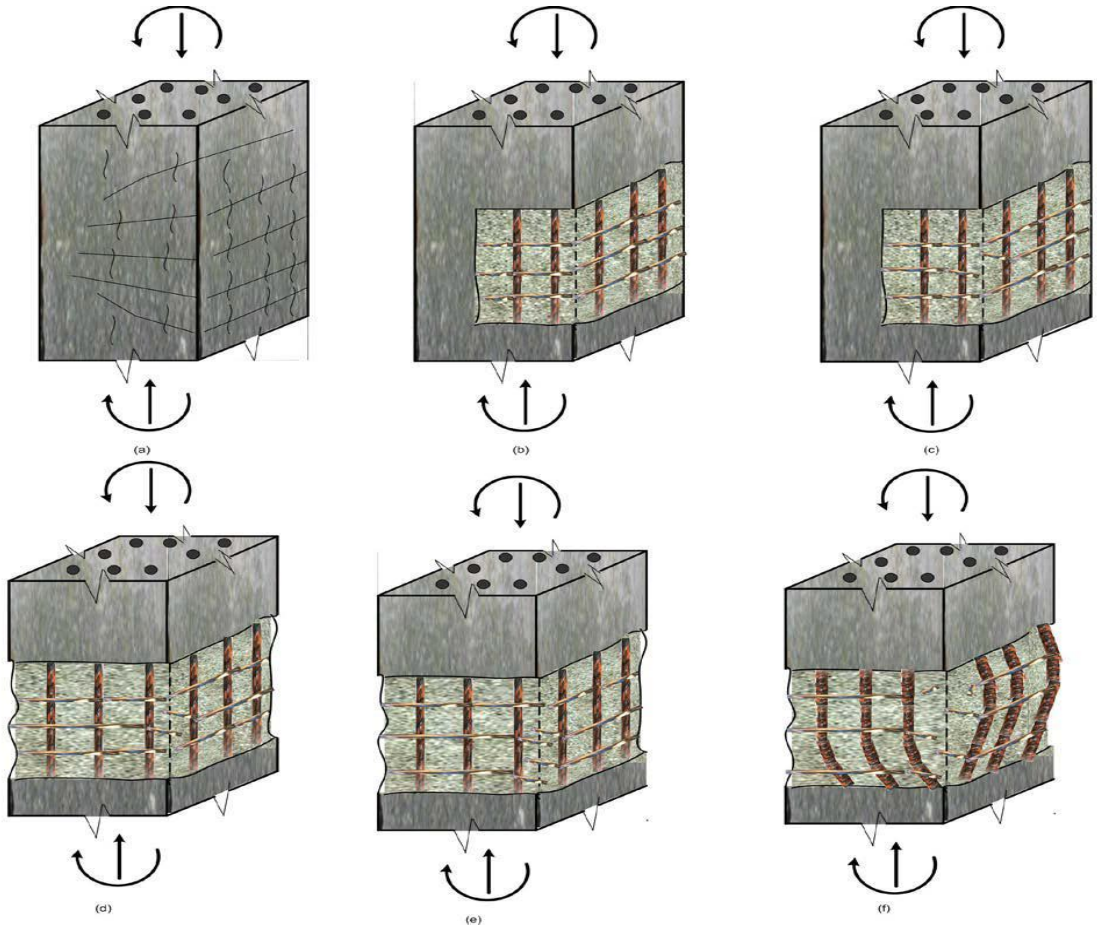


Figure 1.1: Possible damage of RC beam-column subjected to combined external loads and corrosion; (a) flexural and corrosion cracks; (b) initial spalling; (c) one stirrup failure; (d) spalling on all sides; (e) two stirrups failure; (f) loss of confinement and possible buckling.

To plan repair strategy for damaged structures, the strength of the existing structures needs to be estimated. The past research addressed on the flexural behavior of corrosion damaged concrete members (Mangat,1999; Rodriguez,1997; Almusallam, 1996) They indicated that load carrying capacity and ductility decreased as the reinforcing steel bars were corroded. Relatively limited literature exists on the axial behavior of corrosion damaged reinforced concrete columns. Uornoto studied the effects of corrosion damage on the load bearing

capacity of reinforced concrete columns. He reported that the bearing capacities of corroded columns was not simply caused by reductions in strength or effective areas of the reinforcing bars but also by cracks formed during the corrosion process. Tapan [2008] proposed a bridge pier column strength evaluation method that can be adapted into a currently used bridge condition evaluation method. The proposed evaluation method provided a good estimate of the condition and load-carrying capacity of bridge piers that currently cannot be obtained by normal visual surveys. The research studies have shown the influence of corrosion on bond characteristics between steel and concrete. They demonstrated that loss of bond increased with sectional loss.

One of the main reasons responsible for the deterioration is member corrosion and reinforcement corrosion in steel and reinforced concrete structures, respectively. Latter case structural repairing could be a harder task since the reinforcement is not easily accessible. Also in reinforced concrete (RC) structures the corrosion effect cannot be seen as simply as a reinforcement area reduction. In fact, corrosion mechanism leads to the development of several side effects (rust expansion, concrete cracking, bond strength decreasing, among others) responsible for the bridge deterioration acceleration. For this reason it is fundamental to correctly assess the reliability of an existing corroded RC structure in order to adequate a safety service level. On the other hand, the structure reliability or the safety level decreasing due to damage occurrence is related to the robustness concept which has seen growing interest in the last decades as a result of the occurrence of tragic consequences (Eagar and Musso 2001, Pearson *et al*. 2003, NTSB 2008) due to extreme events such as terrorist attacks. However the concept can also be

very useful when applied to deterioration scenarios allowing for instance for a reinforcement concrete column to evaluate the safety susceptibility to corrosion.

Therefore this paper intends to be a contribution to the robustness assessment of reinforced concrete structures subjected to corrosion.

The design of any structure should be able to safely meet the requirements of functionality, aesthetics and economy (Clarke, and Converman,1987). When an engineering structure is loaded in some way it will respond in a manner which depends on the type and magnitude of the load, and the strength as well as stiffness of the structure. Whether the response is considered satisfactory depends on the requirements which must be satisfied.

1.2 Problem statement and Justification of Study

The aim of structural design is to ensure that the designed structure withstands the load to which it is subjected. Safety level assessment is therefore carried out to determine the safety inherent in the structure. It gives an estimate of the safety or failure conditions of the structure taking into account the various uncertainties associated with the design and constructions of the structure, thus reducing structural accidents and minimizing losses during the associated design life of the structure.

It is in this bid that the formulation for reliability-based performance of reinforced concrete columns is evaluated in this work in order to obtain the safety level inherent in the concrete structure as design using the relevant codes aforementioned, also the prediction for durability of reinforced concrete columns exposed to marine environment as reviewed from literature will be evaluated as necessary.

1.3 Aim and Objective

1.3.1 Aim

The aim of this research is to carry out a Probability analysis of reinforced concrete columns subjected to corrosion.

1.3.2 Objective

The research objectives are to:

i. Identify the modes of failure of reinforced concrete columns;

ii. Develop limit state function for each failure;

iii. Develop reliability-based analysis program for the column using MATLAB; and.

iv. Perform reliability-based durability analysis on the column to investigate the effect of uncertainty and corrosion.

1.4 Scope of the Study

The research work covers the probabilistic analysis of reinforced concrete columns subjected to corrosion. The column are designed based on BS8110(1997). The data on the statistical properties of the basic design variables would be obtained from previous research.

CHAPTER TWO

2.0 Literature Review

2.1 Review of works on Structural Reliability

Structural reliability theory has been used by many researchers to measure the structural performance of reinforced concrete bridges deteriorating due to corrosion of steel reinforcement. Frangopol *et al* (1997). However, the deterioration models that have been used in the past are mainly generalized mathematical models which do not reflect the local variation associated with the corrosion process, which are likely to be columns-specific. The effects of corrosion of the steel bars on the bond strength between the steel reinforcement and the concrete has generally not been considered.

Reliability analysis is a method for calculating the probability of a structure exceeding a failure limit state. In a reliability analysis the uncertainty associated with a variable, which is expressed in terms of its probability density function (PDF), is explicitly taken into account, whereas in limit state deterministic analysis the variable is assigned a characteristic value and a partial safety factor to account for extreme events. Reliability analysis begins with the formulation of a limit state function, which represents the performance of a structure, in terms of a number of basic random variables. The basic reliability problem considers only one load effect, S, resisted by one resistance, R, which are described by known PDFs, $F_S(S)$and $F_R(R)$. The limit state function for an element is defined as follows:

$$M = R-S \qquad 2,1$$

where M is the safety margin.

The structural element will be considered to have failed if R is less than S (i.e. $M \leq 0$). If R and S are statistically independent, then the probability of failure, P_f, of the structural element can be determined by the following equation:9

$$P_{f=}\ P\ (R - S\ \leq 0) = \int_{-\infty}^{+\infty} F_R\ (X) F_S(X) d_X \qquad 2.2$$

where $F_R(x)$ is the cumulative distribution function of R, defined as $F_R(x)=P(R < x)$, or the probability that the actual resistance R is less than some value x and $f_S(x)$ represents the probability that S has a value between x and $(x + \delta_x)$ in the limit as δ_x approaches zero.

By considering all possible values of x, that is, by taking the integral over all x, the failure probability is obtained. This integral is known as the `convolution integral', and is shown diagrammatically in Fig. 2.1

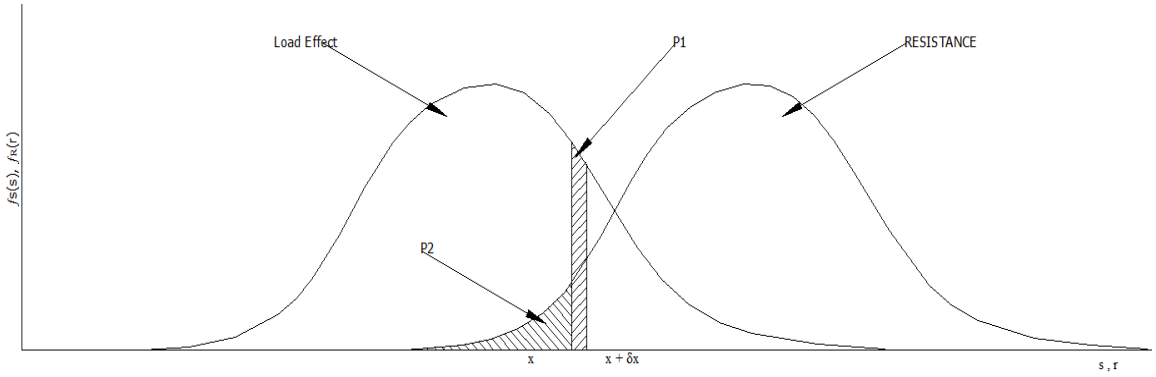


Fig.2. 1. Graphical description of the `convolution integral'

For the general case, closed-form solutions do not exist for this integral. There are, however, a number of distributions for which it is possible to integrate the convolution integral.

In general, a limit state equation is more complex, with R and S being functions of several basic random variables such as material properties, element or structural dimensions, and material densities, and may not be independent. In this case, limit state equation or failure function may be expressed in terms of the set of n basic variables, $\bar{X}$, where $\bar{X}$ = $X_1,X_2,...,X_n$ which affect the structural performance, such that

$$M = G(X_1,X_2,...,X_n) \leq 0 \qquad 2.3$$

corresponds to failure.

The surface defined by M = 0 is commonly referred to as the failure boundary for the limit state under consideration. The probability of failure is equivalent to the integration of the joint distributions over the failure region and is given by

$$P_f = P(M \leq 0) = \int \dots \int f_{X1,X2,\dots,Xn}(X_1, X_2 \dots \dots X_n) X d_{X1}, d_{X2}, \dots \dots d_{Xn} \qquad 2.4$$

where $f_{X1,X2,\dots,Xn}(X_1, X_2, \dots \dots X_n)\, d_{X1}, d_{X2}, \dots \dots d_{Xn}$ is the joint PDF for the n basic variables $\bar{X}$.

If the failure function given by equation (2.3) is linear and the basic random variables are normally distributed, then the following relationship may be obtained:

$$P_f = \Phi(-\beta) \qquad 2.5$$

where Φ is the standard normal cumulative distribution function and β is termed the reliability index and defined by

$$\beta = \frac{\mu_M}{\sigma_M} \qquad 2.6$$

where μ_M and σ_M are the mean and standard deviation of M, respectively.

In general it is not possible to evaluate the probability of failure function Pf directly, as integration of the joint PDF is numerically complex. Numerical integration can only be applied efficiently for problems involving a small number of variables, generally less than six. Analytical solutions for this formulation do not generally exist for the majority of practical problems. However, approximate methods such as first order and second order reliability methods (FORM/SORM) have been developed to solve this formulation. Monte Carlo simulation, which became popular with the development of variance reduction techniques such as importance sampling, can also be employed. There are several proprietary reliability analysis software libraries now available to calculate the reliability index using appropriate methods such as FORM, SORM and Monte Carlo simulation. Reliability analysis can also be used to determine the relative importance of an input variable to the calculated reliability index. This can be used to identify the most important variables and resources targeted to refining their probabilistic model.

Structural reliability theory has been used by many researchers to measure the structural performance of reinforced concrete bridges deteriorating due to corrosion of steel reinforcement. The effects of corrosion of the steel bars on the bond strength between the steel reinforcement and the concrete has generally not been considered. Early study in the field of structural reliability were carried out by Freudenthal (1945), who brought the method of probability and statistics to bear on the characterizing the nature of factor of safety. An illumination account of historical development of the subject is available in the book by Madsen *et al*, (1986). Besides, this book also provides a rigorous account of reliability index based approaches. Some of the other earlier books relevant to this field are those by Pugley (1962), Bokotin (1969), Tichy and Vorlicek (1972), Benjamin and Cornell

(1970), Ang and Tang (1975, 1984), Leporati (1979), Ditlevsen (1981), Elishakoff (1983), Augusti, Baratta and Casciati (1984), Yao (1985), Thoft-christensen and Morutsu (1986) and Madsen (1988). Work of more recent vintage include those by Wen (1990), Soong and Grigoria (1993), Dai and Wang (1992), Nigam and Narayanan (1994) Melchers (1999) and Ditlevsen and Madsen (2005).

Several specialized review articles have also been published. Thus, Shinozuka (1983) presents a discussion on definition and calculation of reliability indices. Method for calculating probability of failure is critically reviewed by Scheller strix (1987). Bjerager (1990) presents a state art review of structural model and random field reliability models. Development in the area of stochastic finite element method (SFEM) have been discussed by Nakagiri (1987), Benaroya and Rehak (1988), Shinozuks (1991), Der Kiureghian *et al*., (1991) Kleiber and Hien (1992) and Liu *et al*.,(1992). Melchers (1999) has outlined the developments of the reliability analysis existing structures and discussed the needs for further research.

As in other branches of structural mechanics, the emergence of computational power in recent years has strongly influenced the developments in this subject. Other factors that have contributed to the growth of the subject are the increased reliability of data.

The uncertainties in structural parameters such as material properties, external loads, geometry, etc., have caused serious attentions to reliability in structural design and analysis (Ghasemi and Yousefi, 2011). Thus, reliability theory, as a branch of theory of probability, provides a firm framework which can introduce a proper factor of safety when required (Reynolds, 2008; ISSC, 2006; Faber and Sorensen, 2002; Smith, 1976). Thus any system made of a satisfied reliability index, may be referred to as safe.

2.1.1 Theoretical framework on structural Reliability

The study of structural reliability is concern with the calculation and prediction of the probability of limit state violation for engineered structures (Melchers, 1999).Ditlevsen and Madsen (2005) considered structural reliability as a method that attempt to treat rationally, the various sources of uncertainties. According to Tsompanakis and Papadrakakis (2000) structural reliability analysis is a tool that assist the design engineer to take in to account all possible uncertainties during the design and construction phases and the lifetime of a structure in order to calculate its probability of failureTsompanakis and Papadrakakis (2000).

Structural reliability analysis is a tool that assists the design engineer to take in to account all possible uncertainties during the design and construction phases and the lifetime of a structure in order to calculate its probability of failure P_f. To *Stanley et al*, (1978) reliability-based design considers the probability that the structure will last a given length of time against the agents that can cause it to fail.

Reliability-based design is a probabilistic design process where the load and strengths of materials and sections represented by their known or postulated distributions, defined in terms of distribution type, mean and standard deviation. The probability of failure (P_f) for a specific design case is calculated as the probability that the maximum total load effect exceeds the resistance to failure. This is opposed to limit state design (LSD) method, which is a semi-statistical design process in which the probabilistic aspects are treated at the code development stage (KadrySmaili, 2007) in order to define the characteristic values and partial safety factors for load and resistance that are used to ensure, on average an acceptably low probability of failure across a full spectrum of design cases.

In the current design codes in Europe, LSD is used as a practical method of incorporating reliability method in the normal design process. This approach fails to fully address the statistical nature of basic design variable. Issues of uncertainties are lumped conservatively within partial safety factors. A limit state code is calibrated to ensure that selected appropriate reliability levels, called target values, are attained in design. (Zimmerman, *et al*, 1998).

In reliability-based design concept, the performance function of a structural system according to a specified mission is given by:

M = performance criterion - given criterion limit = $g(X_1, X_2, \ldots, X_n)$ (2.7)

in which the X_i(1=1,……,n) are the n basic random variables (input parameters), with g( ) being the functional relationship between the random variables and the failure of the system. The performance function can be defined suchthat the limit state of failure surface, is givenby M = 0.the failure event is define as the space where M > 0. Thus the probability of failure can be evaluated by the following integral.

$P_f = \iint \ldots \int f_x(X_1, \ldots., X_n) dx_i \ldots dx_n$ (2.8)

Where f_x is the joint density function of $X_1, X_2, \ldots., X_n$ and the integration is performed over the region where M < 0. Because each of the basic random variables has a unique distribution and they interact, the integral cannot be easily evaluated. Therefore the use of approximate method is made.

Traditionally, the concern of researchers was on the evaluation of structural reliability of steel and concrete structures and/or components. (e.g. Rosowsky *et al*, 2002; Jiquanand Baidurya, 2007 Afolayan and Abubakar, 2003; Afolayan and Opeyemi, 2008; Holicky and Retief, 2005; Hassan, 2006; Chinwedu, 2002). Due to the performance of timber structures during recent extreme events, such as the Northridge earthquake, and the Hurricane Andrew (Mohammed, 2006), much attention is being tailored on the reliability of wood frame components and assemblies under normal and accidental loading (such as : Rosowsky and Ellingwood, 2002; Rosowsky, 2002; Filiatrault and Folz, 2001, 2002; Afolayan 2005; van de Lindt, 2005; Foliente et al, 2001; Afolayan, 2004; Afolayan and Abdulkareem, 2005; Mohammed, 2006; van de Lindt and Mathew, 2003).

Study on the adequacy of the current design codes, that are based on limit state design philosophy are been assessed, as many attempt were made to calibrate this new generation codes (such as: Holicky and Gurvanessian, 2002; Mike and David, 2002; Nowak, 1995;

Ranta-maunus *et al*, 2001; Ranta-maunus and Tomi,2002; Ranta-maunus, 2004; Trezos and Thomas, 2002; Girgio and Silvia, 2002).

