RELIABILITY ANALYSIS OF STRENGTH CHARACTERISTICS OF FIBRE REINFORCED FLY ASH CONCRETE

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CERTIFICATE

This is to certify that the project titled, **"RELIABILITY ANALYSIS OF STRENGTH CHARACTERISTICS OF FIBRE REINFORCED FLY ASH CONCRETE "**, being submitted by **Mr. AMRINDER SINGH PANDHER** , in partial fulfillment of the requirement for the award of degree of **MASTER OF ENGINEERING (STRUCTURAL ENGINEERING)** in the **Department of Civil Engineering, Thapar University, Patiala,** is a bonafide work carried out by him under our guidance and supervision and that no part of this seminar has been submitted for the award of any other degree.

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ABSTRACT

Structural codes and standards provide the foundation of good engineering practice and a framework for addressing safety and serviceability issues in structural design. They identify natural and man-made forces that must be considered and defined magnitude of these forces. The documents on which the structural engineer places so much reliance must address the question: “how safe is safe enough?” on behalf of society

as a whole. In the existing limit state design procedure, which is semi-probabilistic in nature, even through the design load is calculated statistically, research for determining the actual loading on the structure has not yielded adequate data to enable one calculate theoretical values of variations for arriving at the actual loading of the structures. Thus, at the root of the structural safety problem is the uncertain nature of

(i) The man-made and environmental forces that act on structure,
(ii) Material strengths, and
(iii) Structural analysis procedures that at best only theoretical models.

The traditional practice of setting safety factors and revising codes based solely on experience does not work in this environment, where such trial and error approaches to managing uncertainty and safety may have unacceptable consequences. In era in which standards and public safety are being increasingly questioned in public forum, more systematic and quantitative approaches, for public safety, are essential. The probabilistic approach which accounts for the said uncertainties has, in the past two decades, been widely accepted worldwide as a new paradigm, for design of structures and evaluation of the safety of exiting ones.

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In the present study a reliability-based Partial safety factors for compressive strength of concrete with partial replacement of cement by Fly ash having fibres has been developed considering the strength as a random variable. The compressive strength of concrete depends upon the properties of its constituent materials viz. cement, fine

aggregates, coarse aggregates and mineral admixtures such as Fly ash etc. in the present investigation, a step-by-step procedure has been suggested to find out the partial safety factors for fibrous fly ash concrete.

An extensive data bank on the basic variables viz. compressive strength of concrete in terms of mean, standard deviation and with-in-test coefficient of variation corresponding to the variation in parameters have been generated. The compressive strength data generated experimentally has been analyzed using normal-probability distribution functions. Furthermore, based on the analysis of the generated data, partial safety factors have been computed relative to 95 per cent confidence level values.

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CHAPTER 1 INTRODUCTION

1.1 GENERAL INTRODUCTION

Concrete is a relatively brittle material, when subjected to normal stresses and impact loads. Tensile strength of concrete is approximately one tenth of its compressive strength. As a result for these characteristics, plain concrete members could not support loads and tensile stresses that occurred, on concrete beams and slabs. Concrete members are reinforced with continuous reinforcing bars to withstand tensile stresses and compensate for the lack of ductility and strength. The addition of steel reinforcement significantly increases the strength of concrete, and results in concrete with homogenous tensile properties; however the development of micro cracks in concrete structures must be checked. The introduction of fibres is generally taken as a solution to develop concrete in view of enhancing its flexural and tensile strength.

Fly ash is the fine powder produced as a side product from the combustion of pulverized coal and is collected by mechanical and electrostatic separator from fuel gases of power plants. The disposal of fly ash is the one of the major issue for environmentalists as dumping of fly ash as a waste material may cause severe environmental problem. Therefore, the utilization of fly ash as an admixture in concrete instead of dumping it as a waste material can have great beneficial effects.. It can be used particularly in mass concrete applications where main emphasis is to control the thermal expansion due to heat of hydration of cement paste and it also helps in reducing thermal and shrinkage cracking of concrete at early ages. The replacement of cement with fly ash in concrete also helps to conserve energy. The composite matrix that is obtained by combining cement, Fly ash, aggregates and fibres is known as “Fly ash Fibre reinforced concrete”. The fibre in the cement fly ash based matrix acts as crack- arresters, which restrict the growth of flaws and micro cracks in the matrix, and prevent these from enlarging under load. Prevention of the propagation of these cracks, originating from internal flows, can result in the improvement of static and dynamic properties of the matrix.

1.1.1 Fibres in concrete

Fibres can be defined as a small piece of reinforcing material possessing certain dimensional characteristics. The most important parameter describing a fibre is its Aspect ratio. “Aspect ratio” is the length of fibre divided by an equivalent diameter of the fibre. The properties of fibre reinforced concrete are very much affected by the type of fibre. Fibres are secondary reinforcement material and acts as crack arrester. Prevention of propagation of cracks originating from internal flaws can result in improvements in static and dynamic properties of the matrix. The concept that post cracking of concrete can be improved by the inclusion of fibre was first put forward by ***Portar in 1910***, but little progress was made in the development of this material until 1963 when ***Romualdi and Batson 1969*** published their classic paper on the subject. Since then, there has been a wave of interest in fibre reinforced concrete and several interesting experiments have been carried out. Fibres are taken as a new form of binder that combines Portland cement in the bonding with cement matrices. Fibres are generally discontinuous, randomly distributed throughout the cements matrices. Several kinds of fibres such as steel, fibrillated polypropylene, nylon asbestos, coir, jute, sisal, kenaf, glass, and carbon have been tried and these are available in a variety of shapes, sizes, and thickness Fibres can be broadly be classified into two groups as Low Modulus High Elongation Fibres and High Modulus Fibres .

a) Low Modulus High Elongation Fibres

This group includes high elongation fibres having large energy absorption characteristics and are capable of imparting toughness and resistance to impact and explosive loadings. Fibres that are generally included in this group are nylon, polypropylene, polyethylene, rayon, acrylic and polyester fibres.

b) *High Modulus Fibres*

This group includes fibres, which are capable of producing strong composites; they primarily impart strength and stiffness to the composite to varying degrees and resistance under dynamic loadings. Fibres that are included in this group are steel,carbon,asbestos,organic fibres etc

1.1.2Application Areas of Synthetic Fibre in concrete construction

- Industrial flooring where improved abrasion resistance ,impact resistance is required
- Precast concrete blocks, manholes covers and tiles.
- Airport runways
- Retaining walls
- Water contact structures

1.1.3 Fly Ash

Fly ash is the residue obtained from combustion of pulverized coal collected by the mechanical or electrostatic separators from the fuel gases of thermal powe r plants. Its composition varies with the type of fuel burnt, load on boiler and type of separator etc. Fly ash consists mainly of spherical glassy particles ranging from 1 to 150 micrometers in diameter, out of which the bulk passes through a 45 -micrometer sieve. The fly ash obtained from electrostatic precipitators is finer then Portland cement. The fly ash obtained from cyclone separators is comparatively coarse. The carbon content in fly ash should be as low as possible, where as the silicon content should be as high as possible. The fly ash may be used in concrete either as an admixture or in part replacement of cement. The pozzolanic activity is due to the presence of finely divided glassy silica and lime, which produce calcium silicate hydrate (CS H) responsible for strength development. Due to the difference in densities of cement and fly ash, a part replacement by equal mass increases the volume of cementitious material; where as replacement by equal volume reduces the mass in practice the replace ment of cement by fly ash is usually on the mass basis. As per ASTM –C 618-93 fly ash and natural pozzolans are classified into the following into three categories: -

a) Class N Fly ash

Raw or calcined natural pozzolans such as some diatomaceous earths, opaline chert and shale, volcanic ashes and pumice come in this category. Calcined kaolin clay and laterite shale also fall in this category of pozzolans.

b) Class F Fly ash

Fly ash normally produced from burning anthracite or bituminous coal falls in this category. This class of fly ash exhibits pozzolanic property but rarely possesses self-hardening property.

c) Class C Fly ash

Fly ash normally produced from lignite or sub- bituminous coal is the only material included in this category. This class of fly ash has both pozzolanic and varying degree of self cementitious properties. (Most class C fly ashes contain more than 15 % CaO. But some class C fly ashes may contain as little as 10 % CaO.)

The typical physical and chemical requirements of various categories of fly ash are provided in table 1.1

1.1.4 Advantages and Limitations of Fibre reinforced concrete (FRC)

Fibres, which are randomly distributed throughout the concrete, can overcome cracks and control shrinkage more effectively. These materials have outstanding combinations of strength and energy absorption capacity. In general, the fibre reinforcement is not a substitution for conventional steel reinforcement. The fibres and steel reinforcement has their own role in concrete technology. However, fibres are not efficient in withstanding the tensile stresses compare to conventional steel reinforcement. But, fibres are more closely spaced than steel reinforcement, which are better in controlling crack and shrinkage. Consequently, conventional steel reinforcement used to increase the load bearing capacity of concrete member; fibres are more effective in crack control. Due to these differences, there are particular applications that fibres reinforced are advance than conventional steel reinforcement. These include:

•Fibres comprise as ‘primary reinforcement’, in which the conventional steel reinforcement cannot be utilized. The fibre concentrations are comparatively high in thin sheet materials, normally exceeding 5% by volume, acts to increase in toughness and strength of mortar or concrete.
• Fibres can be components to withstand locally high loads or deformations, which applies to structures like Precast piles, Precast walls, blast resistant structures or sewer tunnel and linings.
•Applications that control cracks persuaded by temperature and humidity, such as pavements and slabs, where fibres offered as ‘secondary reinforcement’.

The uses of steel bars and wire mesh require unnecessary labor and material costs for structure concrete. With replacement of randomly distributed short fibres as an alternative reinforcement, will significant reduce both labour and material costs, greatly increase construction and project time.Fibres substantially reduce formation of plastic shrinkage and settlement; enable the concrete to develop its potential long-term application to structural concrete, providing solution to exceed and meet their performance and economical prospect. Additionally, fibres provide an effective secondary reinforcement for shrinkage and crack width control. Macro-cracks and potential problems are prevented and blocked when micro-cracks intersect fibres as concrete hardens and shrink. Effects of crack control reinforcement by additional of fibres in concrete shown in figure 1.6.

1.2 INTRODUCTION TO STRUCTURAL RELIABILITY

The performance of a structure is assessed by its safety, serviceability and economy over its life cycle. However, the information about the input variables is never certain and complete. The uncertainties owe themselves to inherent randomness, limited information, imperfect knowledge and errors. With these uncertainties, to estimate absolute safety of a structure is impossible, due to:

a) Unpredictability of

i. loads on the structure during its life cycle,

ii. In-place material strengths, and,

iii. Human errors.

b) Structural idealizations in formulating the mathematical model of the structure to predict its response or behavior, and

c) The limitations of numerical techniques.

These factors lead to some risk with regard to the unacceptable performance of the structure. In the conventional deterministic analysis and design procedures, it is assumed that all parameters (loads, strength of materials etc) are not subjected to probabilistic variations. However, it is well known that the strengths of materials (concrete, steel etc) and the geometric properties (sectional dimensions, effective depth, diameter of bars etc) are subjected to statistical variations. Hence, to be rational in the estimation of the structural safety, the random variations of the basic parameters are to be taken into account. Since load and strength are random variables, the safety of the structure is also a statistical variable.