2.2 Concept of Structural Reliability Analysis

The fundamental concept of reliability analysis is that the resistance and load factors are statistical quantities with a central tendency(mean), dispersion about the mean (variance) and some form of distribution (probability density function, e.g. normal, gumbel, lognormal, etc). When combined via an expression to describe the limit state (such as failure and collapse) there will be a finite probability that the load will exceed the resistance; this defines the probability of failure (p) and since the reliability is a compliment of failure which is equal to (1-P), the inherent reliability of the component against a particular failure mode, and with a given resistance properties is defined.

In determining the overall reliability of a structural system, account must be taken care of possible failure modes of the structure or structural systems, making due allowance for the correlation arising from the common sources of loading and common material properties. However, from the definition of reliability as the probability that a structure will not attain its limit state (ultimate or serviceability) during a specified reference period, there is thus a need to determine reference period since majority of the structural loads vary with time, in a certain manner. The probability that any specified load intensity will not be exceeded in a fixed time interval is a function of the interval; hence, structural reliability is dependent of exposure of the loading. Also it is affected by material property change with time. Except for rare cases where load and strength are constants then reference period can be ignored. In reliability assessment, it is necessary to determine the probability that the structure will not any of the limit states likely to be violated within a reference period, in presence

uncertainties such as conditions of service, human error, change in load material properties with time e.t.c the probability of structural survival can be mathematically represented as;

$$R=1-P \tag{2.9}$$

Where R: the survival probability and P is the probability of failure.

2.3 Reliability-based Design

Lu *et al*. (1994) pointed out that probability-based structural design became practically realizable in the 1970's and its conceptual framework was developed by Ang and Cornell (1974) influenced by Freudenthal's pioneering work on structural safety (1947). Evaluation of structural safety associated with the design procedure was studied by Ellingwood and Ang (1974) for reinforced concrete beams in pure bending and pure shear. The so-called safety index method was also used by Ravindra *et al*. (1974) to design reinforced concrete beams and structural steel members. In these early studies, the reliability index was calculated by the MVFOSM (Mean-Value First-Order Second Moment) method, which lacks invariance with respect to the formulation of the limit-state function, i.e., equivalent formulations of the limit state lead to different values of the reliability index. Since then, rapid developments have occurred in reliability analysis methods and reliability-based structural design, generating a rich literature on the subject. Ellingwood (1983) further investigated probability-based design criteria, using the FOSM (First-Order Second-Moment) method, which is invariant with respect to the limit-state formulation. Although originally formulated in a deterministic framework, current design codes, such as ACI 318 (1989) for reinforced concrete structures, have gradually incorporated results of probability theory. Once the limit-state functions are defined, usually as complicated functions of the

basic variables, a versatile and efficient reliability algorithm is necessary to estimate the corresponding modal probabilities of failure or a directly related quantity called the reliability index. Reliability analysis methods such as FORM (First-Order Reliability Methods), SORM (Second-Order Reliability Methods) and directional Monte Carlo simulation have been developed extensively in recent years (Melchers, 1983) and implemented in general-purpose computer programs (Lu *et al*., 1994).

In summary, researches carried out thus far, show that reliability-based design require a careful evaluation of the uncertainties in the design variables which can best be assessed in terms of basic variabilities and prediction errors (Ellingwood & Ang, 1974).

2.4 First Order Reliability Method (FORM)

The first-order reliability method, (FORM), is a full probabilistic reliability analysis method devised to evaluate the reliability of a system. The first development of FORM took place almost 30 years ago. Since then the methods have been refined and extended significantly and by now they form one part of the most important methods for reliability evaluations in structural reliability theory. Several commercial computer codes have been developed for FORM analysis and the methods are widely used in practical engineering problems and for code calibration purposes. It is considered to be one of the most reliable computational methods for structural reliability. Its accuracy is generally dependent on three parameters, i.e., the curvature radius of the design point, the number of random variables, and the first-order reliability index.

First order reliability method (FORM) is a convenient tool for accessing the reliability of structural elements. It also provide a means for calculating the partial safety factor, FORM uses a combination of analytical and approximate methods and comprises stages. Firstly,

independent of whether each parameter has been defined as Normal, Log-Normal, or Gumball distribution; all variables are first transformed into equivalent normal space with zero mean and limit space surface. Secondly, the shortest distance between the origin and the limit space surface, termed the reliability index β is evaluated; this is known as the design point, or point of maximum likelihood, and gives the most likely combination of basic variables to cause failure. Finally the probability associated with this point is then calculated. FORM can be easily extended to non-linear states and has a reasonable balance ease of use and accuracy.

2.4.1First and Second Order Reliability Method

First and Second Order Reliability Method (FORM/SORM) are other approximation methods. Safety level of structure is measured by a reliability index. There are different definitions of reliability index, β. The first was introduces by Cornell in the late 1960s (Cornell, 1969). Given the performance function as:

$$\text{M} = R - S = g(X_1, X_2, \ldots, X_n) \tag{2.10}$$

The mean and the standard deviation of the margin can be defined as:

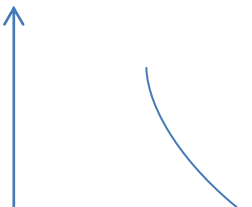
$$\mu_M = \mu_R - \mu_S \tag{2.11}$$

$$\sigma_M = \sqrt{\sigma_R + \sigma_S} \tag{2.12}$$

And the safety index is given by:

$$\beta_c = \frac{\mu_M}{\mu_S} \tag{2.13}$$

This reliability index is define as the shortest distance between the failure surface defined by G(X)= 0 and the standard origin as shown on the figure below.



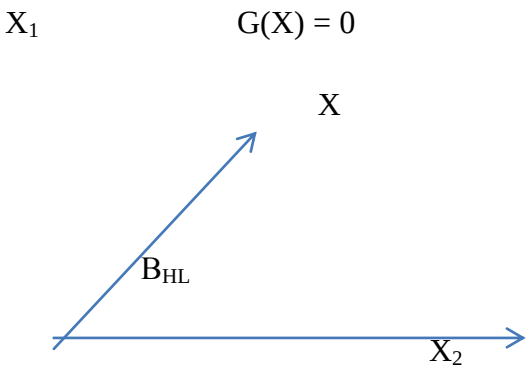


Fig 2.2 Reliability index β and the limit function of the standardized space the standardized variables are expressed as:

$$X = \frac{x-\mu_x}{\sigma_x} \qquad (2.14)$$

The point X on the failure surface is of considerable importance. It is the so called design point. If a design is carried out using values from the design point this will result in the highest probability of failure, therefore the name. If the limit state function is linear and the variables are non-correlated, the shortest distance, that is β_{HL}, can be found directly using basic algebra. However, this is seldom the case. When the limit state function is nonlinear, approximate methods must be considered. For example, geometrical optimization of the problem which can be solve numerically or analytically. The analytical methods are usually based on a first order Taylor approximation of limit state function. That is why the method is called First Order Reliability Method (FORM) (thoft-Christensen and Baker, 1982).

An iterative method that can be used to find β_{HL}, X_i and an estimate of the probability of failure is the Rackwitz algorithm(Ang and Tang, 1984). The design point can be expressed in scalar form.

$$X_i = -a_i \beta_{HL} \qquad (2.15)$$

where a_i are the direction cosines (the unit vector) in the X_i direction.

$$a_i = \frac{A}{B} \tag{2.16}$$

where $A = \frac{\partial g}{\partial x_i}$ (2.17)

and $B = \sqrt{\left[\sum(\frac{\partial g}{\partial x_i})^2\right]}$ (2.18)

First Order Reliability Method (FORM) and the Second Order Reliability Method (SORM) are based on first order approximation of the limit state at the design point (Cheng, 2007; Melchers, 1999; Ditlevsen and Madsen, 2005; Rackwitz, 2000). FORM/SORM requires the calculations of the performance function and/or its partial derivatives with respect to the basic random variables at each iteration step.

A large number of calculations of the performance function and/ or its partial derivatives are therefore needed (Deng, *et al*, 2005). Such calculations may be performed effectively when the performance function g(x) can be expressed as an explicit form or analytical form in terms of the basic variable X. When the performance is implicit, however such calculation requires additional effort and may be time consuming (Deng *et al*, 2005). In many cases of practical importance, particularly for complicated structures, the limit state functions are usually implicit in terms of the random variables. Therefore, the limit state functions are not readily available. Another issue is capability of the conventional structural reliability methods to detect the global minimum during the search for reliability index (Rackwitz, 2000).

2.4.2 Probabilistic Methods of Safety Checking

Methods of probability analysis are classified on the basis of type of calculations and approximation made. There are different levels of reliability analysis, which can be used in any design methodology depending on the importance of the structure. The term “level” is characterized by the extent of information about the problem that is used and provided. The methods of safety analysis proposed currently for the attainment of a given limit state

can be grouped under four basic “levels” (namely levels IV, III, II, and I) depending upon the degree of sophistication applied to the treatment of the various problems.

- In level 1 methods, the probabilistic aspect of the problem is taken into consideration by introducing into the safety analysis suitable “characteristic values” of the random variables, conceived as fractile of a predefined order of the statistical distributions concerned. These characteristic values are associated with partial safety factors that should be deuced from probabilistic considerations so as to ensure appropriate levels of reliability in the design. In this method, the reliability of the design deviate from the target value, and the objective is to minimize such an error. Load and resistance factor design (LRFD) method comes under this category
- Level II methods are reliability methods, which employ two values of each uncertain parameter (i.e., mean and variance), supplemented with a measure of correlation between parameters. The method involves certain approximate iterative calculation procedure to obtain an approximation to the failure probability of a structure or structural system.
- Level II methods encompass complete analysis of the problem and also involve integration of the multidimensional joint probability density function of the random variables extended over the safety domain. Reliability is expressed in terms of suitable safety indices, viz, reliability index, β and failure probabilities.
- Level IV methods are appropriate for structures that are of major economic importance, involve the principles of engineering economic analysis under certainty, and consider costs and benefits of construction, maintenance, repair, consequences of failure, and interest on capital, etc. foundations for sensitive

projects like nuclear power projects, transmission towers, highway bridges are suitable objects of level IV design.

Z in equation (3.9) is a function of more than one variable, only if Z is a linear function and all variables are normally distributed (and independent) equation (3.9) is an equality and not an approximation

2.4.3 Computaton of Reliability Index

As both "load" S and "strength" R may depend on more than one variable, in order to perform a level II calculation, the variables $X_1, X_2, \ldots\ldots X_n$ have to be independent and it must be possible to linearize the reliability function Z. suppose the reliability function fulfills this requirements and the variables X_i are all normally distributed and independent then

$$Z_{lin} = Z(X_i^*, X_i^*, \ldots\ldots X_n^*) + \sum_{i=1}^{n} \left(\frac{\partial z}{\partial xi}\right)_{x_i = x_i^*} (X_i - X_i^*) = 0 \qquad 2.19$$

Z_{lin}=linearized reliability function

$\left(\frac{\partial z}{\partial xi}\right)_{x_i = x_i^*}$ = partial derivative of Z with respect to x_i evaluated in $x_i = x_i^*$

The mean value and standard deviation of Z_{lin} are

$$\mu_{Z_{lin}} = Z(X_i^*, X_i^*, \ldots\ldots X_n^*) + \sum_{i=1}^{n} \left(\frac{\partial z}{\partial xi}\right)_{x_i = x_i^*} (\mu_{x_i} - X_i^*) \qquad 2.20$$

$$\sigma_{Z_{lin}} = \sqrt{\sum_{i=1}^{n} \left(\frac{\partial z}{\partial xi}\right)^2_{xi = xi^*} \sigma_{xi}^2} \qquad 2.21$$

respectively.

If the mean values $X_i^* = \mu_{xi}$…..$X_n^* = \mu_{xn}$ are substituted, a so called mean value approximation is obtained, if the failure boundary is nonlinear a better approximation can be achieved by linearising the reliability function in the design point, which is only defined if the variables are normally distributed (or are transformed). The design point is defined as a point on the failure boundary in which the (joint) normal probability density function is maximum.

The design point $X_i^* = \mu_{x_i} - \alpha_i \beta \sigma_{x_i}$ 2.22

Where;

α_i= the factor of influence for the variable $= \frac{\sigma_{x_i}}{\sigma_{z_{lin}}} \frac{\partial z}{\partial xi}$ 2.23

Unless Z is a linear, the design point cannot be determined directly.

The research work contains two (2) parts both of which require the use of FORM. In the first part, Sensitivity analysis of some of the basic design variables on the reliability index (β) of the various failure modes of the flat slab, the variables are therefore taken in turn and varied and their effect on the reliability index (β) is noted.

2.5 Interaction between Resistance and Applied Load.

It has been established that design criteria may be formulated most rationally within the probabilistic framework, as lack of invariance in a possible deficiency in deterministic safety measures. The reliability of a reinforced concrete slab is the complement of its probability of a failure, P_f.For a reinforced concrete slab, failure is easily defined as a limit state where the structural member is either damaged, failed, or collapsed and cannot perform its intended function. P_f is the probability that the load effect S exceeds the structure strength or resistance R. as stated above, the problem of a reliability of a structure, may be cast basically on as a problem of the load or stress, S, versus strength, R.

It should be noted that S may be obtained from the applied load Q through a structural analysis (either deterministic or with a random component s). It is important that R and S are expressed in the same units. Only the safety of the structural member is considered here for convenience. Thus a structural member will be considered to have failed if the strength, R is less than the stress, S acting on it. The objective of reliability analysis is to ensure that probability P(R≤S) be smaller than a predefined value. Therefore the probability of failure P_f of the structural element can be stated in any of the following ways.

$$P_f = P(R \leq S) \quad (2.24)$$

$$= P(R - S \leq 0) \quad (2.25)$$

$$= P\left(\frac{R}{S} \leq 1\right) \quad (2.26)$$

$$= P\left(\ln R - \ln S \leq 0\right) \quad (2.27)$$

Or in general

$$P_f = P[G(R, S \ln S \leq 0)] \quad (2.28)$$

Where G is termed "limit state function" and the probability of failure is identified with the probability of limit state violation. Hence, Equation (3.15) to (3.19) could, of course, also have been written in terms of R and Q for the structure as a whole.

Each of S and R is described by a known probability density function, $f_S(S)$ and f_R respectively. The required probability may be then formulated as follows.

$$P_f = P(R \leq S) = \sum_{alls} P\left(R \leq \frac{S}{s} = s\right) P(S = s) \quad (3.20)$$

Equation (3.14) can be expressed in terms of the joint PDF as follows.

$$P_f = P(R - S \leq 0) = \int_0^{\infty} . \int_0^{s} f_{RS}(r, s) d_r d_s \quad (2.29)$$

Where R and S are statically independent, $f_{RS}(r, s) = f_R(r) f_S(s)$

Then (3.21) becomes.

$$P_f = P(R - S \leq 0) = \int_{-\infty}^{\infty} . \int_{-\infty}^{s>r} f_{RS}(r,s) d_r d_s \quad (2.30)$$

For any random variable x, the cumulative distribution function $F_x(X)$ is given as:

$$F_x(X) = P(X) \leq x = \int_{-\infty}^{x} f_x(y) d_y \quad (2.31)$$

Provided that $X < Y$. It follows that for the common, but special case when R and S are independent, equation (3.19) can be written in the form.

$$P_f = P(R - S \leq 0) = \int_{-\infty}^{x} F_R(x) f_S(x) d_x \quad (2.32)$$

Alternatively, equation (3.19) can be expressed as follows.

$$P_f = P(R - S \leq 0) = \int_{-\infty}^{\infty} [1 - F_S(x)] f_S(x) d_x \quad (2.33)$$

For a few distributions of R and S, it is impossible to integrate the convolution integral (3.24 and 3.25) analytically. The most notable example is when S and R are normal random variables with means of $\mu_R$and μ_S and variances σ^2_R and σ^2_S respectively.

Therefore, the safety margins Z=R-S, with a mean and variance given by

$$\mu_Z = \mu_R - \mu_S \quad (2.34)$$

$$\sigma^2_Z = \sigma^2_R + \sigma^2_S \quad (2.35)$$

Using well-known rules for addition (subtraction) of normal random variables, Equation (3.28) then becomes.

$$P_f = P(R - S \leq 0) = P(Z \leq 0) = \Phi\left(\frac{0-\mu_R}{\sigma^2}\right) \quad (2.36)$$

Where μ is the standard normal distribution function (zero mean and unit variance) are extensively tabulated in statistical texts.It may observe that the reliability is a function of the ratio of$\frac{\mu_Z}{\sigma_Z}$, which is called "Reliability index" and is denoted by β.it follows that:

$$P_f = \Phi\left[\frac{-(\mu_R - \mu_S}{\sigma_S^2 + \sigma_R^2}\right] = \Phi(-\beta) \quad (2.37)$$

Another in structural engineering is the "safety factor", which is defined as:

$$\Leftrightarrow = \frac{R}{S} \quad (2.38)$$

Since R and S are random variables, so is⇔. Its distribution function may be derived from those of R and S. For many problems, the simple formula above is not adequate, since it may not be possible to reduce the structural reliability problem to a simple R versus S formation, with Rand S independent variables. It is possible to have a situation where R and S are independent, such as when some loads act oppose failure (e.g. overturning) or when the same dimension affect both R and S. in this case, it is not valid to use the convolution integral (3.24 or 3.25). It is not valid when there is more one applied stress resultant acting at a section, or more than one factor contributing to the resistance of the structure (Melchers, R.E.,1987). A more general formulation is required. Basic variable are fundamental variable which define and characterize the safety of the structure. Basic are usually the variables employed in conventional structural analysis and design. Typical examples are dimensions, densities or unit weights, materials, loads, material strengths.

Dependence between basic variables usually adds complication to a reliability analysis. It is important that the dependence format between the dependent variables be known and expressible in some form. This will usually be in the form of correlation matrix. The probability distributions for the basic variable can be obtained by direct inference from observed data, by subjective assessment or by some combination of these techniques. Past observation and experience for similar structures can be validly used for the structure under assessment. Sometimes physical, reasoning maybe used to suggest an appropriate probability distribution. Finally when a model's parameters have been selected the model should be compare with data if at all possible. If vector X represent the basic variables of the problems, then resistance R could be expressed as $R = G_R(X)$ and the loading or load effect as $S = G_S(X)$.

The function G_R and G_S may be non-linear, and in general the cumulative distribution function F_R, must be obtained over the multiple integration over the relevant basic variables, thus.

$$F_R(r) = \int^{r} \dots \int f_x(x) d_x \qquad (2.39)$$

A similar applies for S and F_S.

Fortunately, it is always necessary to derived F_R or F_S respectively, since the limit state function G(R,S), in equation (3.21) can also be generalized. When the function $G_R(X)$ and $G_S(X)$ are used in G(R,S), the resulting limit state function can be written as G(X), where X is the vector of all relevant basic variables and G( ) is some function expressing the relationship between the limit state and the basic variables. The limit state equation G(X) = 0 now define the boundary between the satisfactory "safe" G > 0 and unsatisfactory "unsafe" G[0 domain in n-dimensional basic variable space.