Code of practice for the design of reinforced concrete structures (IS 456 -2000) is based on the concept of partial coefficients. It defines two basic limit states: ultimate limit state and the serviceability limit state. The influence of uncertainties and variability in basic variables are taken into account by characteristic values and partial coefficients. This method is “semi-probabilistic” in nature as it uses a set of predefined characteristic values, which have predetermined probability of not being achieved.

To minimize the effect of uncertainties in the design parameters, safety is ensured by taking the smallest value of load resisting capacity (R) and largest value of the load (Q). The safety factor is then defined as; $\nu = R/Q$ .The safety so defined is very conservative and leads to uneconomical design. An alternative method of fixing the safety is as follows:

Let ΔR and ΔQ be the admissible deviation from R and Q, respectively. For the structure to be safe

$$\frac{R}{Q} > \frac{\left(1+\frac{\Delta Q}{Q}\right)}{\left(1-\frac{\Delta R}{R}\right)}$$

$$v = \frac{\left(1+\frac{\Delta Q}{Q}\right)}{\left(1-\frac{\Delta R}{R}\right)} \qquad (1.1)$$

$$Q = \frac{\mu_s}{\sigma s}$$

$$P_f = \frac{1}{2}\left[\left(\frac{\mu_R - \mu_Q}{\sigma s}\right)\right]$$

$$R - \Delta R > Q + \Delta Q \qquad (1.2)$$

$$R\left(1-\frac{\Delta R}{R}\right) > Q\left(1+\frac{\Delta Q}{Q}\right) \qquad (1.3)$$

$$\frac{R}{Q} > \frac{\left(1+\frac{\Delta Q}{Q}\right)}{\left(1-\frac{\Delta R}{R}\right)}$$

(1.4)

Hence, the minimum value of safety factor is

$$v = \frac{\left(1+\frac{\Delta Q}{Q}\right)}{\left(1-\frac{\Delta R}{R}\right)}$$

(1.5)

The above definition of factors of safety not only widely varies but is also probabilistically inaccurate.

The safety factor can also be expressed as the ratio of the mean values of R and Q, which is called the central safety factor ν_c

$$\nu_c = \text{mean value of R/mean value of } Q = \frac{\mu_R}{\mu_Q} \tag{1.6}$$

The drawback in this definition of safety factor can be clearly understood from the following.

Fig.1.1 shows the probability density functions $f_R(r)$ and $f_Q(q)$ of R and Q. It is observed that both distributions overlap. The shaded portion gives an indicative measure of the probability of failure of the element or the structure. It can be easily shown that for the same value of ν_c, the value of probability of failure p_f can be different.

First, consider the case where mean action and resistance are increased by some proportion (say by k_1) as in Fig. 1.2, keeping their standard deviations constant. Thus

$$\nu = \frac{k_1\mu_R}{k_1\mu_Q}$$

$$= \frac{\mu_R}{\mu_Q} \tag{1.7}$$

It is observed from Fig.1.2 that even though ν_c the same, overlap of the two curves changes, which indicates change in p_f.

Next, if the mean values of R and Q i.e. μ_R and μ_Q are kept constant (i.e. ν_c is maintained constant) and dispersions in R and Q are changed as shown in Fig.1.3, it is again observed that the overlap of the two curves changes, indicating a change in the value of p_f .

Thus, the area of overlap, which provides the qualitative measure of the probability of failure, depends upon three factors:

i. The shapes of the curves (represented by the probability distribution functions $f_Q(q)$ and $f_QR(r)$

ii. The relative positions of two curves (represented by the means of two variables), and
iii. The dispersion of the curves, characterized by standard deviations of two variables σ_Q (q) and σ_R (r).

Therefore, in order to achieve a safe design, the design variables must be so chosen that the area of overlap between the two curves lead to an acceptable probability of failure. The best way, therefore, to define safety is by the probability of failure or reliability. According to ***Freudenthal (1956)***: “Because the design of a structure embodies uncertain predictions of the performance of structural materials as well as of the expected load patterns and intensities, the concept of probability must form an integral part of any rational analysis or design; any conceivable condition is necessarily associated with a numerical measure of the probability of its occurrence. It is by this measure alone that the structural significance of a specified condition can be evaluated”.

The concept of reliability has been applied to several fields and has been interpreted in many ways. Structural reliability is concerned with quantifying the performance levels of structures using the theories of probability and statistics and is always oriented towards relating element to system reliability and identifying the failure sequence or path ***Kalaga, 1998***. Reliability, as is most commonly defined, is the probability of an element or system performing its intended function over a given period of time under the operating conditions encountered. The above definition stresses four significant parameters, viz.

i. Probability,
ii. Intended function,
iii. Time, and
iv. Operating conditions.

Because of the uncertainties involved, the reliability is the first parameter in the definition. The intended function signifies that reliability is a performance characteristic. For a structure to be reliable, it must perform certain functions satisfactorily for which it has been designed and for all time. Hence, reliability is related to time, thus bringing in the concept of life cycle of structures. The last parameter is the operating conditions, which establishes the actions or stresses that will be imposed on the structure.

The basic principle is that reliability explicitly considers the probability that the structure shall fail during its life span, and that this probability is a function of the failure probabilities of its individual elements. The American Society of Civil Engineers (ASCE) Task committees (as in Kalaga, 1998) have provided guidelines for the application of reliability methods to structures. The requirements include:

1) A database to describe the probability of occurrence and duration of loading.
2) Probabilistic models of strength of elements of the structure, quantifying the load resistance capacity 'R'.
3) Models to describe the loading and response.
4) Procedure for computing reliability measures such as probability of failure, safety or reliability index etc.
5) Types of limit states used in defining element/system failure.

In structural analysis and design, reliability is defined as the probability that a structure will not attain each specified limit (flexure, shear, torsion or deflection criteria) during a specified reference period (life of the structure). For convenience, the reliability

R is defined in terms of probability of failure, P_f which is taken as

$$R_0 = 1 - p_f \tag{1.8}$$

In case of classical reliability theory, for reliability prediction information on life characteristics of the system, operating conditions and failure distribution are needed. However, for structural systems, it is difficult to predict the expected life or the expected failure rate or the expected time between breakdowns. So in the reliability format, it is assumed that structural failures are not due to deterioration. The structures cannot be assumed to be normally identical. The structural failures cannot be expressed in terms of relative frequency. Thus, the structural reliability differs from the classical reliability theory in many such aspects except in the probabilistic nature of the uncertainties. The probability of failure of a structure is a subjective probability. The reliability of a structure is not a unique property. It changes as the state of knowledge about the structure changes.

The acceptable probabilities for structural failures are very low viz.i.e of the order of 10^{-3} for serviceability limit states, meaning thereby that on an average, out of 1000 normally identical structures, one may deform excessively or

ii. Of the order of 10^{-6} for ultimate limit states, this means that out of one million identical structures, one may collapse.

However, in practice structures are never identical in such large numbers and also these low probabilities have to be estimated from the statistical properties extrapolated from the available statistical data around the central values of the random variables. Thus, probabilistic methods play an important role in making rational comparisons between alternative designs.

1.2.1 Computation of Structural Reliability

Consider a simple structure with one element only. Let R be the resistance (capacity or strength) of the structure and Q the action (load or load effect, viz, bending moment, shear force etc.) on the structure. The structure is said to fail when the resistance of the structure is less than the action.

i.e. $$P_f = P(R < Q) = P((R - Q) < 0) \qquad (1.9)$$

where p_f is the probability of failure of the structure.

Let us define safety margin as a function S, represented as

$$S = R - Q \qquad (1.10)$$

Let S have μ_s mean and σ_s as standard deviation. A plot of probability distribution function $f_S(s)$ is shown in Fig.1.4.

The shaded portion of Fig.1.4 represents cases where $S < 0$. If the type of probability distribution of S is known then the probability of failure can be computed from the value of standard deviation that exceeds zero. This is shown in Fig. 1.4 as $\beta\sigma_S$ where β is 'safety index' or 'Reliability index' defined as,

$$\beta = \frac{\mu_s}{\sigma s}$$

Which is the reciprocal of the coefficient of variation If the function S is normally distributed, the probability of failure is

$$P_f = \frac{1}{2}\left[\left(\frac{\mu_R - \mu_Q}{\sigma s}\right)\right] \tag{1.11}$$

or in terms of reliability index as,

$$P_f = 1/2 - \psi(\beta) \tag{1.12}$$

Where $\psi(\beta)$ is standard normal function. If β is a normal variate, then P_f can be approximated as

$$P_f = 1 \times 10^{-\beta} \tag{1.13}$$

1.2.2 Reliability-based Methods

The Joint Committee on structural safety (1976) has classified reliability analysis and the safety checking into three groups terming them as Level 1, 2 and 3 methods. These levels have been defined as:

Level – 1 method: A design method in which appropriate levels of structural reliability are provided on a structural element basis by the specification of a number of partial safety factors related to some predefined characteristic value of the basic variables. The present structural design methodology (IS: 456 -2000) with consideration of the number of limit states is nothing but Level – 1 design. The limit state is a criterion to define a particular failure or performance condition.

Level – 2 methods: A design method incorporating safety checks only at selected point(s) on the failure boundary rather than a continuous process. In these methods, certain idealizations and assumptions are used. Mean values and variances of the random variables are only required. In advanced Level – 2 reliability methods, distributions are taken care of in an approximate way. Reliability levels are defined by safety indices or equivalent “operational” or “notional” probabilities. These methods are approximate as compared to level 3 methods.

Level – 3 methods: In this method safety checking is based on exact probabilistic analysis for whole structural systems or structural elements, using full distributional approach based on failure probabilities, possibly being derived from optimization studies or assessed by other approach criteria. They are purely probabilistic methods and are exact in estimating the reliability.

In the Level – 2 methods certain idealizations and assumptions have to be made. Mean values and variances along with probability distributions are used for carrying out reliability analysis and design. Level – 2 reliability methods are more practical and are quite suitable for design ***Ranganathan, 1990***. They are suitable for calibrating codes on reliability basis. The present study uses Level – 2 methods for arriving at the design procedure for concrete mix.

1.3 AIM AND SCOPE OF THE PRESENT INVESTIGATION

It has been generally recognized that some uncertainty exists in the results of design formulae for evaluating the resistance of reinforced concrete sections. One of the major factors contributing to this uncertainty is the variability of the strengths of the constituent materials. Some previous studies have examined ways in which safe designs could be satisfactorily developed in view of these uncertainties. Two approaches have been reported in the literature (Harr, 1987). One approach has been to apply the techniques of error statistics to the results of full-scale tests. Another approach is to treat the material strength and the ultimate resistance of the section as random variables, where in the ultimate strength is a function of material strength with the functional relationship derived from the structural theory.

The aim of the present study is to carry out reliability analysis of concrete mix with partial replacement of cement by Fly ash including certain fixed amount of fibres by considering the materials properties (compressive strength of concrete) as random variables. The compressive strength of concrete in turn depends upon the properties of its constituent materials viz, cement, fine aggregates, coarse aggregates, fly ash etc. In the present investigation effort has been made to develop partial safety factors of fibre reinforced fly ash concrete.

The compressive strength data required for the probabilistic analysis has been generated in the laboratory by varying the quantities of various constituent materials. The compressive strength data thus generated has been probabilistically analyzed and based on the analysis of the data thus generated, partial safety factors have been found out relative to 95 per cent confidence level values, which can be used in designing structural elements.