It follows that, with the limit state function expressed as G(X), the generalization of the equation (3.21) is

$$P_f = P[G(X)(0)] = \int \dots \int_{G(x)\leq 0} f_x(x) d_x \qquad (2.40)$$

Here $F_x(x)$ is the joint probability density function for the n-vector X of basic variables. Note that the resistance R and load effect S need no longer be independent, as each is now described in terms of the basic variable X. In general, it is preferable for the basic variables themselves to the independent, if so, then:

$$F_x(x) = \prod_{i=1}^{n} f_{xi}(x_i) = f_{x1}(x_1) f_{x2}(x_2) f_{x3}(x_3) \qquad (2.41)$$

Where $f_{xi}(x_i)$ is the "marginal" probability density function for the basic variable X_i. The region of integration $G(X) \leq 0$ in equation (3.24) denotes the space of limit state violation and is directly analogous to that for the simple R-S≤0 limit state equation. Except for the some special cases, the integration over the failure domain $G(X) \leq 0$ cannot be performed analytically.

2.6 Evaluation of the Performance Function using Genetic Algorithm (GA) -Based FORM

Genetic algorithms (GA) are adaptive heuristic search algorithm based on evolutionary ideas of natural selection and genetics. They represent an intelligent exploitation of a random search used to solve optimization problems (Sivaraj and Ravicharand, 2011). The basic techniques of the gas are designed to stimulate processes in natural systems necessary for evolution; especially those that follow the Charles darwin's principle of "survival of the fittest".

This principle is similar to competition among individuals for scanty resources in nature resulting in the fittest individuals dominating over the weaker ones. GA simulates the survival of the fittest among all the individuals in the population over successive generations for solving a problem. Each generation consists of a population of character strings that are analogous to the chromosome that are seen in DNA (Deoxyribonucleic acid). Each individual represents a point in a search space and a possible solution. The individuals in the population are then made to go through a process of evolution.

GA is based on an analogy with the genetic structure and behavior of chromosomes within a population of individuals using the following foundations: individuals in a population compete for resources and mates. Those individuals most successful in each competition will produce more offspring than those individuals that perform poorly. Genes from good individuals propagate throughout the population so that two good parents will sometimes produce offspring that are better than either parent. Thus each successive generation will become more suited to their environment (Goldberg, 1989).

2.6.1 Structural Reliability Using Genetic Algorithms

Many human inventions were inspired by nature (Dyer, 2003), notably, the use of artificial intelligence in civil engineering domain. Advancement in artificial intelligence and its applications have led to the development of comprehensive search algorithms that lend themselves to immediate use in the reliability evaluation of structural systems and the identification of dominant failure modes. Areas that attracted considerable research outputs are the genetic algorithms and artificial neural networks (Lagaros and Papadopoulus, 2006).

According to Rackwitz (2000), genetic algorithms (GA) is known to be capable of detecting global minima during the search for the reliability index. The Genetic algorithm is an adaptive heuristic search method based on population genetics. Genetic algorithm were introduced by John Holland in the early 1970s (Manoj *et al*, 2010; Black, 1996) It is an iterative procedure that consists of a constant size population of individuals, each one represented by a finite string of symbols, known as 'genomes', encoding a possible solution in a given problem space. This space referred to as the search space, comprises all possible solution to the problem at hand.

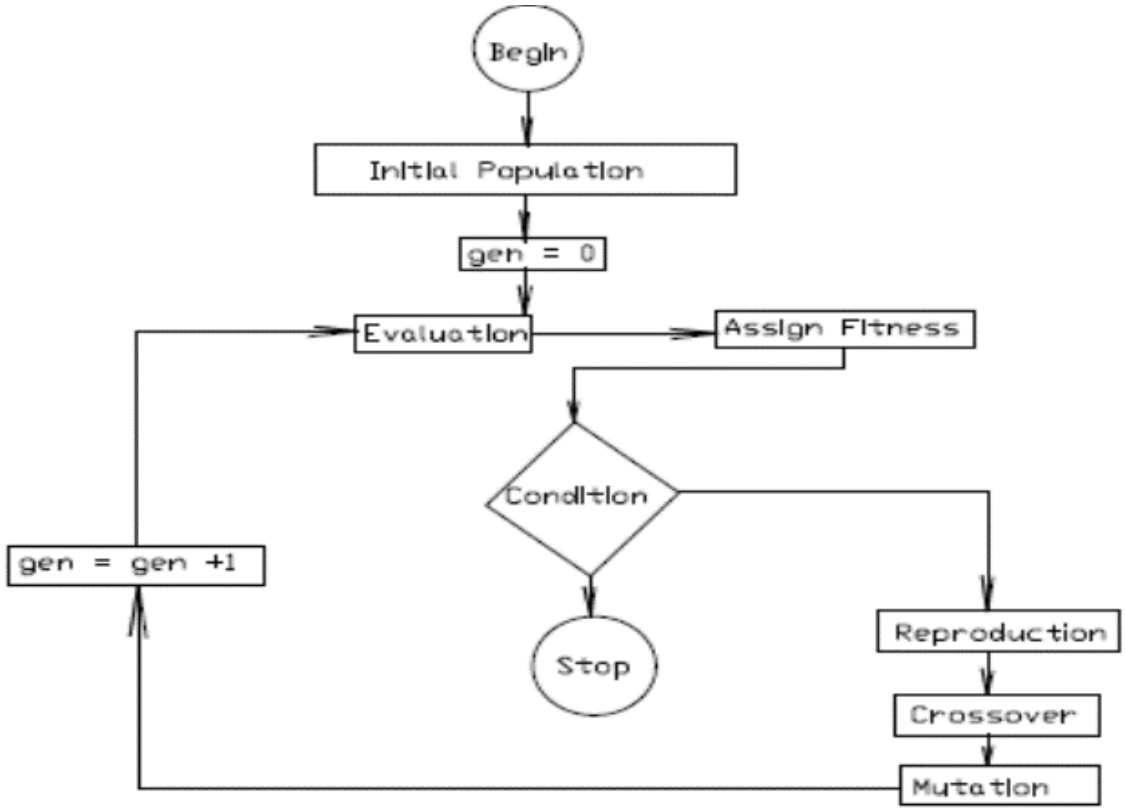
GA-base structural reliability method was first developed by Shao and Murotsu (1999). It consists of generating search directions in the space formed by the random variables affecting the reliability of a structural system. By following a search path until failure is reached, the reliability index, β, corresponding to the search direction is obtained. Genetic search algorithm simulates the biological process of evolution in an attempt to identify the parameters that is genes that define an optimum system.

Genetic algorithm is an iterative procedure that consists of a constant-size population of individuals, each one represented by a finite string of symbols, known as the "genome", encoding a possible solution in a given problem space, comprises all possible solutions to the problem at hand. The standard genetic algorithm proceeds as follows (Black, 1996): an initial population of individuals is generated at random or heuristically. Every evolutionary step, known as generation, the individuals in the current population are decoded and evaluated according to some predefined quality criterion, referred to as fitness. The fundamental principle for building an artificial system that reproduces and mimics the working of evolution date back to the biologist Barricelli who formulated an article about artificial methods to realize evolutionary processes (Baricelli, 1957).

The issue of efficiency of GA, which is of concern in all types of applications, is the subject of continuous research (Goldberg and Samtani, 1986; Goldberg, 1989; Grigoriu and Turkstra, 1979). Deng *et al* (2005) proposed a method for improving the efficiency of genetic algorithm during the search for the minimum reliability index of a structural system. This is based on a splitting technique that shreds a chromosome into many small fragments, which can be easily identified. A dominant structural failure mode normally occurs when a set of structural members fail. Also, the failure of subsets of those members would lead to the occurrence of 'partial failure' can be identified, and then some of these are very likely to contribute to the dominant failure mode. Conversely, a possible solution that does not contain any important 'partial failure' is unlikely to be the solution that would lead to a dominant failure mode. Linkage shredding genetic algorithm for reliability of structural system was proposed by Wang and Ghosn (2005).

The genetic algorithm (GA) that differs from other classical optimization in four ways (Goldberg, 1989) is a part of evolutionary computational technique and probabilistic and global search method. Due to these advantages, the GA has been preferred in wide ranges of optimization problems. In order to apply the genetic algorithm, a population of solutions within a search space is initialized on the contrary of the traditional optimization methods that starts from a single point solution. The population can be viewed as points in the search space of all solutions to the optimization problem. Each individual in population has a fitness value defined by a fitness function. Then the artificial evolution processes called the genetic loop which mimic natural evolution are applied to produce new candidate solutions. At the end of the process, the newly created generation replaces previous generation and revolution is repeated until a satisfying solution to the problem is obtained ensuring certain design criteria are satisfied or a maximum number of generations are reached.

2.6.2 **Development of Flow Chart for Genetic Algorithm (GA) - Base FORM Algorithm**



2.3 Background of Probabilistic Calculation

The method of probabilistic calculation in most cases is restricted to the comparison of two quantities, the resistance or strength R, and the load or action S.

The reliability function can be written as:

$$Z = R - S \tag{2.42}$$

The probability of failure is giving by :

$$P\{Z < 0\} = \iint_{r<s} F_R(r) F_S(s) dr ds \tag{2.43}$$

Equation (2.40) is the summation integration of all probability of combination for which load is greater than strength is valid.

Integrating (2.39) with respect to R:

$$P\{Z < 0\} = \int_0^\infty \{F_R(r)ds\}F_S ds = \int_0^\infty F_R(s) f_S(s) ds \quad (2.44)$$

Similarly by integrating with respect to S

$$P\{Z < 0\} = \int_0^\infty \{1 - F_S(r)\}F_R(r) dr \quad (2.45)$$

The relation to be chosen depends on the type of problem considered in many cases R and S are functions of other (basic i.e not correlated) variables in such cases the reliability function can be expressed in general form by

$$Z=Z(X_1,X_2,\ldots\ldots\ldots\ldots\ldots X_n) \quad (2.46)$$

For this general case the equivalent of equation (2.43) is

$$P\{Z < 0\} = \iint_{z(x)<o} \ldots . \int \prod_{i=1}^{n} fxi(xi)dxi \quad (2.47)$$

Here the variables X_i are supposed to be independent.

Equation (2.44) shows the determination of failure probability by a level iii approach, and as shown for independent basic variables it comes down to the calculation of an n-fold integral for n number of stochastic variables, but when the value of n exceeds 5 or 6, solving becomes possible using Monte Carlo simulation especially when the reliability function is relatively simple and the probability of failure is not too small.

In this research the reliability analysis is done using FORM (level II) approach**.**

2.8 Reinforced Concrete Materials

Concrete is one of the most popular materials for buildings because it has high compressive strength, flexibility in its form and it is widely available. The history of

concrete usage dates back for over a thousand years. Contemporary cement concrete has been used since the early nineteenth century with the development of Portland cement. Despite the high compressive strength, concrete has limited tensile strength, only about ten percent of its compressive strength and zero strength after cracks develop. In the late nineteenth century, reinforcing materials, such as iron or steel rods, began to be used to increase the tensile strength of concrete. Today steel bars are used as common reinforcing material. Usually steel bars have over 100 times the tensile strength of concrete; but the cost is higher than concrete. Therefore, it is most economical that concrete resists compression and steel provides tensile strength. Also it is essential that concrete and steel deform together and deformed reinforcing bars are being used to increase the capacity to resist bond stresses (Kang-kyu, 2002).

Concrete structures exposed to marine environments have been found to suffer mainly from corrosion of the reinforcing steel. It is necessary for engineers to understand the importance and mechanisms of chloride induced corrosion of steel in concrete. The effect of the quality of concrete on the resistance to chloride penetration, chloride threshold and corrosion rate of the reinforcing steel are critical in determining the service life of structures in this environment. The concept of service life prediction for concrete structures is becoming an area of increasing interest for engineers. Therefore, a good understanding of the concept and its application in structural design is essential to ensure that optimum concrete performance is achieved before a costly repair to the structure becomes necessary.

When designing concrete for durability performance, attention must be paid to both the performance standards required as well as selecting a set of compatible compliance criteria. This is to ensure that the hardened concrete “performs” in an environment for

which it is expected to meet the service life requirement. The specifications should focus on tests to be carried out on in-situ hardened concrete. The test results should become the basis for accepting or rejecting the concrete at site. Improved performance-based specifications are being developed. However, suitable short-term "performance" tests necessary for such specifications are not yet available. With the availability of mineral admixtures in many parts of the world, it is now possible to tailor-make improved quality concrete for most applications. A good knowledge of blended cement technology is necessary to ensure its full potential in being utilized for making durable concrete (Ferreira, 2008 and Lim Char Ching, 2008).

2.9 Corrosion of Reinforced Steel in Concrete

In general, concrete provides reinforcing steel with a non-aggressive environment.

During cement hydration the aqueous phase rapidly acquires a high pH value of (-13) that is buffered to resist downward change in pH values but the sparing soluble hydration product of cement (Glass and Brumfield, 2000). The steel reinforcement in contact with this alkaline solution is normally passive. The thermodynamically most stable corrosion products are insoluble oxides, and a passive oxide film covers surface, presenting is essential. Two processes can break down the passive environment in carbon dioxide (CO_2) in the atmosphere reacts with water in the concrete pores to form carbonic acid, which then neutralizes the alkalis in the pores. This can then lead to the corrosion of the reinforcing steel. In contrast, the introduction of chloride, such as sodium chloride (NaCl) found in the seaand de-icing salt or calcium chloride (CaCl) still found in concrete admixtures can results in chloride attack and depassivation of the steel reinforcement (Darek, 2012). Carbonation is more common in older structures, particularly older RC building and

reconstituted stone elements containing reinforcement that often have low cement content and are very porous. Conversely, it is very relatively rare in modern highway bridge and other civil engineering structures where awter-cement ratio are low, cement content are high, and proper compaction curing and cover are provided to prevent the carbonation front from advancing into the concrete to the depth of the steel within the lifetime of the structure (Broomfield, 2007). In RC structures in coaster regions, exposed to sea water, or RC structures in colder climates where the seasonal use of de-icing salts is standard practice, the chloride usually penetrate to the reinforcement and cause corrosion long before carbonation becomes problem (Page, 2007).

Once the reinforcing steel is depassivated, corrosion proceeds in electrochemical cells formed on the surface of the metal and an electrolyte of solution surrounding the metal as illustrated in Figure 2.4, each cell consists of a pair of electrodes on the surface of the metal (the anode and its counterpoint, the cathode), a return circuit, and the electrolyte. In the reforced concrete, the anode and the cathode are located on the reinforcing steel bars, which also serve as the return circuits, while the surrounding concrete acts as the electrolyte (Alsuleiman *et al*, 1990). Concrete exposed to wet-dry cycles has sufficient conductivity to serve as the electrolyte. At the anode location on the metal, the iron undergoes oxidation, which is accompanied by production of electrons and subsequently iron dissolution (Ahmad, 2003). In other words, the anode is the location on reinforceing bar where corrosion is taking place, metal is being consumed, and iron atoms lose electrons becoming iron ions (Fe^{+2}). The donated electrons move through a return circuit, which is a part in th metal itself to reach a cathodic spot on the metal, where these electrons are consumed through reactions involving substances found in the electrolyte. At

the cathode, oxygen, in the presence of water, accepts these electrons to form hydroxyl ions (OH^-). Both the anodic and cathodic reactions are essential for the corrosion process to occurand theey must take place concurrently.

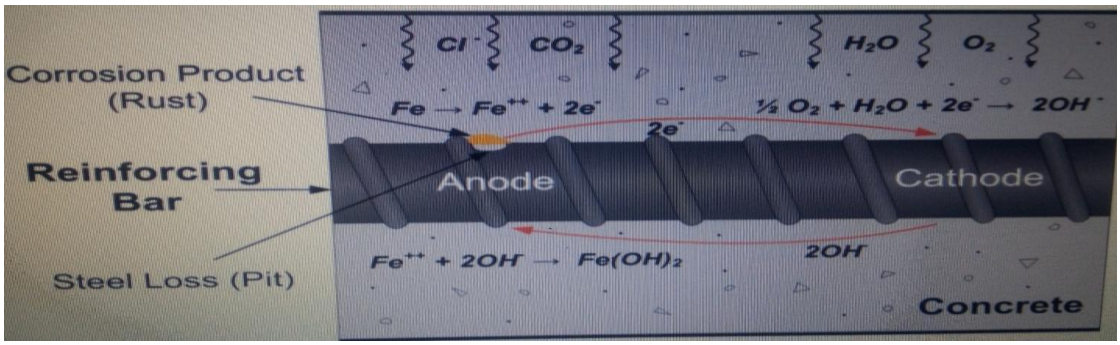
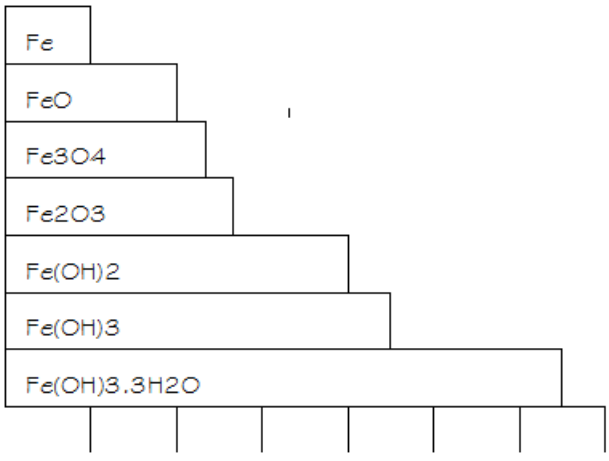


Fig 2.4: Schematic illustration of the corrosion of reinforcement steel in concrete as an electrochemical process (adapted from Ahmad 2003)

If he iron were to dissolve in the pore water (ferrous ion Fe^{2+} is soluble), there would be no cracking and spalling of the concrete. However, unhydrated ferric oxide Fe_2O_3 has a volume of about twice that of the steel it replaces when fully dense. When it becomes hydrated, it dilates even more, which implies that the volume increase at the steel-concrete interface is even larger. The various corrosion products resulting from the corrosion of steel reinforcing bars occupy a volume that varies from two to six times that of the original steel (Lin and Weyer, 1998) as illustrated by figure 2.5



Volume

Fig 2.5: Relative volumes of iron and its corrosion products (adapted from Liu and Weyers 1998a)

Expansion of the corrosion products result in very high pressure stresses in the concrete surrounding the corroding steel, causing cracking, delamination, and spalling of the concrete cover (see Fig 2.6), which inturn further accelerates the corrosion process by providing an easy pathway for the water and chloride to reach the steel. If laft unattended, this process can ultimately lead to loss of serviceability, strength and safety of the affected structure (ASTM, 1991).

The initiation of corrosion in an RC column is likely to occur at the stirrup reinforcement surfaces, which has the minimum concrete cover. Then the overall corrosion mechanism of the tensile steel becomes more complex with portion of longitudinal bars acting as anodes and other as cathodes. If the anode and cathode are well separated (by several hundred millimeters) and the anode is starve of oxygen, for instance by being underwater, the iron as Fe^{2+} will stay in solution (Broomfield, 2007), as a result there will be no expansive forces to crack the concrete, thus the corrosion remain undetected.