Table 1.1 Chemical and Physical requirements for fly ash for use as a mineral admixture in Portland cement as per ASTM C 618 -93

Requirements	Fly Ash Classification		
	N	F	C
Chemical Requirements			
$SiO_2+Al_2O_3+Fe_2O_3$, min%	70.0	70.0	50.0
SO_3 , max%	4.0	5.0	5.0
Moisture content, max%	3.0	3.0	3.0
Loss on ignition , max%	10.0	6.0	6.0
Physical Requirements			
Amount retained when wet sieved on 45 µm Sieve max%	34	34	34
Pozzolanic activity index, with Portland cement at 28 days, min % of control	75	75	75
Pozzolanic activity index, with lime, at 7 days, min(Mpa)	5.5	5.5	-
Water requirement, max % of control	115	105	105
Autoclave expansion or contraction, max %	.8	.8	.8
Specific gravity, max variation from average.	5	5	5
Percentage retained on 45 sieve, max variation, and percentage points from average.	5	5	5

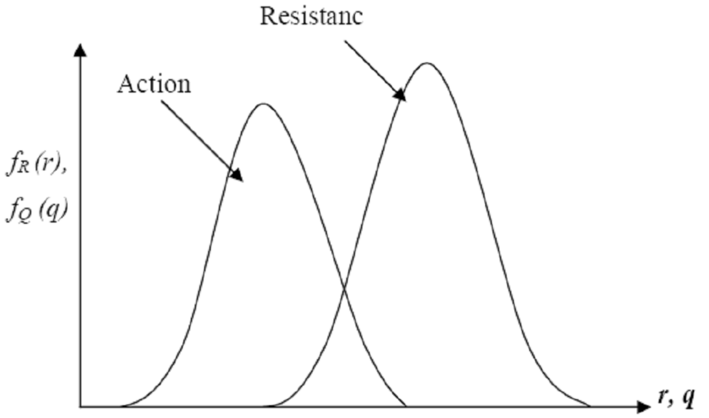


Fig. 1.1 Overlap of action and resistance distributions indicating failure probability *(Ranganathan, 1990)*

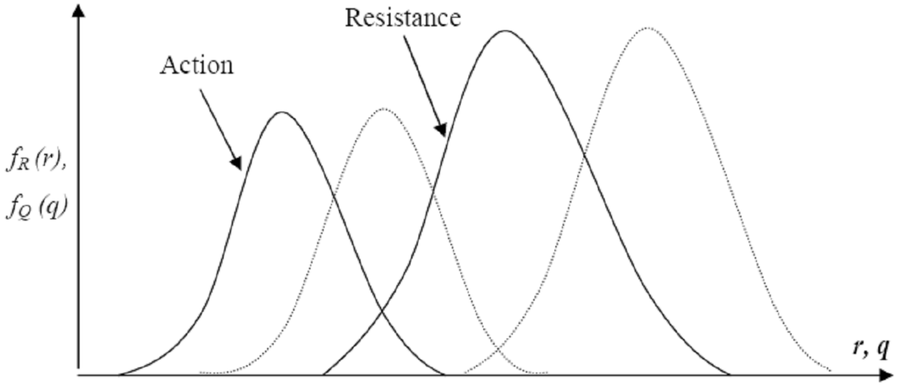


Fig. 1.2 Effect on failure probability due to proportional change in action and resistance *(Ranganathan, 1990)*

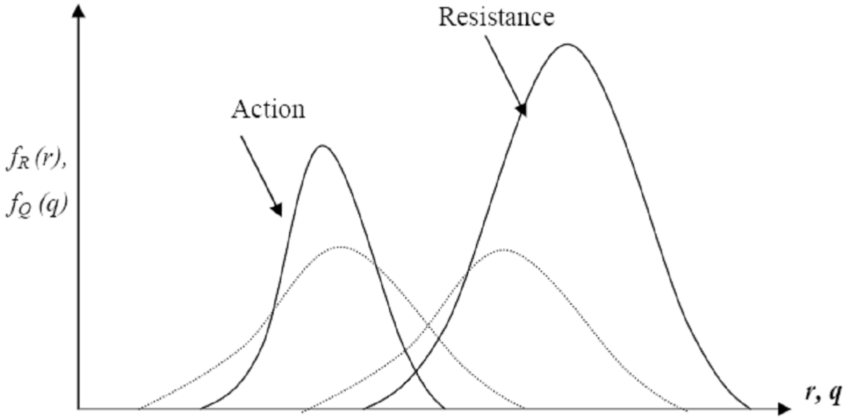


Fig. 1.3 Effect on failure due to change in dispersion of action and resistance *(Ranganathan, 1990)*

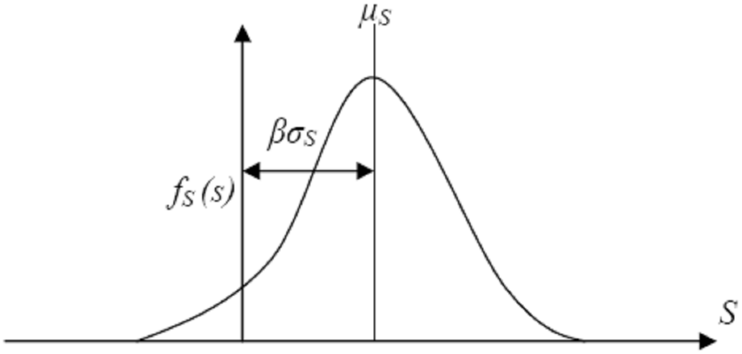


Fig. 1.4 Definition of reliability index, β (*Kumar, 2002)*

Figure 1.5 Application of Fibre reinforced concrete used in the modern industries.