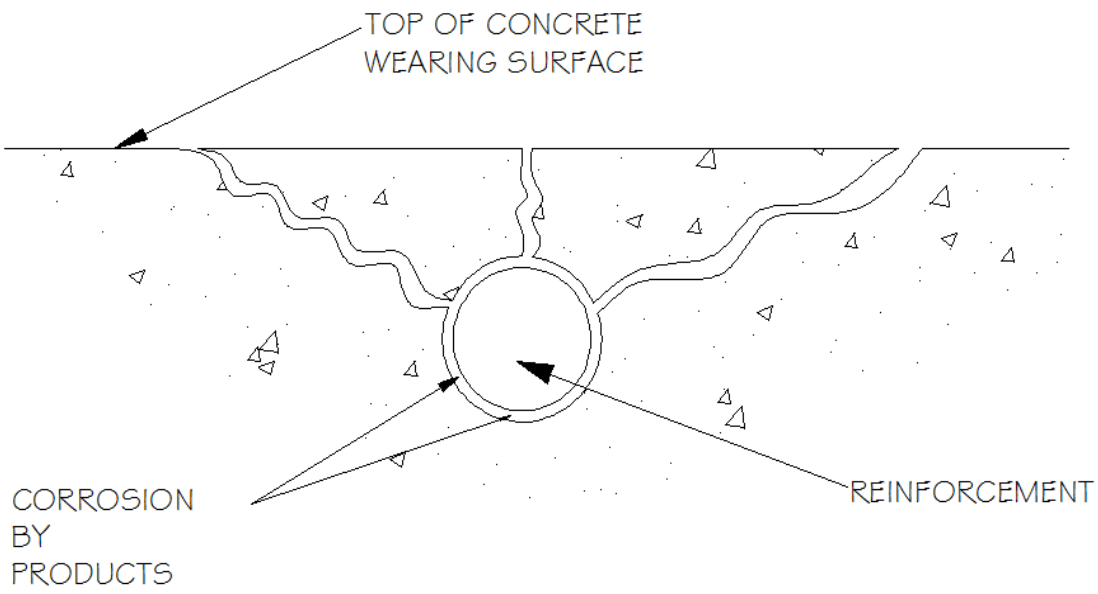


Fig 2.6: Corrosion induced cracking and spalling of the concretes cover (Reproduced from CAC, 1995).

This type of corrosion, known as ‘’Black’’ rust due to the colour of the affected bar when first exposed to air after breakout, is often found under damaged waterproof mambranes and in underwater and other water saturated condition (Broomfield, 2007). It is potentially dangerous as there is no indication of corrosion by cracking and spalling of the concrete and the reinforcing steel may be severally weakened before corrosion is detected. The rebar may be rapidly hollowed out or even completely penetrated and severed. Although occasional rust staining on the concrete surface may be indicative of this type of attack, the presence of water under a membrane exclude oxygen, making it unlikely that the iron solution will reach the concrete surface where it will precipitate out to form rust stain.

2.9.1 Corrosion of Reinforcement

Corrosion of steel in concrete is a complex phenomenon, influenced by many internal and external factors. These include the pH of the concrete pore solution, temperature, internal stresses, stray currents and electrolytic potentials. Reinforcement cast into concrete is initially rendered passive by a protective layer of ferric oxide that forms under normal alkaline conditions (pH 12.5-13.5). The ferric oxide layer may be disrupted by the presence of sufficient chloride ions or by a reduction in pH of concrete below 10.5 due to carbonation or acidification. Once depassivation has occurred, corrosion of steel is possible provided oxygen and moisture are available .Chloride ions act as catalysts in the disruption of the passive ferric oxide layer and are recycled for use elsewhere along the steel surface. A minimum concentration of chlorides at the steel, known as the corrosion threshold level, is required to depassivate reinforcement under normal alkaline conditions. Once the steel is effectively depassivated, the corrosion rate and subsequent damage will depend on micro-effects such as availability of oxygen and moisture and macro-effects such as structural geometry, anode/cathode ratios and general ambient conditions. Corrosion of reinforcement in concrete structures may be divided into two separate phases; an initiation period during which little damage occurs as chlorides diffuse towards the reinforcement, and a propagation phase in which damage occurs progressively once the corrosion threshold had been exceeded and corrosion has been initiated. The insidious nature of steel corrosion makes effective repairs expensive if remedial work is only contemplated once significant damage occurs (Ferreira, 2008 and Mackechnie, 2001).

Corrosion of reinforcing steel in concrete leads to the premature failure of many structures exposed to harsh environments. Rust products form on the bar, expanding its volume and creating stress in the surrounding concrete. This leads to cracking and spalling, both of

which can severely reduce the service life and strength of a member. Corrosion of reinforcing steel in concrete structures is one of the most expensive problems facing civil engineers in the United States. The structural integrity of many bridges overpasses, parking garages, and other concrete structures has been impaired by corrosion, and repairs are urgently required to ensure public safety (Jones 1996). As structures approach the end of their design life, new and improved methods for repair and rehabilitation of corroded members must be developed. To prevent and arrest corrosion activity at an early stage, accurate detection techniques are necessary. Because of the increasing number of corrosion related problems in the field of structural design, research is continuously being conducted to evaluate and implement efficient repair strategies (Kabir, 2007).



Fig Concrete cover spalling off from corroded rebars (adapted from Kabir 2007}

CHAPTER THREE

3.0 ANALYTICAL TECHNIQUE

3.1 BS 8110 1999: Design Criteria Of Reinforced Concrete Columns

Columns are defined in BS8110:Part 1:1997, Clause 3.8.1.3 as either short or slender. Slender columns are subjected to moment due to the deflection of the columns, which must be added to those calculated for the loading and sway effects. The definition of short and slender is dependent on the l_{ex}/h and $l_{ey}/\ b$ ratios of the columns, where :

l_{ex} the effective height in respect of the major axis,

l_{ey} the effective height in respect of minor axis,

h the depth of the cross-section measured in the plane under consideration.

b the width of a column (dimension of a column perpendicular to h).

Reinforce concrete column design resistance of a short reinforced concrete column is given as:

$\alpha_u = \beta_a K h$... 3.1

in this expression β_a has the value obtained from Table 3.21 or alternatively from equation 3.1 from which the table is derived, where K is a reduction factor that correct the deflection to allow for the influence of axial load. K is derived from the following equation:

$K = \frac{N_{uz} - N}{N_{uz} - N_{bal}} \leq 1$... 3.2

where $N_{uz} = 0.45 f_{cu} A_c + 0.95 f_y A_{sc}$ (including allowances, as appropriate for γ_m).

The appropriate values of K may be found iteratively, taking an initial value of 1. Alternatively, it will always be conservative to assume that K =1.

Table 3.21 is derived from the following equation:

$$\beta_a = \frac{1}{2000}\left(\frac{l_e}{b_r}\right)^2 \qquad 3.3$$

3.2 The Structural Model of the Corroded Column

The reinforced column adopted in this study is based BS 8110 Design Criteria of reinforced concrete column. The dimensions of the RC column in the present study are arrived at based on such design guidelines which aim to achieve the prescribed minimum failure modes. The structural configuration of the columns is shown in fig 3.1. The column is subjected to imposedload of 1550KN, and dead to live load ratio ranging from 0.5-0.9, yield strength from 400-600N/mm^2 and concrete grade from 20-40N/mm^2.

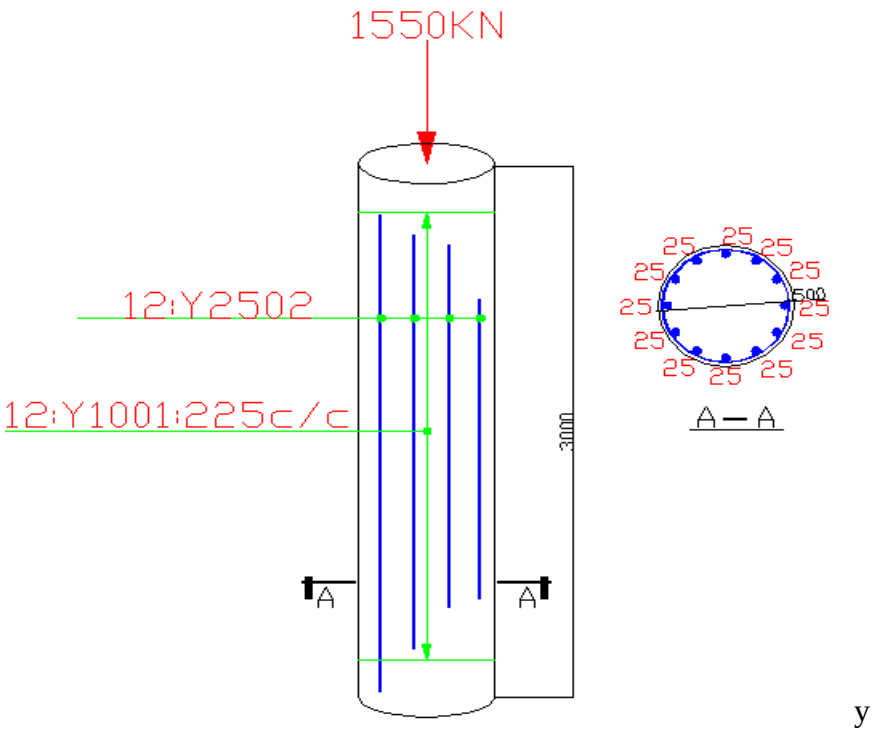


Fig. 3.1. Axially loaded column with section

3.3 Limit State functions for the reliability Index of Columns

The limit state function gives a discretized assessment of the state of structure element of a given failure mode. it is usually obtained from certain traditional deterministic analysis but uncertain parameters are treated statistically.

The limited state g(x0 = R-S is a function of material properties , load and dimension as well. The region of integral of the functions are stated below

G(x) > 0 : signifies safe

G(x) = 0 : signifies limit state attainment

G(x) < 0 : signifies failure

Computation of reliability index

The expression for performance function G(X) in the limit state equation is given as:

$$G(x) = \left[\frac{0.35 \times f_c \times d_c^2}{4} + 0.67 f_y \times n\pi\left(\frac{d_{st} - \left(I_{corr}\left(T_{elapsed} - T_i\right)\right)}{4}\right)^2\right] - \left[q(\gamma_g \alpha + \gamma_q) \times 10^3\right]^4$$

Where $G(X)$ is the limit state function

$f_c = Concrete\ grade = 30N$

$d_c = column\ diameter = 500mm$

$d_{st} = Diameter\ of\ steel = 25mm$

$I_{corr} = Corrosion\ rate\ in\ mm/years$

$T_i = Corrosion\ initiation\ time$

$T_{elapsed} = Column\ age\ from\ corrosion$

$\gamma_q = Live\ load\ factor = 1.0$

$\gamma_g = Dead\ load\ factor = 1.0$

$\alpha = \frac{G_k}{Q_k} = load\ ratio$

Π = 3.143

N – 12 nos bars

3.4 Statistical Model of the Basic Design Variable of the Columns

In order to conduct reliability analysis, it is required to determine mean, standard deviation, coefficient of variation of the basic design variable by the use of laboratory data or data generated from relevant literature if available.

Table 1: Statistical Models of the Random Variable (JCSS Probability Model Code, 2000)

S/No	BASIC VARIABLES	DISTRIBUTION TYPE	EX (1)	COV	SX (1)
1	Live Load q (KN/m^2)	Gumbel	1550 KN	0.20	310
2	Area of Steel (As) (m^2)	Normal	varies	0.05	Varies
3	Concrete grade (fc) (N/mm^2)	Log-normal	30 N/mm^2	0.10	3
4	Diameter of Column (Dc) (mm)	Normal	12mm	0.05	0.6
5	Steel strength	Log-normal	500N/mm2	0.17	85

	(N/mm^2)				

3.5 Computer Analysis Procedure

A First Order Reliability Analysis was carried out using MATLAB program on a RC Column and the safety indices under corrosion failure modes were carried out in accordance with the Eurocode 2 specification by varying the following parameters;

- Load ratio was varied from 0.5 to 0.9.
- Age of Column was varied from 0 to 30 years.
- Corrosion rate was varied from 0.05mm/year to 0.40mm/year.
- Diameter of columns 500mm.
- Imposed load 1550 KN.
- Steel strength varied from 400 to 600 N/mm^2.
- Diameter of steel 25mm.
- Concrete grade varied from 20 to 40N/mm^2
- Number of reinforcement 12 bars.

3.6 Development of the MATLAB Program

MATLAB program language was used to develop the program for the reliability analysis. The program flowchart of the MATLAB based program used in this research is as follow. The program starts by calling on the user to enter a target value of safety index, β_T, and then reads same. After that, it reads the other parameters which are also supplied by the user. FORM is subsequently called to calculate the reliability index for the data provided aided by GA.

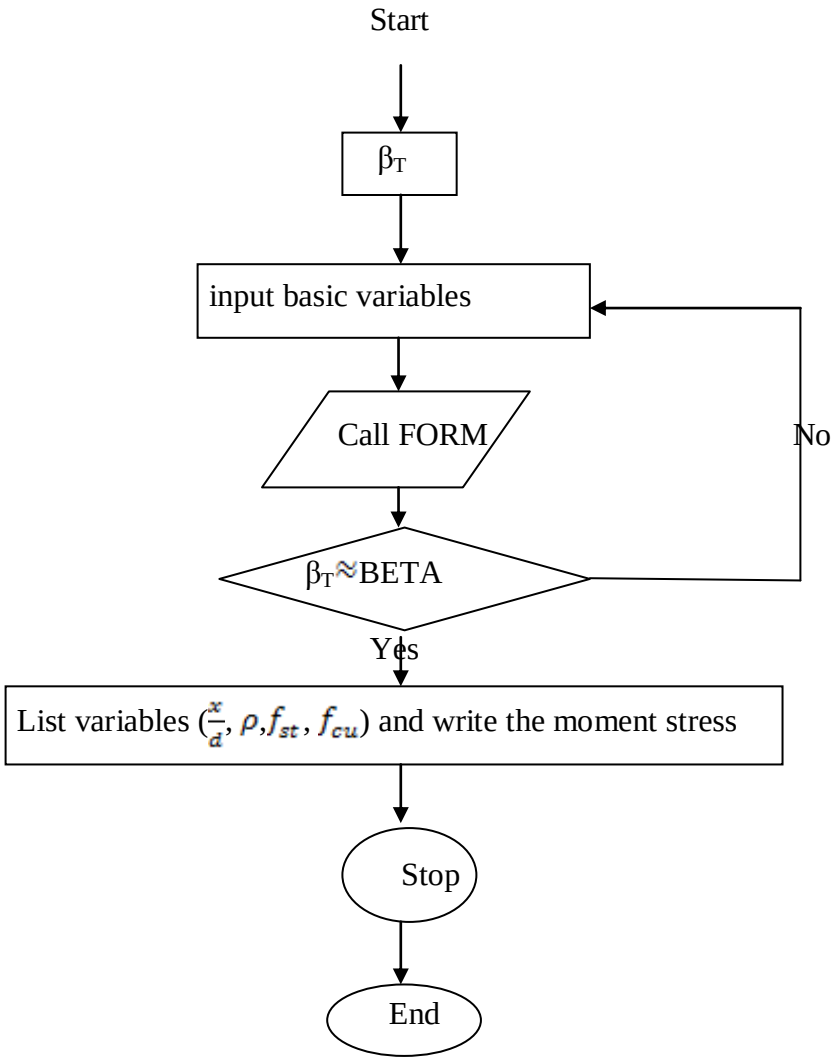


Fig. 3.2Program Flowchart

3.6.1 Main Directory

The main directory contains the main program. This program was developed to perform the reliability analysis in this study. While running this program, the user would be required to select among five options of failure modes, each failure mode with its inputand

the capability of execution a specific function. The detail of this section program is given in Appendix A.

3.6.2 Form directory

The form directory contains the subroutines that calculate the safety index for each failure mode.

3.6.3 Distribution Model Setup Directory

The 'distribution model setup directory' contains the MATLAB function files that assign appropriate distribution model to each random variable. Only five distribution models were considered in this study. These are, the normal distribution, lognormal distribution, gumbel distribution, weibull distribution and frechet distribution

3.6.4 Distribution transformation directory

The distribution model transformation directory contains the function files that transform gumbel and lognormal variables to equivalent normally distributed random variables.

3.6.5 Coefficient of variation directory

The coefficient of variation directory containing the function file that store the values of the coefficients of variation of each random varible based of test result and data from internation literature.

3.6.6 Probability of failure directory

Probability of failure directory contains the MATLAB function that computes probability of failure with safety index as an input.

3.7 **Hasofer-Lind Reliability Index** (1974)

A modified reliability index: it did not exhibit the invariance problem. The "correction" is to evaluate the limit state function at a point known as the "design point" instead of the mean values. The design point is generally not known a priori, an iteration technique must be used (in general). Consider a limit state function g(X_1,X_2,.....,X_n) where the random variables Xi are all uncorrelated. The limit state function is rewritten in terms of the standard form of the variables using

$$Z_{i=}\frac{X_i-\mu_{xi}}{\sigma_{xi}} \qquad (3.4)$$

As before, the Hasofer-Lind Reliability Index is defined as the shortest distance from the origin of the reduced variable space to the limit state function g= 0,

An iteration is required to find the design point $\{Z_1^*, Z_2^*,.....,Z_n^*\}$in reduced variable space such that β still corresponds to the shortest distance (shown in Figure)

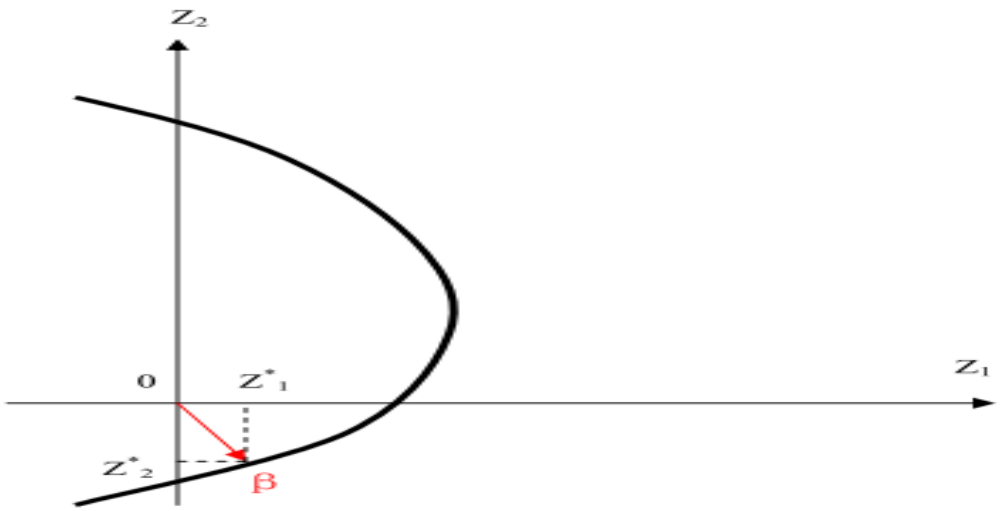


Fig 3.2: Reduced and reliability index

The iterative procedure requires us to solve a set of (2n+1) simultaneous equations with (2n+1) unknowns:

$$\beta, \alpha_1, \alpha_2, \ldots, \alpha_n, Z_1^*, Z_2^*, \ldots Z_n^* \tag{3.5}$$

$$\alpha_1 = \frac{-\frac{\partial g}{\partial z_i}|evaluated\ at\ design\ point}{\sqrt{\sum_{k=1}^{n}(\frac{\partial g}{\partial z_k})^2}|evaluated\ at\ design\ point} \tag{3.6}$$

$$\frac{\partial g}{\partial z_i} = \frac{\partial g}{\partial x_i}\frac{\partial x_i}{\partial z_i} = \frac{\partial g}{\partial x_i}\sigma_{x_i} \quad . \tag{3.7}$$

$$\sum_{i=1}^{n}(\alpha_i)^2 = 1 \tag{3.8}$$

$$Z_i^* = \beta\alpha_i \tag{3.9}$$

$$g(Z_1^*, Z_2^*, \ldots Z_n^*) = 0 \tag{3.10}$$

β, α_1, α_2,....., α_n, Z_1^*, Z_2^*,....., Z_n^*

The simultaneous equation procedure:

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

ii. Express the limit state function in terms of reduced variable Z_i.

iii. Use equation (d) to express the limit state function in terms of β and α_i.

iv. Calculate the n α_i values. Use Eq. (d) here also to express each α_i as a function of all α_i and β (Note the coupling effect between α_i).

v. Contact the initial cycle: Assume numerical values of β and all α_i, noting that the α_i values must satisfied Eq,(c)

vi. Use the numerical value of β and α_i on the right hand sides of the equations formed in steps 3 and 4 above.

vii. Solve the n+1 simultaneous equations in step 6 for β and α_i.

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

3.8 Setup of Reliability Analysis by Genetic Algorithm

Considering the independent standard normal U-space where failure is defined by a number of failure modes, g_1,.g_2, , , , ,g_k. These modes are reached by following the failure paths in direction $\boldsymbol{I}_i$. In the genetic algorithm (GA) according to Shao and Murotsu (1999), $3^n - 1$ discrete search directions are used in n-dimensional space. The search direction vectors are expressed by a three-digit system where each of the digits belong to the binary set (-1,0,1) as illustrated in Fig 3.10.

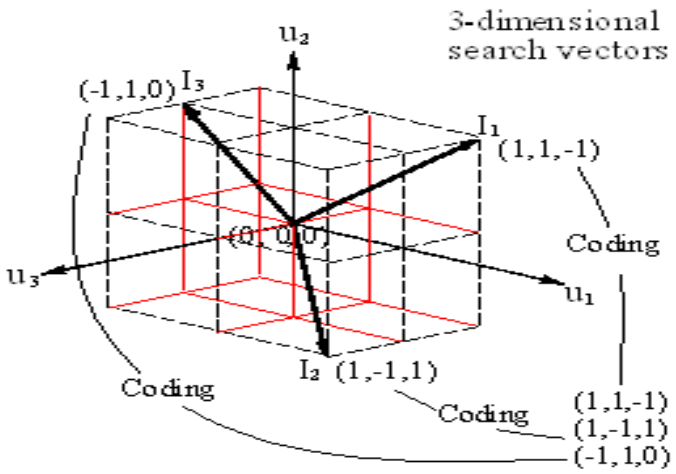


Fig. 3.3: Coding of search direction vectors

The rectangular coordinates of a direction vector constitute a string of digits known as chromosomes in GA parlance. GA simulates the natual selection process based on the

survival of the fittest (in this case, which direction should survive and which direction should be killed off). In the GA-based reliability analysis, a fitness is defined as the inverse of the reliability index, β, where β is the distance from the orogin of the U-space to the failure surface as shown in Fig. 3.11.

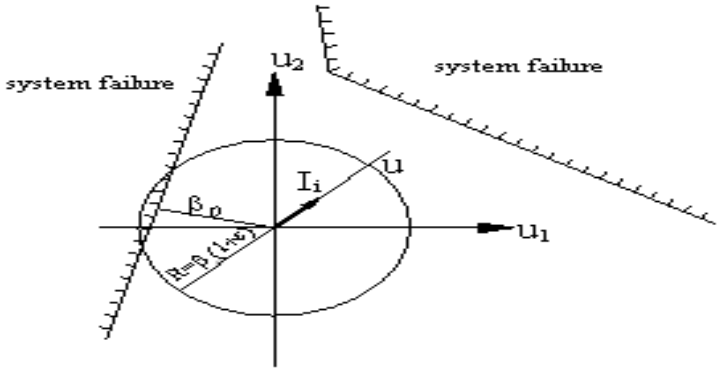


Fig. 3.4 Illustration of GA search strategy

To initiate the search process, GA randomly generates a set of chromosomes to form a first generation of directions. The search is refined by creating a "new generation´of chromosomes using three operators: Selection, Crossover and Mutation. The objective is to improve the fitness of the population of chromosomes with each generation.

Selection simulates the evolution process that allows the chromosomes with best fitness´to survive and mate. The chromosomes in a new generation are the offsprings (or children) of selected chromosomes of the previous generation. The selection is done randomely such that chromosomes with high fitness have the higher probabilities to contribute offsprings in a new generation.

Crossover simulates the natural mating phenomenom whereby offsprings in a new generation are obtained by crossing the chromosomes of two "parents" directions. Each offspring will keep some of the genes from each of its two parents as shown in Fig. 3.12

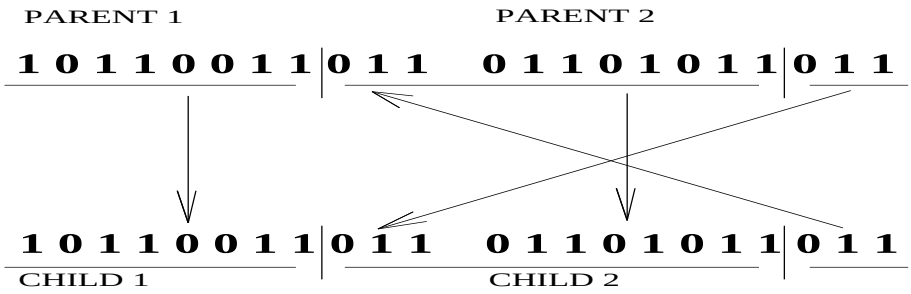


Fig. 3.5 Illustration of crossover operation

Mutation is secondary operator that simulate an inheritance that goes astray. It is an explorative operator that allows the search to wander into regions of the solution space which may never have been reached from the crossover operation. The purpose of the mutation operator is to reduce "in-breeding" that may misdirect the search process. This, new "genetic material" is occationally introduced into the new generation by randomly changing digits in each chromosome. For example , a 1 is randomly chamged to 0, or vice versa. This is done only with a very low probability to generate new genetic material without loosing track of the important directions that the search has produced so far. Considering a typical chromosome that is formed by a total of n genes (or variables). For example (1 0 1 1 0 0 1 1 1 0 …….) is one representation of such a chromosome. The m genes associated with 1 in the sequence are called active variables. In this example, the active variables are located in positions 1,3,4,7,8,9,……. The other genes are not active.

The reliabity index associated with the failure modes described by the chromosome is β. An approximation to the probability of failure P_f can be obtained by assuming that the safety marging M (or failure function) is linear in the space defined by the n variables (X_1, X_2, . . .,X_n). This linear equation can be expressed as:

$$M = \frac{1}{\sqrt{m}}(X_1 + X_3 + X_4 + X_7 + X_8 + X_9 + \ldots) + \beta \quad (3.11)$$

Only the active variables appear in eqn. (3.97), because the non-active variables are known constants (deterministic) parameters that do not affect the safety index.For a given search direction, on the U-space, the failure point (design poibnt) is designated by **A** and its coordinates are expressed as :

$$O\mathbf{A} = \beta(k_1\mathbf{x}_1, k_2\mathbf{x}_2, \ldots\ldots\ldots, k_n\mathbf{x}_n) \quad (3.12)$$

Where $k_1, k_2, \ldots\ldots, k_n$ are **OA**'s unit vector coordinates in direction $x_1, x_2 \ldots x_n$ suck that:

$$(k_1^2 + k_2^2 + \ldots\ldots + k_n^2)^{1/2} = 1 \quad (3.13)$$

For the coding adopted in this work, as outlined in Fig. 4.8, the k's can be represented as:

$$(k_1, k_2, \ldots\ldots k_n) = (\boldsymbol{I}_1\boldsymbol{I}_2 \ldots\ldots \boldsymbol{I}_n)\frac{1}{\sqrt{m}} \quad (3.14)$$

Where $\boldsymbol{I}_i$ = 1 or 0. A value of 1 indicates an increase in the direction of variable x_i. 0 indicate that x_i remains at its mean value (origin). The index i = 1,2, ...n, identifies the variable. m gives the total number of non-zero terms in the set $(k_1, k_2, \ldots\ldots, k_n)$. This would lead to the alternate representation of OA as:

$$OA = \frac{\beta}{\sqrt{m}} = (\boldsymbol{I}_1 x_1 \boldsymbol{I}_2 x_2 \ldots\ldots \boldsymbol{I}_n x_n) \quad (3.15)$$

Each set of $\boldsymbol{I}_i$ values defines a chromosome that represents the direction along the n-dimensional space $x_1, x_2, \ldots\ldots x_n$. The n variables X_n in turn represents the sequence of random variables $X_1, X_2, \ldots\ldots X_n$ in tha order in the normalized U-space. These random

variables are denoted $l_1, l_2, \ldots\ldots, l_n$. An approximate number of chromosomes define the population of each generation. If too few chromosomes are generated, GA has only few possibility to execute the crossover operation. On the other hand, if there are too many chromosomes, GA slows down. Research has shown that increasing the population size beyond a certain limit will not help improve convergence (Deng 2005). Following the approach of Cornell (1996), it is suggested that an optimum population size would be on the order of:

$$N_{popsize} = \text{order}(r\frac{L}{2k}3^k) \tag{3.16}$$

Where $n_{popsize}$ is the optimum population size, L is the length of the chromosome, and k is the average number of bits per parameter. The value that each active variable takes when failure occur is given by:

$$L_{active} = \beta\frac{i}{\sqrt{m}} = \frac{(x_{active} - \mu_{active})}{\sigma_{active}} \tag{3.17}$$

Where m is the number of non-zero (active) genes, μ_{active} is the mean value of the active variable, σ_{active} is the standard deviation of the active variable, β is the safety index.

3.9 MATLAB PROGRAM

The MATLAB procedure used in this research for the calculation of the safety index requires the user to enter in the following;

1. Input dead load safety coefficient
2. Input live load safety coefficient
3. Input dead to live load ratio $\propto$

4. Input corrosion rate (values used range from 0.01mm/year to 0.2mm/year)
5. Input time in years after corrosion initiation (3-20 years)
6. Input imposed load on frame (KN/m^2)
7. Select steel yield strength identifier
8. Input diameter of reinforcing steel in mm
9. Input number of reinforcing steel
10. Input compressive strength of the concrete in N/mm^2
11. Input depth of column in mm
12. Input width of column in mm
13. Input number of population generation
14. Input crossover probability (>0 and < =1)
15. Input mutation probability (>0 and < =1)

The program then computes the reliability index β. The results obtained are tabulated in the appendix (tables) and results discussed in the next chapter.

CHAPTER FOUR

4.0 Discussion of Results

The variation of seven (7) selected vital parameters varying from corrosion rate, age of corrosion, steel yield strength, load ratio, depth, diameter of bolts and imposed load whose strength was obtained from Eurocode 2 Design criteria was carried out using the MATLAB program 'PROGRAM kamal.m'(Appendix A) and the reliability indices was tabulated (appendix B) and then represented in Figs.

4.1.1 Variation of Corrosion rates on the Safety Index with respect to Load ratio on RC Columns

The variation of corrosion rate from 0.02-0.2 years, for a age range of 5-50 years whose strength class of concrete and steel as C30 and S500 respectively were adopted from Eurocode 2, 2008 was carried out using MATLAB program column_corrosion.m' (see appendix) and the reliability indices for the failure mode obtained were tabulated (see appendix) and thus represented in the graphs shown below.

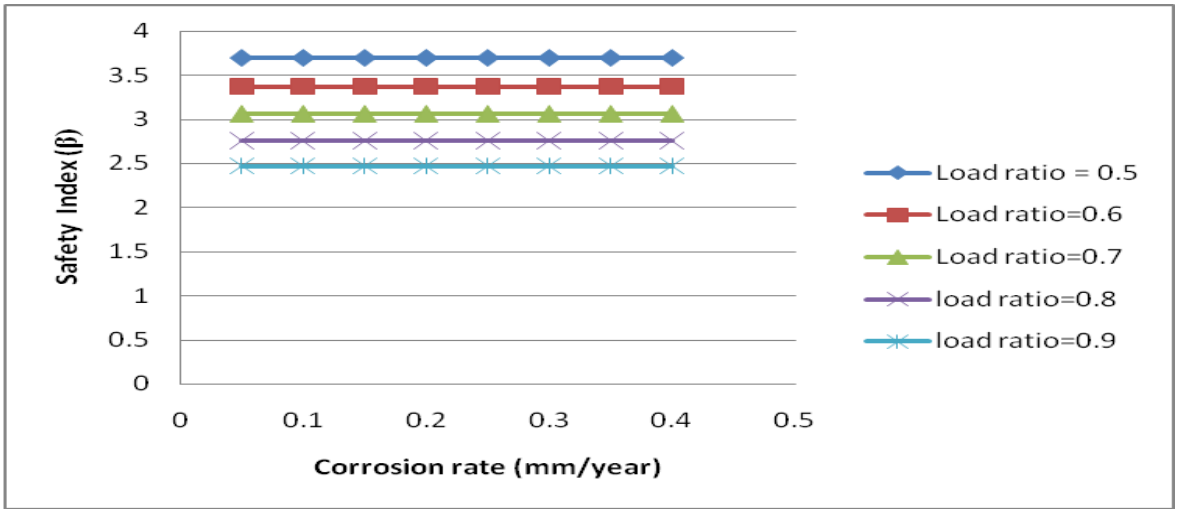


Figure 4.1: The relationship between the safety index and corrosion rates for load ratio in zero year.

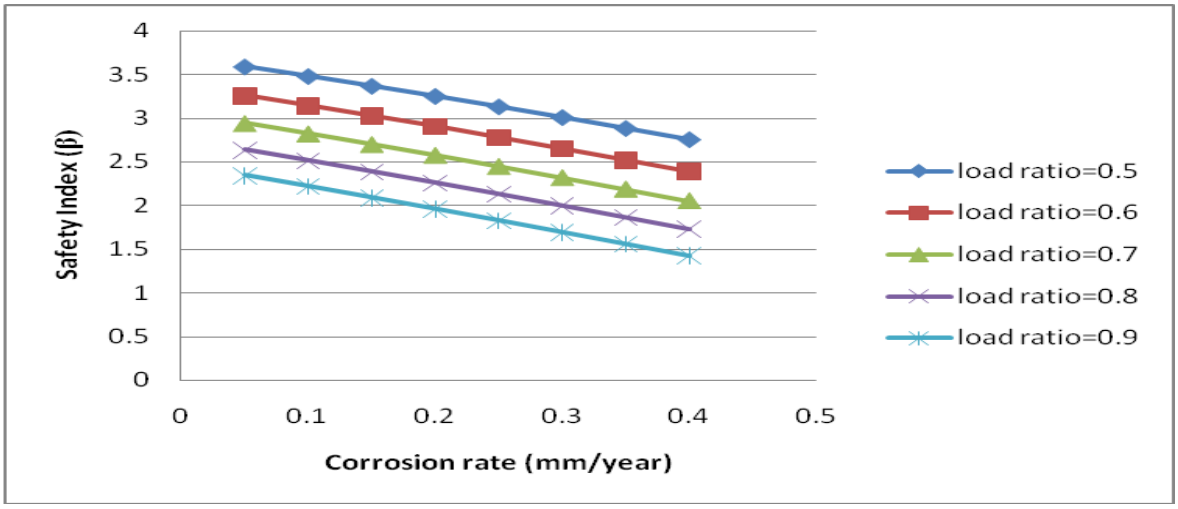
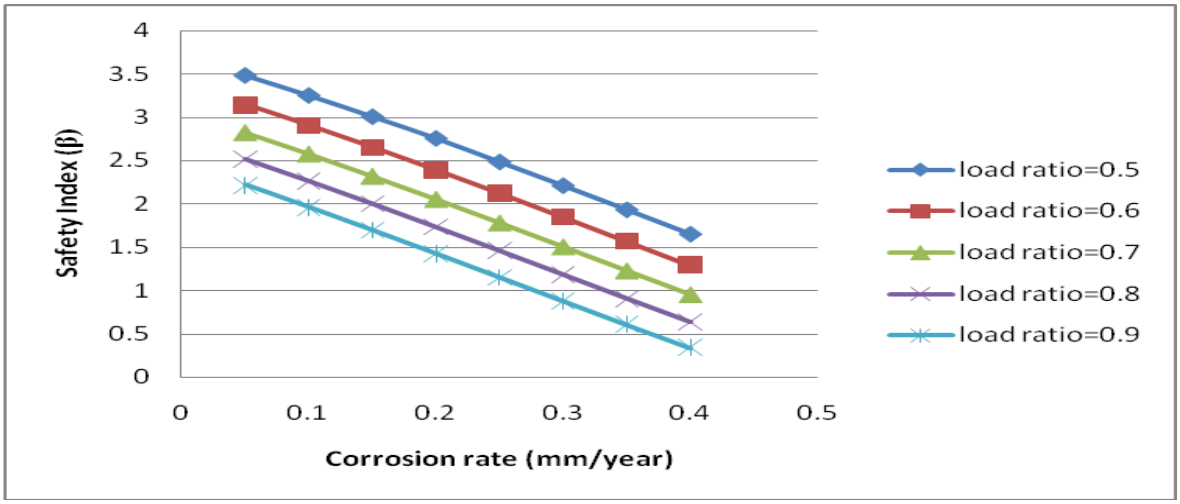


Figure 4.2: The relationship between the safety index and corrosion rates for load ratio in five (5) years



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Figure 4.3: The relationship between the safety index and corrosion rates for load ratio in ten (10) years

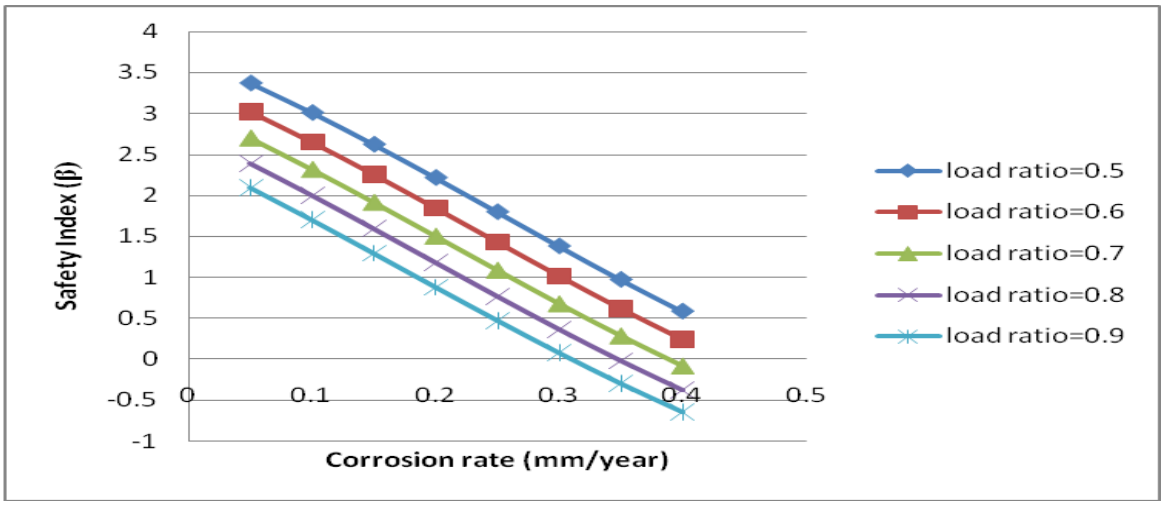


Figure 4.4: The relationship between the safety index and corrosion rates for load ratio in Fifteen (15) years.