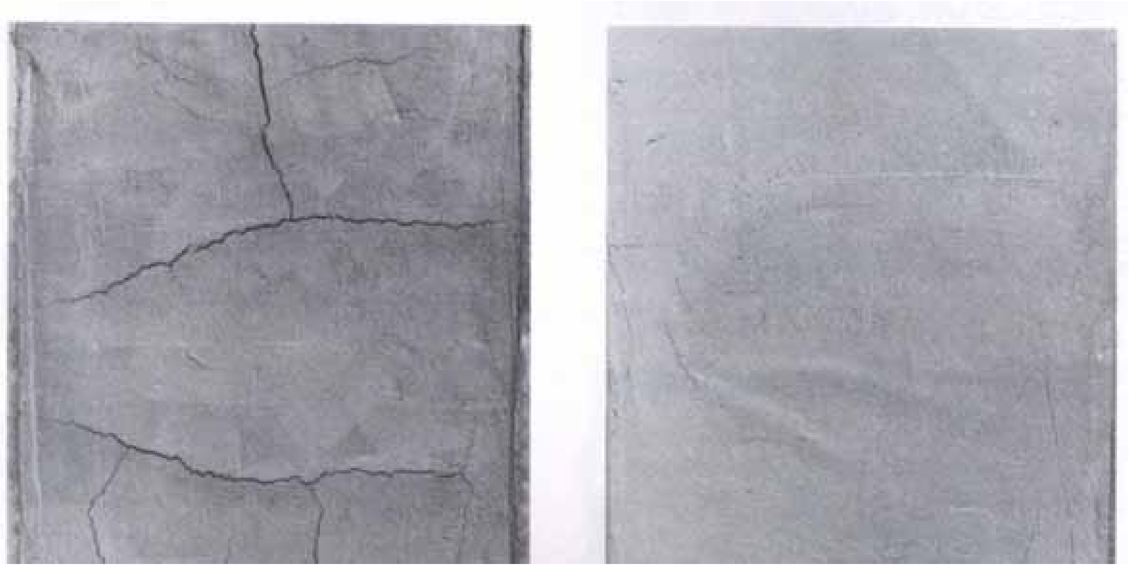
Car park, driveways

Airport Runway

Heavy duty slabs

Sewer tunnels

Figure 1.6: Comparison of cracks with and without Fibre reinforced concrete *(Source: Fibremesh, 1989)*



Without fibre-reinforced **With fibre-reinforced**

CHAPTER 2 LITERATURE REVIEW

2.1 GENERAL

The estimation of failure probability of structures has been an active area of research for more than two decades. During this period, efficient procedures have been developed for reliability evaluation of individual limit states in large structures. The present day codes of practice for the design of reinforced concrete structures are based on the design format of partial coefficients recommended by International Standard Organizations. The partial coefficient format defines two limit states: the ultimate limit state and serviceability limit state. The influence of uncertainties and variability's for basic variables is taken into account by characteristic values and partial coefficients. This method involving use of partial coefficients is also called "semi -probabilistic" because it uses a set of predefined values which have a prescribed probability of not being obtained. "More developed" methods use a predefined set of stochastic variables directly and employ the failure probability instead of design equations in the evaluation of safety of structures.

From the foregoing it is clear that the provisions of existing codes, for the sake of being simple, do not consider explicitly the randomness of load and strength. In recent years much literature has appeared in the area of reliability based design and probability of failure of structures due to random variations of load and strength. The earliest reported work on the introduction of probabilistic concepts and procedures to problems in structural engineering dates back to 1968, when a series of papers were developed and written by members of subcommittee – E (Probabilities) of ACI committee – 348 for presentation at the 1968 ACI fall meeting at Memphis in November 1968.

In the papers presented by ***Shah (1969)***, ***Sexsmith and Nelson (1969), Benjamin and Lind (1969) and Cornell (1969)***, a goal had generally been established of designing a structure and structural material (concrete) for a specified allowable failure probability. A brief review of the literature representing the work done by various researchers in the field of reliability analysis and design is presented in the following sections.

2.2 STATISTICAL DESCRIPTION OF STRENGTH OF CONCRETE

Elingwood and Ang (1974) have provided the qualitative analysis of the design uncertainties and have shown their effect on the level of risk. For purpose of analysis and design, mean and the basic variability have been estimated from the data reported in the literature. A prediction error has been assigned to the predicted mean to account for the inaccuracies in its estimation. It has been stressed that the choice of the prediction error, Δ, would reflect the confidence placed by an engineer on the accuracy of his prediction, relying perhaps on his past experience with similar situations. The prediction error with respect to compressive strength of concrete has been discussed.

The strength of reinforced concrete member may vary from the calculated or "nominal strength" due to variations in the material strengths and dimensions of the member, as well as uncertainties inherent in the equations used to compute member strengths. A review of these sources of uncertainty and their effects on the mean and coefficient of variation in the member resistances is presented below. The variability in the compressive strength of concrete has been reported for tests on standard cylinders for concrete proportioned for a nominal compressive strength ***(Elingwood and Ang, 1974)*** of $f'c$ of 3,000 psi (21 MN/m^2) and 3,456 psi (23.8 MN/m^2), with coefficient of variation of 0.12. Additional data defining the mean of the strength of concrete delivered by a ready mixed concrete company during separate monthly periods has indicated an error for the measured strength of 0.07. A comparison of the strength of in-situ concrete with that determined from standard cylinders (Bloem, 1968) indicated that under field conditions, the strength is 10 to 21 per cent lower than the strength observed under laboratory conditions. Assuming these factors to contribute a combined uncertainty of 0.16, a prediction error has been determined to be 0.18 and corresponding uncertainty has been determined to be 0.21.