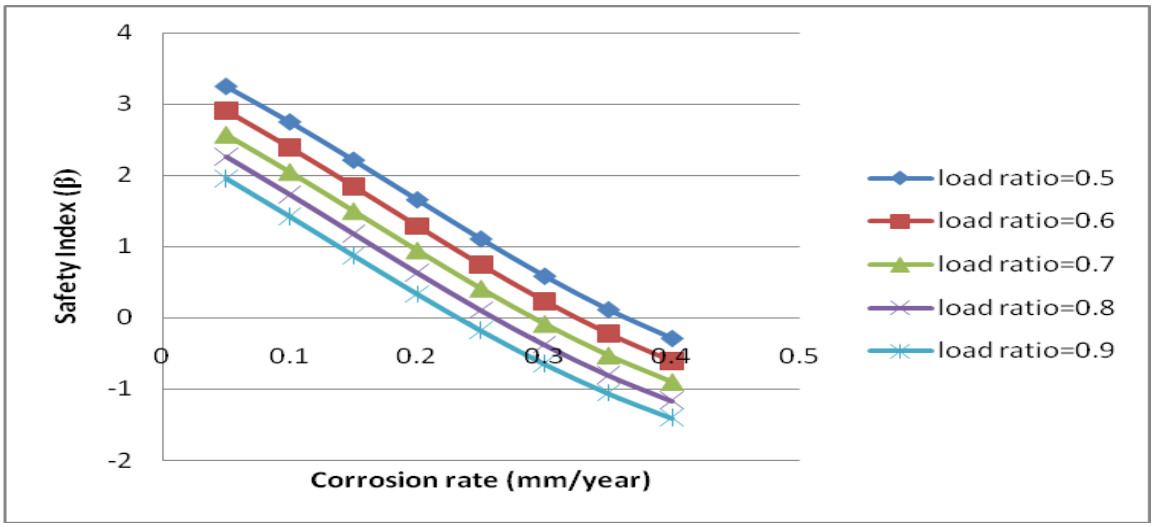


Figure 4.5: The relationship between the safety index and corrosion rates for load ratio in Twenty (20) years

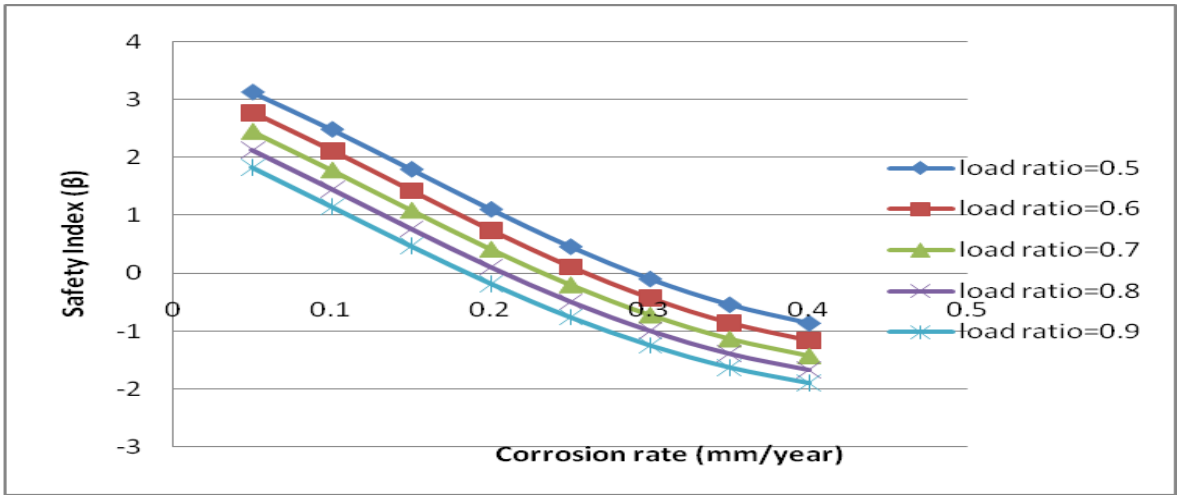


Figure 4.6: The relationship between the safety index and corrosion rates for load ratio in Twenty-five (25) years.

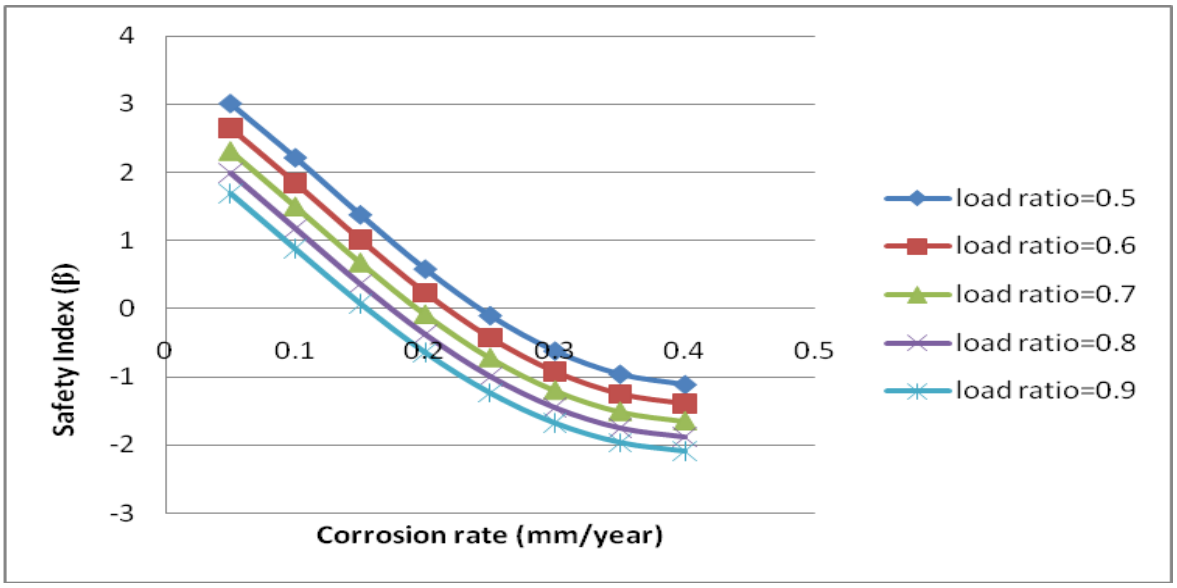


Figure 4.7: The relationship between the safety index and corrosion rates for load ratio in Thirty (30) years.

4.1.2 **Variation of Corrosion rates on the Safety Index with respect to Steel strenght on RC Columns**

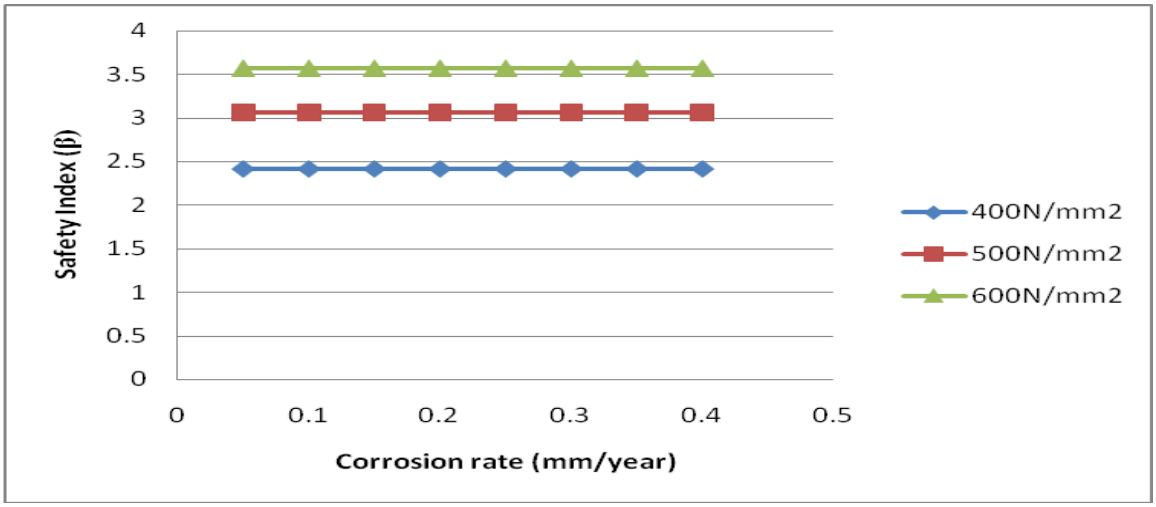


Figure 4.8: The relationship between the safety index and corrosion rates for Yield strenght in Zero (0) years.

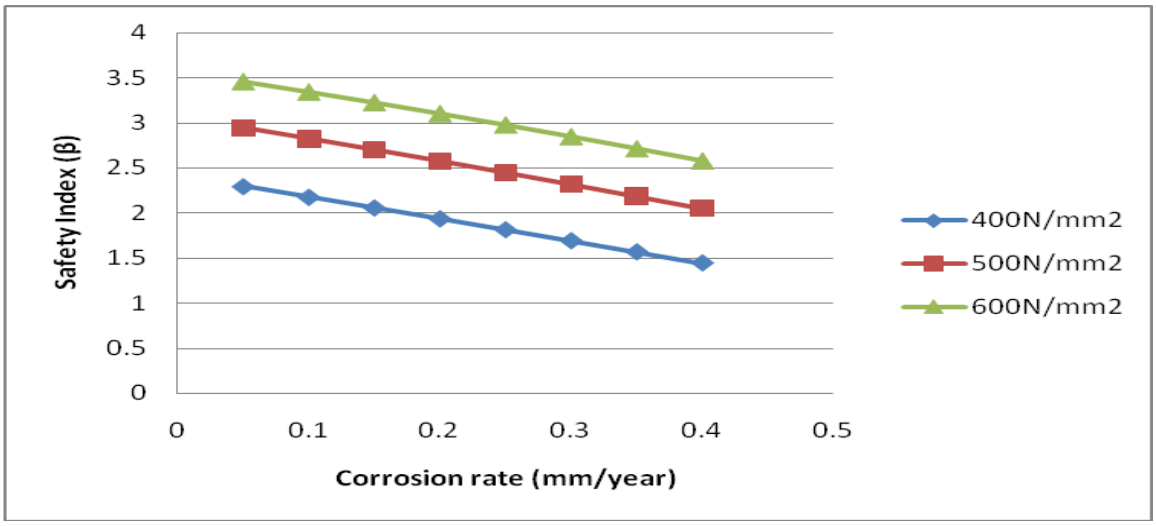


Figure 4.9: The relationship between the safety index and corrosion rates for Yield strenght in five (5) years.

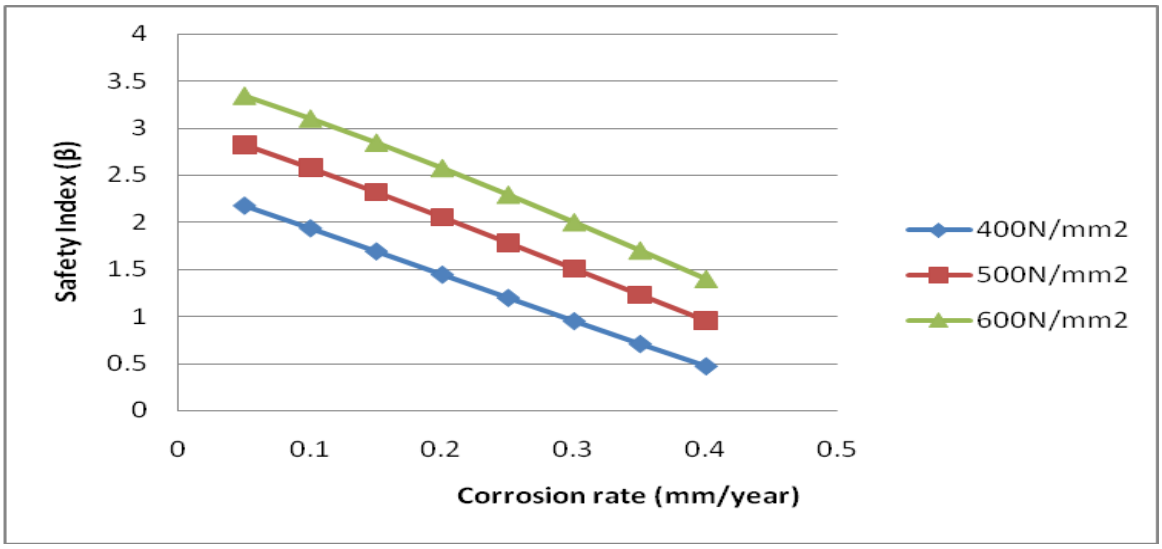


Figure 4.10: The relationship between the safety index and corrosion rates for Yield strenght in Ten (10) years.

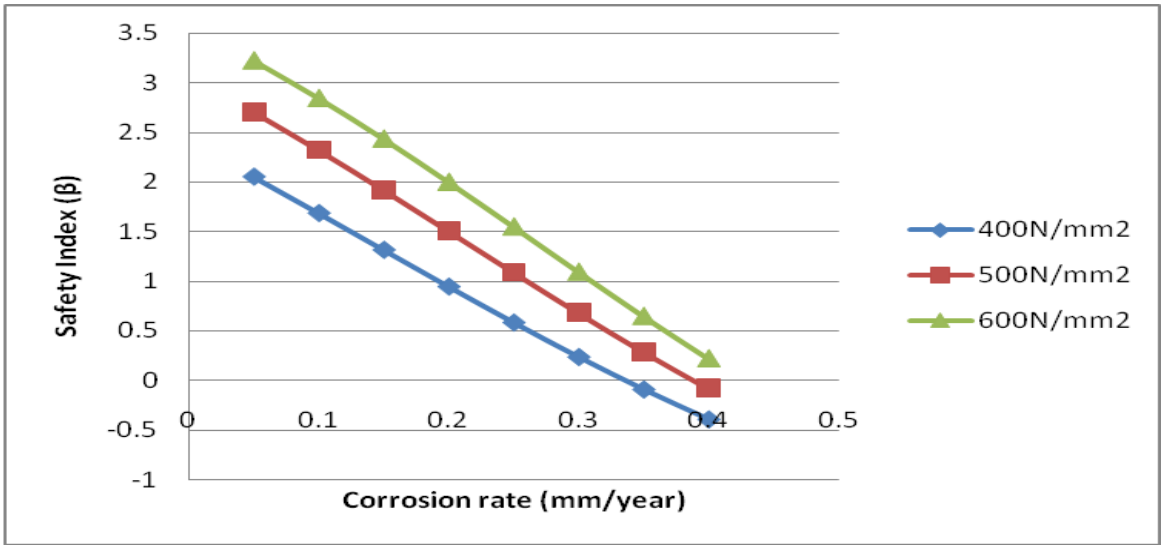


Figure 4.11: The relationship between the safety index and corrosion rates for Yield strenght in Fifteen (15) years.

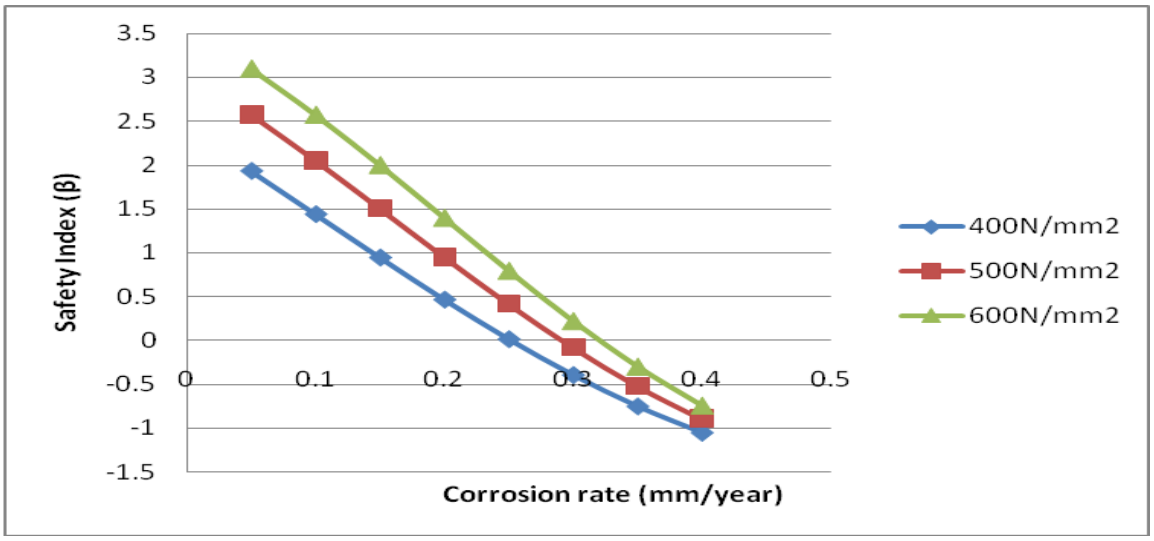


Figure 4.12: The relationship between the safety index and corrosion rates for Yield strenght in Twenty (20) years.

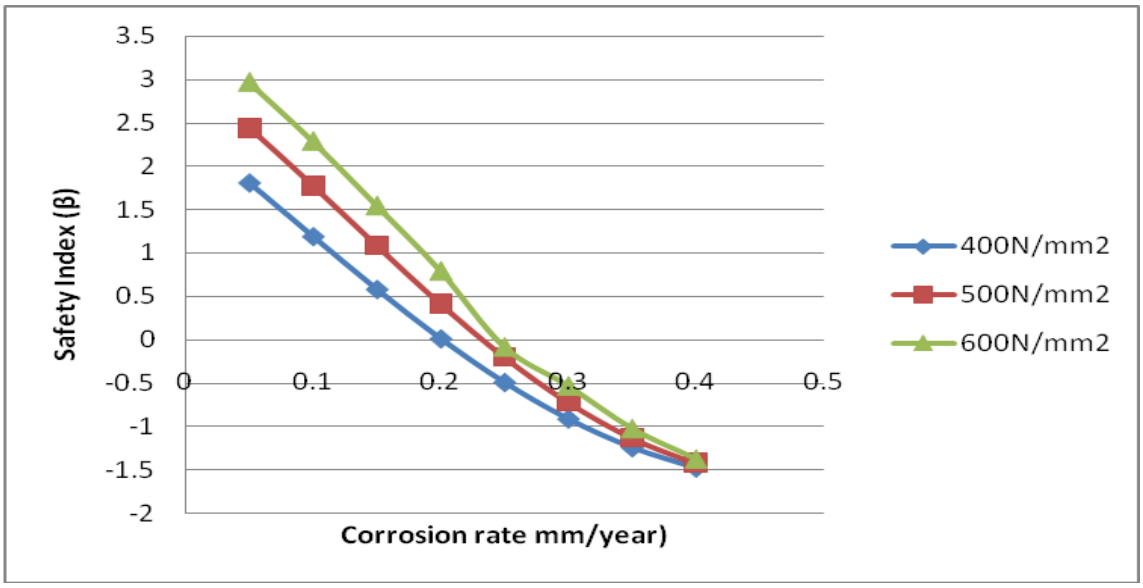


Figure 4.13: The relationship between the safety index and corrosion rates for Yield strenght in Twenty-five (25) years.

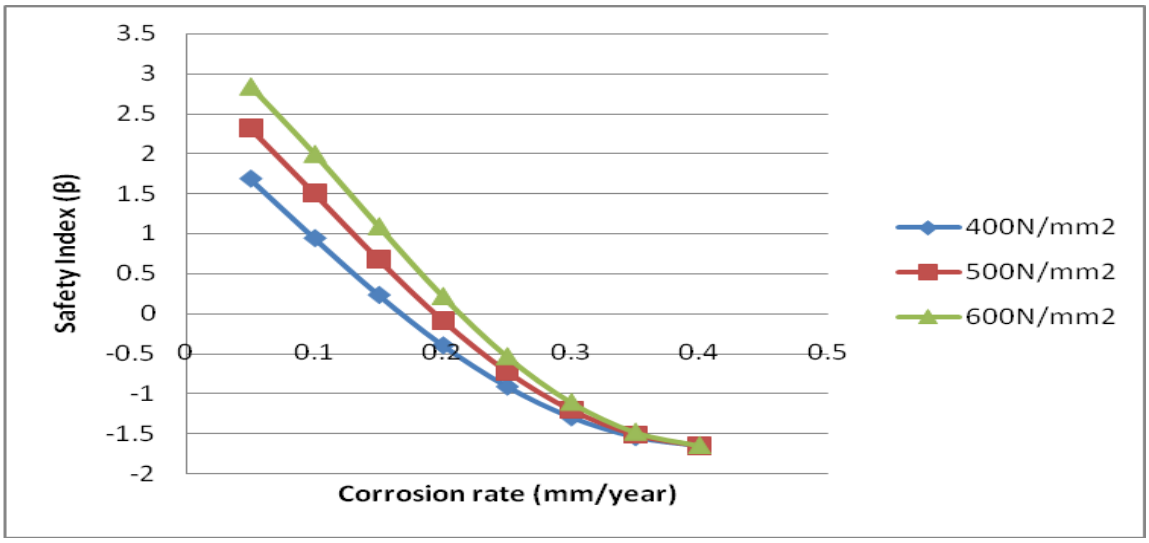


Figure 4.14: The relationship between the safety index and corrosion rates for Yield strenght in Thirty (30) years.

4.1.3 **Variation of Corrosion rates on the Safety Index with respect to Concrete Grade on RC Columns**

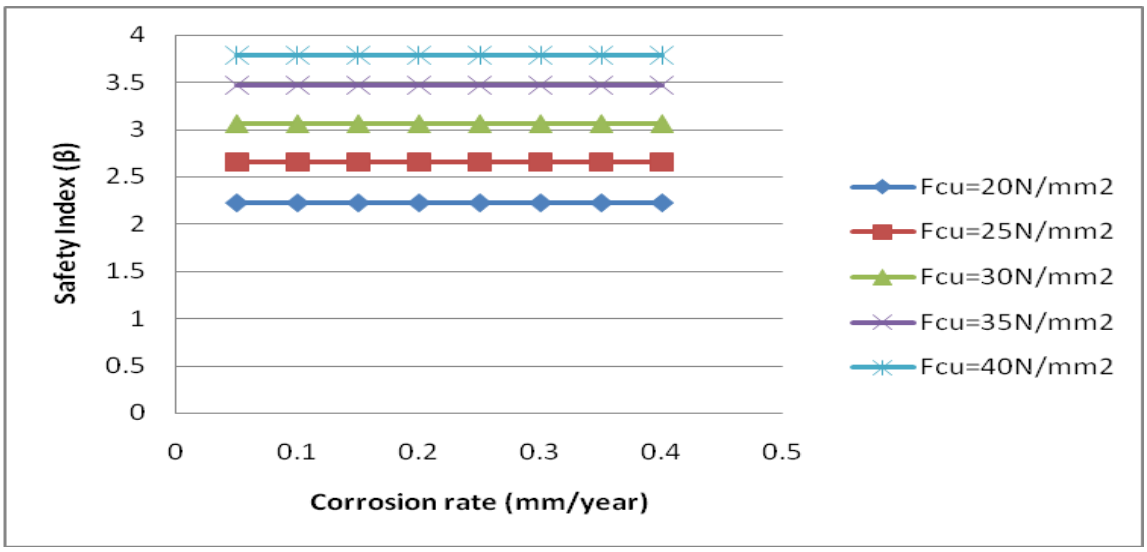


Figure 4.15: The relationship between the safety index and corrosion rates for concrete grade in Zero (0) years.

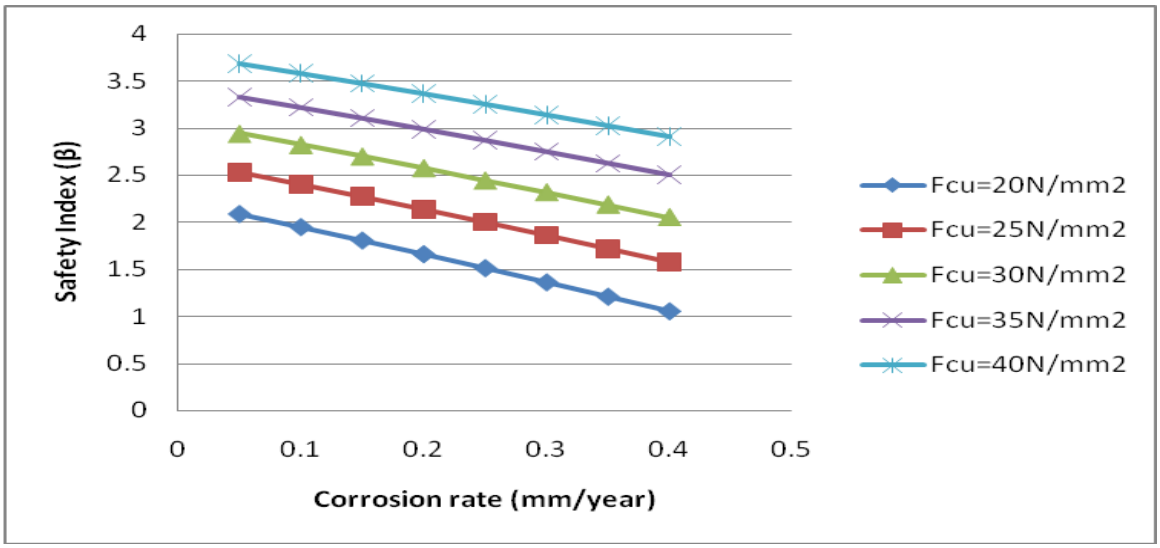


Figure 4.16: The relationship between the safety index and corrosion rates for concrete grade in Five (5) years.

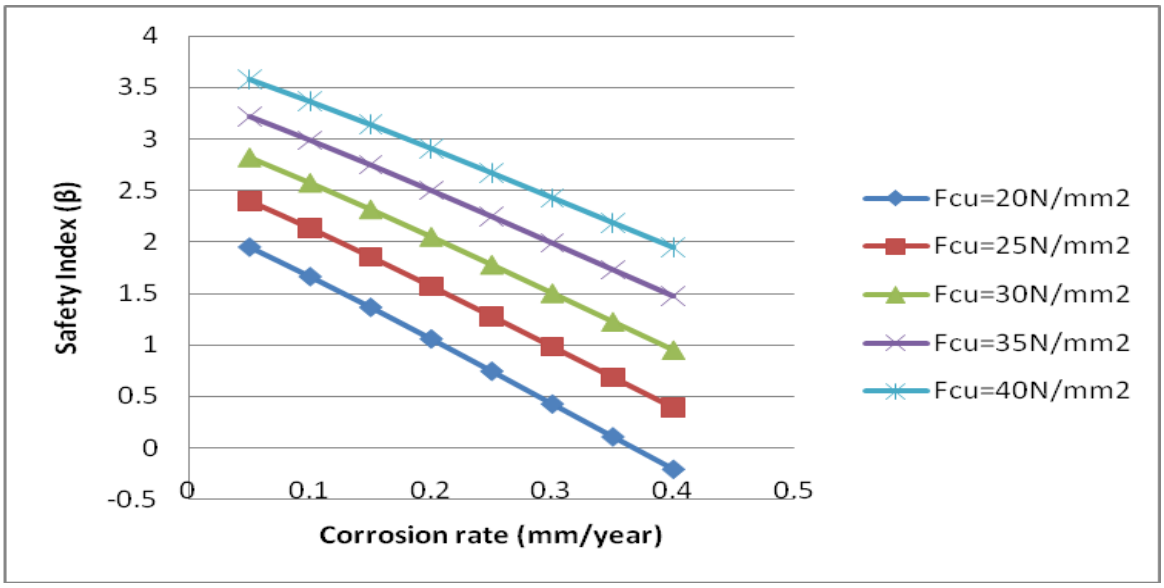


Figure 4.17: The relationship between the safety index and corrosion rates for concrete grade in Ten (10) years.

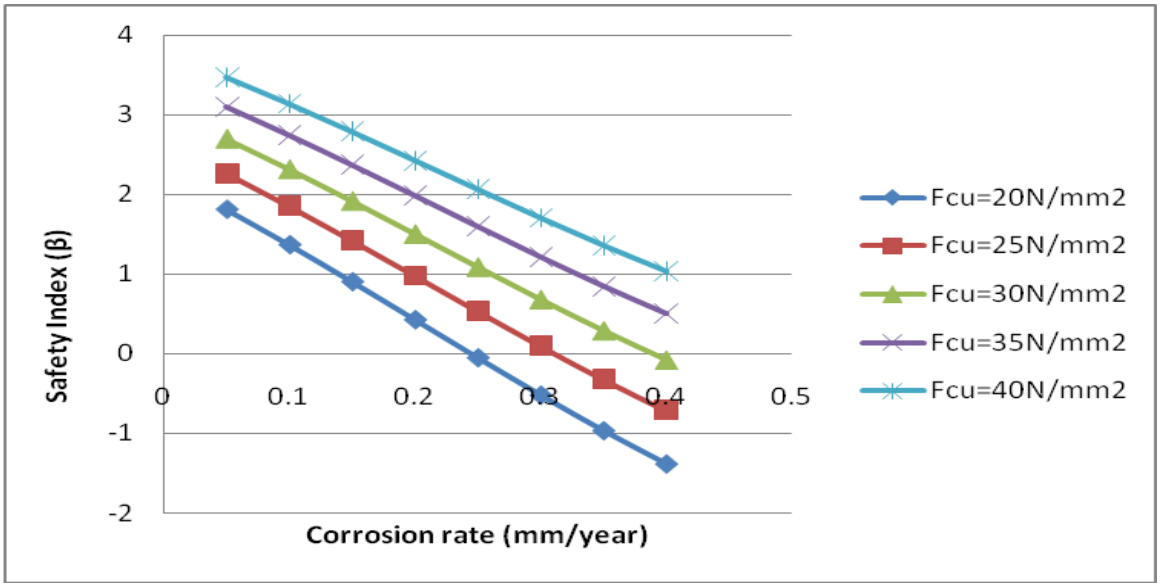


Figure 4.18: The relationship between the safety index and corrosion rates for concrete grade in Fifteen (15) years.

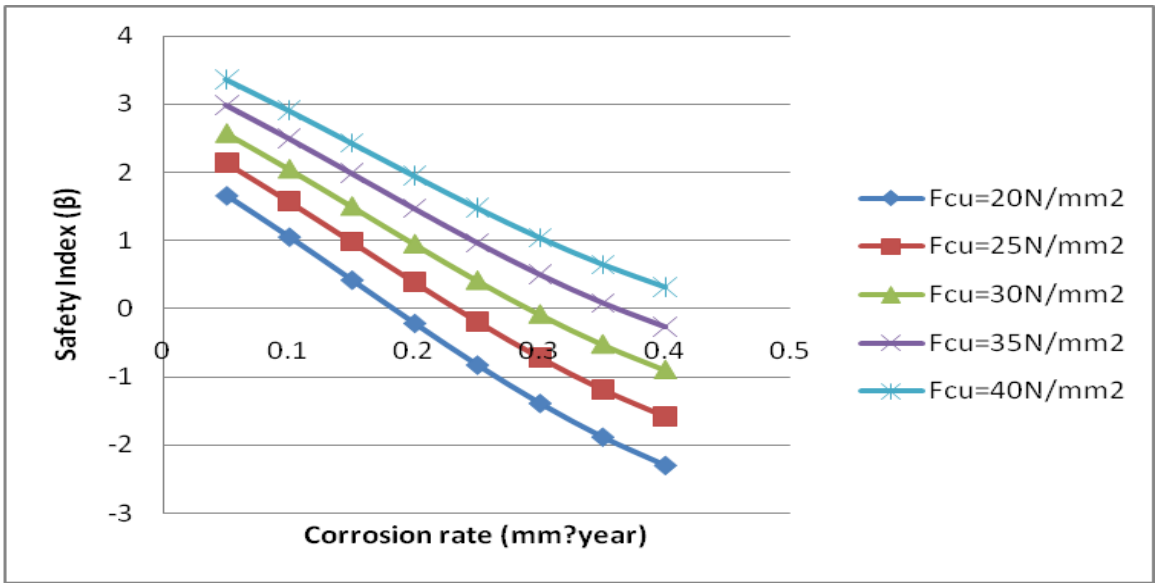


Figure 4.19: The relationship between the safety index and corrosion rates for concrete grade in Twenty (20) years.

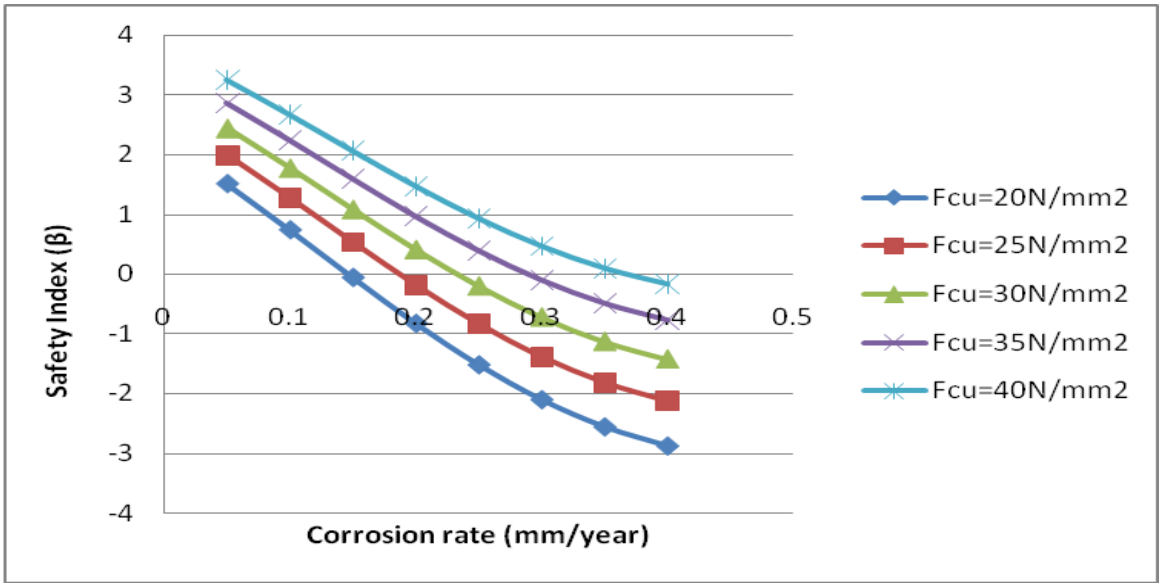


Figure 4.20: The relationship between the safety index and corrosion rates for concrete grade in Twenty-five (25) years.

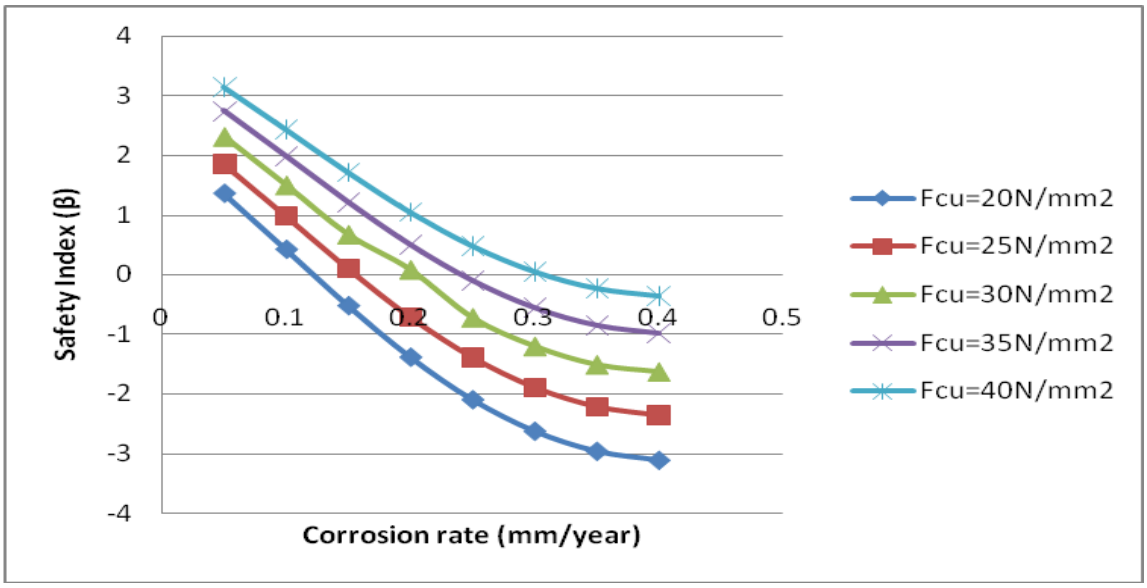


Figure 4.21: The relationship between the safety index and corrosion rates for concrete grade in Thirty (30) years.

4.2. **Discussion of Results**

The program developed for the probabilistic analysis of reinforced concrete columns subjected to corrosion using MATLAB and the procedure was repeated for different variation of factors and parameters.

The year of the corrosion was varied from 0 year to 30 years, load ratio from 0.5 to 0.9, yield strength (Fy) from 400N/mm2 to 600N/mm2 and concrete grade (Fcu) from 20N/mm2 to 40N/mm2.

Figures 4.1 to 4.7 show the relationship of safety indices to corrosion rate for load ratio parameters, Figures 4.8 to 4.14 show the relationship of safety indices to corrosion rate for yield strength parameters and figures 4.15 to 4.21 show the relationship of safety indices

to corrosion rate for concrete grade parameters of reinforced concrete columns subjected to corrosion.

Figure 4.1, 4.8 and 4.15, show the reliability of based result/relationship when corrosion is at o years and the corrosion rate is varied from 0.05mm/yr to 0.4mm/yr. the result and safety indices are taken at interval of 0.05mm/yr.

Careful observation from Figure 4.1, 4.8 and 4.15 shows that the safety indices remain constant while an increase in corrosion rate and load ratio. The result generally indicates that there is no failures occur within the first couple of years of the corrosion initiation. This stage depends on so many factor and statistic show that it varies in the range of 0-5years. After the first couple of years, the rate of probability of failure increase at a nearly constant value.