Ellingwood (1978) estimated the COV of the in-situ strength as 0.207 for average control. It has been concluded that the COV in the tensile and compressive strengths of concrete could be of the same order. ***Mirza et al (1979)*** have derived a relationship for the coefficient of variation on in-situ compressive strength as

$$Vc = (V^2ccyl + 0.0084)^{½} \tag{2.3}$$

Where Vccyl is the coefficient of variation (COV) of the cylinder tests. For average control Vccyl is about 15 and 12 per cent for 3000 and 5000 psi concretes, respectively, and Vc can be taken as 18 and 15 per cent.

According to ***Mirza et al (1979)*** the COV of the in-situ tensile strength can be taken equal to 18 per cent, which is the value assumed for the compressive strength. The coefficients of variation obtained by assuming that both the tensile and compressive strengths follow a normal distribution have been tabulated in Table 2.1.

MacGrego et al (1983) have used Monte Carlo technique for determination of probability distribution and statistics for capacities of reinforced and pre-stressed concrete members. It involved comparing the results of the concrete and methods of calculating the member's resistance to the actual test results. The bias and variability of the procedure was focused out using the concept of 'model error.'

Leon et al (1978) have compared the accurate calculation procedure to tests, to get the mean and COV of the ratio of test strength divided by calculated strength. The Variability determined in this way was assumed to result from three causes

$$V_{T/C} = \sqrt{V_m{}^2 + V_{test}{}^2 + V_{spec}{}^2} \tag{2.4}$$

Where $V_{T/C}$ is the coefficient of variation obtained directly from the comparison of the measured and calculated strengths; Vm represents the variability of the model itself, V_{test} represents the uncertainties in the measured loads due to such things as the accuracies of the gauges, errors in readings, definition of failure; and V_{spec} represents errors introduced by such things as differences between the strengths in the test specimen and in control cylinders, and variations in actual specimen dimensions from those measured.

Duval and Kadri (1998) have proposed a model to evaluate the compression strength of silica fume concrete at any time. The model is related to the water - cementitious materials and silica -cement ratios.

Kumar (2002) have used the normal and lognormal distribution for representing the concrete compressive strength and developed the mix design procedure for concrete with partial replacement of cement by fly ash. He also considered other variables as w/c ratio, zone of aggregates. The governing equation for evaluation of the design values had been given in the form

$$(\mu_{CSrev} + \sigma_{CSrev} z_{cs}) - \frac{1}{A}(\mu_{Wcr} + \sigma_{Wcr} z_{wcr})^{B} = 0 \qquad (2.5)$$

where μ_{CSrev} and σ_{CSrev} are the revised means and standard deviations of compressive strength and μwcr and σwcr are the revised means and standard deviations of water-cementitious ratio. The values of α_1 and α_2 have been computed using Hasofer Lind’s method for different cases using normal and lognormal distribution functions.

Bhanja and Sengupta (2002) present a paper which deals with a mathematical model development using statical methods to predict the 28 -days compressive strength of silica fume concrete with water-to-cementitious ratios ranging from 0.3 to 0.42 and silica fume replacement percentage from 5 to 30 per cent. Strength results of 26 concrete mixes, on more than 300 test specimens, have been analyzed for statical modeling. The relationship between 28-days strength of SF and control concrete and silica fume replacement percentage based on test results , has been obtained as:

$$\frac{f_{SF}}{f_C} = 1.0663 + 0.0159(SF\%) + .007(SF\%)^2 - .00003(SF)^3 \qquad (2.6)$$

2.3 CONCRETE STRENGTH IN COMPRESSION

Under the current design, production, testing, and quality-control procedures the strength of concrete in a structure may differ from its specified design strength and may not be uniform throughout the structure. The major sources of variations in concrete strength are the variations in material properties and proportions of the concrete mix, the variations in mixing, transporting, placing and curing methods, the variations in testing procedures, and the variations due to concrete being in a structure rather than in control specimens.

2.3.1 Degree of control

First and foremost, the variability of concrete strength depends on the quality control of the concreting operation. In the construction of the Skylon Tower at Niagara Falls ***(Lauer and Rigby, 1966)***, coefficients of variation ranging from 7 per cent to 10 per cent were achieved for field-cast laboratory-cured cylinders using exceptional quality-control methods. The coefficients of variation ***(Mirza et al, 1979)*** of field-cast laboratory-cured specimens have been, in many cases, found to be 15 per cent to 20 per cent, which suggested that 20 per cent was a reasonable maximum value for average controls.

According to ***Mirza et al (1979)***, based upon the data available in the literature (Erntroy, 1960; Murdock, 1953 and ACI-214, 1965) the average coefficient of variation can be taken as roughly constant at 10 per cent, 15 per cent and 20 per cent for strength levels below 4,000 psi for excellent, average, and poor control, respectively. For concrete with an average strength above 4,000 psi, the standard deviation remains approximately constant with values of 400, 600, and 800 psi, respectively, for the three levels of control listed previously. This is expected since the greater control required for the production of higher strength concrete contributes to smaller variability. The total variation in the concrete strength measured by control cylinders included the variation in concrete strength within a single batch. This in-batch variation may be considered as the variation in testing procedures, mixer inefficiencies, and in actual concrete strength. The in-batch coefficients of variation of laboratory tests as reported by ***Komlos (1970) and Ramesh and Chopra (1960)***, varied from 0.5 per cent to 8.1 per cent with an overall average value of 3.6 per cent.

American Concrete Institute Committee, ACI – 214 (1965) has recommended that the level of control for within-batch tests could be divided into three classes with corresponding coefficients of variation as 4 per cent to 5 per cent for good control, 5 per cent to 6 per cent for average control and above 6 per cent for poor control.

The ACI committee report 214 (1988) has subsequently revised the control standards for evaluating performance of testing program using the within-test coefficient of variation as listed in the Table 2.2.

According to the ACI: 214 report if within-test coefficient of variation is high (fair or poor), testing may be the reason for poor test results rather than the concrete quality.