When the year of corrosion rate is increase from 0-10years, with the variation in load ratio from 0.5 to 0.9 and yield strength from 400N/mm2 to 600N/mm2 and concrete grade from 20N/mm2 to 40N/mm2, the safety indices obtained were 1.66, 1.29, 0.95, 0.63, 0.34 for load ratio, 0.47, 0.95, 1.4 for yield strength and 0.21, 0.39, 0.95, 1.47, 1.95 for concrete grade respectively. This signifies the safe state and early age of the columns.

When the corrosion rate increases to 0.4mm/years with the variation of load ratio from 0.5, 0.6, 0.7, 0.8, 0.9, and yield strength from 400N/mm2, 500N/mm2, 600N/mm2 and concrete grade from 20, 25, 30 ,35, 40 N/mm2, the safety indices obtained were 0.58, 0.23,-0.08,-0.38,-0.65 for load ratio, -0.39,-0.08, 0.22 for yield strength and -1.39, -0.72,-0.08, 0.50, 1.04 for concrete grade respectively. This signified the serviceability limit state

of the columns with significant impact of corrosion damage with age of 15 years and constant imposed load of 1550KN.

With corrosion rate of 0.4mm/years with the same variation of load ratio, yield strength and concrete grade, the safety indices obtained were 0.58, 0.23,-0.08,-0.38,-0.65 for load ratio, -1.65, -1.65, -1.65 for yield strength, -3.12, -2.37, -1.63, -0.98, -0.37 for concrete grade respectively. This signified the ultimate limit state of the columns which requires complete rehabilitation at age of 30 years

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

From probability analysis of reinforced concrete columns exposed to corrosion was carried out was carried out using MATLAB program kamal.m

* From the results obtained using MATLAB, the safety index of the reinforced concrete column decreases as the load ratio of the column is increasing. The reliability indices for the column decreased as the corrosion rate increases at various load ratio with time.

* The graph show that an increase in yield strength and concrete grade leads to corresponding increase in the safety indices. This show at higher values of yield strength and concrete grade, the reinforced concrete column subjected to corrosion is safe.

* At the age of 15 years, the columns experience serviceability limit state due to the safety indices obtained;0.58, 0.23,-0.08,-0.38,-0.65 for load ratio, -0.39,-0.08, 0.22 for yield strength and -1.39, -0.72,-0.08, 0.50, 1.04 for concrete grade respectively as corrosion rate increase to 0.4mm/year.

* At the age of 30 years, it has reached ultimate limit state due to the safety indices obtained; 0.58, 0.23,-0.08,-0.38,-0.65 for load ratio, -1.65, -1.65, -1.65 for yield strength, -3.12, -2.37, -1.63, -0.98, -0.37 for concrete grade respectively and need total rehabilitation.

* The variation is more dependent on member geometry, particularly member slenderness. Since increase in corrosion results to reduced cross-section, then increase in corrosion rate leads to decrease in buckling capacity of the columns.

5.2 Recommendations

To ensure the safety and durability of reinforced concrete structure exposed to marine environment, safe lives and property, the following are recommendations from this study:

- For reinforced concrete columns and other aggressive environment prone to corrosion attack, a minimum of 0.5 load ratio , 500 N/mm2 yield strength and 30N/mm2 of concrete grade at corrosion rate of 0.4mm/year with good anti-corrosion properties (epoxy resins) be used in the construction of columns as load carrying components
- Repair should be made when the reinforced concrete columns reached 15 years in service.

- The quality control of concrete mi x should be adequate to increase durability and reduce fire effect under heat.

- A higher steel grade and concrete grade is better than a lower steel grade or other reinforcements.

- Reliability based method should be integrated in the BS 8110 design criteria of Reinforced Concrete Columns subjected to corrosion in order to fully accommodated the uncertainties in design parameters, since the partial safety concept is deterministic in nature, derived from the long- ti me experience of building tradition.

- Data from laboratory tests is needed to perform additional component and model uncertainty analyses.

- The thesis can be used as a guide for reinforced concrete structure (columns) exposed to marine environment and reference for further academic research.

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APPENDIX: PROGRAM LISTING

```
%%%
```

```
%
%     DR. J. M. Kaura
%
%
%
%%%
```

```
      As = ( n * pi * (dia – 2 * ( Icorr * (Telapsed - Ti) )  ) ^ 2 ) ) / 4.0;
end
%
disp('
%%%
```

APPENDIX B: RESULTS GENERATED

T_i=3,T_e= 0, F_y =500, F_{CU}=30					
Corrosion rate	Load ratio				
	0.5	0.6	0.7	0.8	0.9
	β	β	β	β	β
0.05	3.6971	3.3720	3.0588	2.7578	2.4690
0.10	3.6971	3.3720	3.0588	2.7578	2.4690
0.15	3.6971	3.3720	3/0588	2.7578	2.4690
0.20	3.6971	3.3720	3.0588	2.7578	2.4690
0.25	3.6971	3.3720	3.0588	2.7578	2.4690
0.30	3.6971	3.3720	3.0588	2.7578	2.4690
0.35	3.6971	3.3720	3.0588	2.7578	2.4690
0.40	3.6971	3.3720	3.0588	2.7578	2.4690
T_i=3,T_e= 5, F_y =500, F_{CU}=30					
Corrosion rate	Load ratio				
	0.5	0.6	0.7	0.8	0.9
	β	β	β	β	β
0.05	3.5906	3.2600	2.9423	2.6376	2.3459
0.10	3.4806	3.1448	2.8229	2.5148	2.2203
0.15	3.3670	3.0264	2.7005	2.3893	2.0925
0.20	3.2500	2.9094	2.5754	2.2614	1.9626

0.25	3.1298	2.7804	2.4477	2.1313	1.8309
0.30	3.0064	2.6532	2.3177	1.9993	1.6976
0.35	2.8801	2.5235	2.1855	1.8655	1.5629
0.40	2.7511	2.3915	2.0515	1.7303	1.4272

T_i=3,T_e= 10, F_y =500, F_{CU}=30					
Corrosion rate	Load ratio				
	0.5	0.6	0.7	0.8	0.9
	β	β	β	β	β
0.05	3.4806	3.1448	2.8229	2.5148	2.2203
0.10	3.2500	2.9049	2.5754	2.2614	1.9626
0.15	3.0064	2.6532	2.3177	1.9993	1.6976
0.20	2;7571	2.3915	2.0515	1.7303	1.4272
0.25	2.4858	2.1218	1.7790	1.4566	1.1536
0.30	2.2130	1.8463	1.5027	1.1807	0.8792
0.35	1.9354	1.5680	1.2251	0.9052	0.6067
0.40	1.6558	1.2897	0.9492	0.6328	0.3384
T_i=3,T_e= 15, F_y =500, F_{CU}=30					
Corrosion rate	Load ratio				
	0.5	0.6	0.7	0.8	0.9
	β	β	β	β	β
0.05	3.3670	3.0264	2.7005	2.3893	2.0923
0.10	3.0664	2.6532	2.3177	1.9993	1.6976
0.15	2.6194	2.2575	1.9159	1.5939	1.2906
0.20	2.2130	1.8463	1.5027	1.1807	0.8792
0.25	1.7956	1.4286	1.0868	0.7684	0.4718

0.30	1.3775	1.0143	0.6778	0.3660	0.0769
0.35	0.9694	0.6133	0.2850	-0.0179	-0.2979
0.40	0.5815	0.2350	-0.0833	-0.3759	-0.6454

T_i=3,T_e= 20, F_y =500, F_{CU}=30					
Corrosion rate mm/years	Load ratio				
	0.5	0.6	0.7	0.8	0.9
	β	β	β	β	β
0.05	3.2500	2.9094	2.5754	2.2614	1.9626
0.10	2.7511	1.3915	2.0515	1.7303	1.4272
0.15	2.2130	1.8463	1.5027	1.1807	0.8792
0.20	1.6558	1.2894	0.9492	0.6328	0.3384
0.25	1.1037	0.7449	0.4136	0.1075	-0.1756
0.30	0.5815	0.2350	-0.0833	-0.3759	-0.6454
0.35	0.1107	-0.2211	-0.5247	-0.8026	-1.0582
0.40	-0.2930	-0.6100	-0.8992	-1.1636	-1.4057
T_i=3,T_e= 25, F_y =500, F_{CU}=30					
Corrosion rate mm/years	Load ratio				
	0.5	0.6	0.7	0.8	0.9
	β	β	β	β	β
0.05	3.1298	2.7804	2.4477	2.1313	1.8309
0.10	2.4858	2.1218	1.7790	1.4566	1.1536
0.15	1.7956	1.4286	1.0868	0.7684	0.4718
0.20	1.1037	0.7449	0.4136	0.1075	-0.1756
0.25	0.4583	0.1153	-0.1994	-0.4884	-0.7544
0.30	-0.1002	-0.4245	-0.7208	-0.9919	-1.2405

0.35	-0.5459	-0.8527	-1.1322	-1.3875	-1.6210
0.40	-0.8665	-1.1596	-1.4263	-1.6694	-1.8917

T_i=3,T_e= 30, F_y =500, F_{CU}=30					
Corrosion rate mm/years	Load ratio				
	0.5	0.6	0.7	0.8	0.9
	β	β	β	β	β
0.05	3.0064	2.6532	2.3177	1.9993	1.6976
0.10	2.2130	1.8463	1.5027	1.1807	0.8792
0.15	1.3775	1.0143	0.6778	0.3660	0.0769
0.20	0.5815	0.2350	-0.0833	-0.3759	-0.6454
0.25	-0.1002	-0.4245	-0.7208	-0.9919	-1.2405
0.30	-0.6202	-0.9239	-1.2005	-1.4530	-1.6840
0.35	-0.9587	-1.2478	-1.5107	-1.7503	-1.9692
0.40	-1.1122	-1.3943	-1.6508	-1.8845	-2.0979

T_i=3,T_e= 0, α =0.7, F_{CU}=30						
Corrosion rate mm/years	Yield Strenght					
	400N/mm2		500N/mm2		600N/mm2	
	β	P_f	β	P_f	β	P_f
0.05	2.4117		3.0588		3.5700	
0.10	2.4117		3.0588		3.5700	
0.15	2.4117		3.0588		3.5700	
0.20	2.4117		3.0588		3.5700	
0.25	2.4117		3.0588		3.5700	
0.30	2.4117		3.0588		3.5700	
0.35	2.4117		3.0588		3.5700	
0.40	2.4117		3.0588		3.5700	
T_i=3,T_e= 5, F_y =500, F_{CU}=30						
Corrosion rate mm/years	Yield Strenght					
	400N/mm2		500N/mm2		600N/mm2	
	β	P_f	β	P_f	β	P_f
0.05	2.2949		2.9423		3.4593	
0.10	2.1765		2.8229		3.3446	
0.15	2.0566		2.7005		3.2260	
0.20	1.9355		2.5754		3.1034	
0.25	1.8132		2.4477		2.9770	
0.30	1.6901		2.3177		2.8470	
0.35	1.5665		2.1855		2.7134	

0.40	1.4424		2.0515		2.5765	

T_i=3,T_e= 10, α =0.7, F_{CU}=30						
Corrosion rate mm/years	Yield Strenght					
	400N/mm2		500N/mm2		600N/mm2	
	β	P_f	β	P_f	β	P_f
0.05	2.1765		2.8229		3.3446	
0.10	1.9355		2.5754		3.1034	
0.15	1.6901		2.3177		2.8470	
0.20	1.4424		2.0515		2.5765	
0.25	1.1943		1.7790		2.2937	
0.30	0.9476		1.5027		2.0011	
0.35	0.7046		1.2251		1.7014	
0.40	0.4671		0.9492		1.3979	
T_i=3,T_e=15, F_y =500, F_{CU}=30						
Corrosion rate mm/years	Yield Strenght					
	400N/mm2		500N/mm2		600N/mm2	
	β	P_f	β	P_f	β	P_f
0.05	2.0566		2.7005		3.2260	
0.10	1.6901		2.3177		2.8470	
0.15	1.3183		1.9159		2.4365	
0.20	0.9476		1.5027		2.0011	
0.25	0.5851		1.0868		1.5499	
0.30	0.2371		0.6778		1.0943	
0.35	-0.0905		0.2850		0.6468	

0.40	-0.3929		-0.0833		0.2194	

T_i=3,T_e= 20, α =0.7, F_{CU}=30						
Corrosion rate	Yield Strenght					
	400N/mm2		500N/mm2		600N/mm2	
	β	P_f	β	P_f	β	P_f
0.05	1.9355		2.5754		3.1034	
0.10	1.4424		2.0515		2.5765	
0.15	0.9476		1.5027		2.0011	
0.20	0.4671		0.9492		1.3979	
0.25	0.0161		0.4136		0.7943	
0.30	-0.3929		-0.0833		0.2194	
0.35	-0.7508		-0.5247		-0.3007	
0.40	-1.0517		-0.8992		-0.7470	
T_i=3,T_e= 25, F_y =500, F_{CU}=30						
Corrosion rate	Yield Strenght					
	400N/mm2		500N/mm2		600N/mm2	
	β	P_f	β	P_f	β	P_f
0.05	1.8132		2.4477		2.9770	
0.10	1.1943		1.7790		2.2937	
0.15	0.5851		1.0868		1.5499	
0.20	0.0161		0.4136		0.7943	
0.25	-0.4875		-0.1994		-0.0834	
0.30	-0.9086		-0.7208		-0.5340	
0.35	-1.2382		-1.1322		-1.0263	

0.40	-1.4732		-1.4263		-1.3793	

T_i=3,T_e= 30, α =0.7, F_{CU}=30						
Corrosion rate mm/years	Yield Strenght					
	400N/mm2		500N/mm2		600N/mm2	
	β	P_f	β	P_f	β	P_f
0.05	1.6901		2.3177		2.8470	
0.10	0.9476		1.5027		2.0011	
0.15	0.2371		0.6778		1.0943	
0.20	-0.3929		-0.0833		0.2194	
0.25	-0.9086		-0.7208		-0.5340	
0.30	-1.2928		-1.2005		-1.1082	
0.35	-1.5406		-1.5107		-1.4807	
0.40	-1.6527		-1.6508		-1.6490	

T_i=3,T_e= 0, F_y =500, α =0.7					
Corrosion rate mm/years	Concrete Grade (Fcu)				
	20	25	30	35	40
	β	β	β	β	β
0.05	2.2245	2.6542	3.0588	3.4665	3.7865
0.10	2.2245	2.6542	3.0588	3.4665	3.7865
0.15	2.2245	2.6542	3/0588	3.4665	3.7865
0.20	2.2245	2.6542	3.0588	3.4665	3.7865
0.25	2.2245	2.6542	3.0588	3.4665	3.7865
0.30	2.2245	2.6542	3.0588	3.4665	3.7865
0.35	2.2245	2.6542	3.0588	3.4665	3.7865
0.40	2.2245	2.6542	3.0588	3.4665	3.7865
T_i=3,T_e= 5, F_y =500, α =0.7					
Corrosion rate mm/years	Concrete Grade (Fcu)				
	20	25	30	35	40
	β	β	β	β	β
0.05	2.0891	2.5287	2.9423	3.3282	3.6858
0.10	1.9503	2.4000	2.8229	3.2170	3.5814
0.15	1.8083	2.2682	2.7005	3.1030	3.4749
0.20	1.6632	2.1335	2.5754	2.9865	3.3660
0.25	1.5153	1.9961	2.4427	2.8675	3.2548
0.30	1.3647	1.8563	2.3177	2.7464	3.1415
0.35	1.2119	1.7142	2.1855	2.6232	3.0263
0.40	1.0570	1.5701	2.0515	2.4983	2.9094

T_i=3,T_e= 10, F_y =500, α =0.7					
Corrosion rate	Concrete Grade (Fcu)				
	20	25	30	35	40
	β	β	β	β	β
0.05	1.9503	2.4000	2.8229	3.2170	3.5814
0.10	1.6632	2.1355	2.5754	2.9865	3.3660
0.15	1.3647	1.8563	2.3177	2.7464	3.1415
0.20	1.0570	1.5701	2.0515	2.4983	2.9094
0.25	0.7424	1.2774	1.7790	2.2442	2.6718
0.30	0.4240	0.9808	1.5027	1.9863	2.4305
0.35	0.1049	0.6831	1.2251	1.7272	2.1881
0.40	-0.2116	0.3876	0.9492	1.4695	1.9467
T_i=3,T_e= 15, F_y =500, α =0.7					
Corrosion rate	Concrete Grade (Fcu)				
	20	25	30	35	40
	β	β	β	β	β
0.05	1.8083	2.2682	2.7005	3.1030	3.4749
0.10	1.3647	1.8563	2.3177	2.7464	3.1415
0.15	0.9004	1.4244	1.9159	2.3718	2.7912
0.20	0.4240	0.9809	1.5027	1.9863	2.4305
0.25	-0.0539	0.5349	1.0868	1.5980	2.0671
0.30	-0.5220	0.0972	0.6778	1.2156	1.7089
0.35	-0.9694	-0.3223	0.2850	0.8477	1.3638
0.40	-1.3863	-0.7145	-0.0833	0.5019	1.0385

T_i=3,T_e= 20, F_y =500, α =0.7					
Corrosion rate	Concrete Grade (Fcu)				
	20	25	30	35	40
	β	β	β	β	β
0.05	1.6632	2.1355	2.5754	2.9865	3.3660
0.10	1.0570	1.5701	2.0515	2.4983	2.9094
0.15	0.4240	0.9808	1.5027	1.9863	2.4305
0.20	-0.2116	0.3876	0.9492	1.4695	1.9467
0.25	-0.8232	-0.1851	0.4136	0.9682	1.4769
0.30	-1.3863	-0.7145	-0.0833	0.5019	1.0388
0.35	-1.8819	-1.1828	0.5247	0.0862	0.6472
0.40	-2.2983	-1.5782	-0.8992	-0.2679	0.3124
T_i=3,T_e= 25, F_y =500, α =0.7					
Corrosion rate	Concrete Grade (Fcu)				
	20	25	30	35	40
	β	β	β	β	β
0.05	1.5153	1.9961	2.4477	2.8675	3.2548
0.10	0.7424	1.2774	1.7790	2.2442	2.6718
0.15	-0.0539	0.5349	1.0868	1.5980	2.0671
0.20	-0.8232	-0.1851	0.4136	0.9682	1.4769
0.25	-1.5171	-0.8379	-0.1994	0.3928	0.9361
0.30	-2.1005	-1.3901	-0.7208	-0.0991	0.4722
0.35	-2.5551	-1.8233	-1.1322	-0.4892	0.1026
0.40	-2.8762	-2.1312	-1.4263	-0.7693	-0.1638

T_i=3,T_e= 30, F_y =500, α =0.7					
Corrosion rate mm/years	Concrete Grade (Fcu)				
	20	25	30	35	40
	β	β	β	β	β
0.05	1.3647	1.8563	2.3177	2.7464	3.1415
0.10	0.4240	0.9808	1.5027	1.9863	2.4305
0.15	-0.5220	0.0972	0.6728	1.2156	1.7089
0.20	-1.3863	-0.7145	0.0833	0.5019	1.0385
0.25	-2.1005	-1.3901	-0.7208	-0.0991	0.4722
0.30	-2.6300	-1.8949	-1.2005	-0.5541	0.0409
0.35	-2.9677	-2.2192	-1.5107	-0.8499	-0.2407
0.40	-3.1189	-2.3651	-1.6308	-0.9841	-0.3688