2.3.2 Distribution of compressive strength of concrete specimens

The majority of researchers including ***Julian (1955) and Shalon and Reintz (1955)*** have represented the distribution of concrete compressive strengths with a normal distribution. ***Rusch et al (1980)*** (as reported by Mirza et al, 1979) have performed 829 individual series of tests and found 93 per cent of these to be normally distributed. The balance was divided between positively and negatively skewed or multi-peaked distributions. At the same time, however, ***Fruedenthal (1956), Julian (1955) and Shalon and Reintz (1955)*** have suggested that the lognormal distribution gives a better fit for concrete strength in which the control is poorer than average (COV greater than 15 per cent to 20 per cent). From the evidence available particularly from the study by Rusch et al (as reported by Mirza et al, 1979) it has been found reasonable to assume a normal distribution for the compression strength of concrete. In the present study both normal and lognormal distributions have been used for evolving the reliability based design procedure. ***Rangantahan (1990)*** has presented the results of the statistical analysis of the data on various properties of concrete collected from various sources in the Table 2.3.

2.3.3 In-situ strength versus control strengths

The strength of concrete in a structure tends to be somewhat lower than the strength of control cylinders or cubes moulded from the same concrete. This difference is due to the effects of different placing and curing procedures, the effects of vertical migration of water during the placing of concrete in deep members, the effects of difference in size and shape, and the effects of different stress regimes in the structure and the specimens (Petersons, as reported by Mirza et al, 1979). The difference in directions of casting and loading of the structure and the specimens may also influence the strength.

The average ratios of core strength to standard cylinder strength from various studies ***(Bloem,1969; Campbell and Tobin, 1967)*** varied from 0.74 to 0.96 with an overall average from all studies of 0.87. Petersons (as reported by Mirza et al, 1979) has shown, however, that the ratio of the strength of concrete in a structure to the strength of the same concrete in a standard cylinder is not a constant and decreases as the strength level increases. ***Bloem (1969)*** had observed that the strengths of cores drilled from slabs were 93 per cent of the strength of push-out cylinders from the same slab. This was essentially independent of the type of cement used, the curing conditions, and the age of the concrete at testing. It was also observed that the strength of concrete cores from well- cured slabs was 90 per cent of the strength of well-cured standard cylinders moulded from the same concrete. The ratio reduced to 79 per cent for cores cut from slabs subjected to lower standards of curing nearly typical of usual field practice and cylinders from the same concrete. This suggested a reduction of approximately 12 per cent in the in-situ strength of concrete for minimum acceptable field-curing conditions.

Petersons (as reported by Mirza et al, 1979) has also concluded that the difference in in-situ strengths for minimum acceptable and good curing standards could be approximated by a factor of 0.9. The reduction in the in-situ strength of concrete is partially offset by the requirement that the average cylinder strength must be about 700 psi to 900 psi greater than the specified strength to meet the existing design codes (ACI Standard 318-77). Based on this observation and on equations and data from ***Allen (1970), Bloem (1969), and Petersons*** (as reported by Mirza et al, 1979), it has been suggested by ***MacGregor (1976)*** that the mean 28-day strength of concrete in a structure

$$\bar{f}_{CSTR35} = .675 f_c^{'} + 1100 \leq 1.15 f_c^{'} psi \qquad (2.7)$$

in which f'c is the design compressive strength of concrete defined in accordance with ACI standard 318-77

2.3.4 Effect of Volume

The in-situ strength of concrete is affected by the difference in the volumes of material under stress. The statistical theory of brittle fracture of solids ***(Bolotin, 1969)*** gives good estimates for the influence of size in geometrically similar specimens. According to this theory, the dependence of mean compressive strength on the volume can be represented by the expression

$$\bar{X} = \bar{X}_0 \left[.58 + .42 \left(\frac{v_o}{v} \right)^{1/3} \right] \qquad (2.8)$$

in which Xo and Vo represent the strength and volume of a 4-inch side cube specimen. Eq. (2.8) indicates that as the volume increases, the mean strength decreases. When the volume tends to infinity, the mean strength tends to the strength of the weakest constituent element of the material and approaches 0.58 X o. ***Bolotin (1969)*** has also suggested an alternative expression for estimating the dependence of the compressive strength variability as

$$V_x = \frac{0.147 \left(\frac{V_0}{V} \right)^{1/3}}{.58 + .42 \left(\frac{V_0}{V} \right)^{1/3}} \qquad (2.9)$$

From Eq. (2.9) it can be observed that as the volume increases, the coefficient of variation decreases, the coefficient of variation tends to zero when volume tends to infinity.

2.3.5 Speed of loading

The observed strength of concrete is considerably affected by the rate of application of the load: the lower the rate of loading, the lower the apparent strength. This is probably due to the increase in strain with time owing to creep and micro cracking. The normal rate of loading for the standard cylinder test is approximately 35 psi/sec (ASTM standards, 1976) and for the cube test is 5.25 kN/sec (IS: 516-1959). As reported by ***Mirza***

et al (1979), Jones and Richart have compared the normal rate of loading for testing cylinders and observed that loading at 1 psi/sec reduces the apparent strength of concrete by approximately 12 per cent whereas loading at 1,000 psi/sec increases the strength by approximately 12 per cent. Based on standard compression cylinder tests, Jones and Richart have proposed a relationship between compressive strength of concrete and the rate of loading that has been rewritten in Eq. (2.8) to relate the 28 day mean strength of concrete to the nominal testing speed for cylinder tests, which has been taken as 35 psi/sec.

$$\bar{f}_{CR} = .89\,\bar{f}_{C35}(1 + .08\log R) \qquad (2.10)$$

in which (0.1 ≤ R psi/sec ≤ 10,000). Although no information is available on variability of the concrete strength due to speed effects, a very small dispersion in the strength was observed. According to ***Rusch (1960)***, concrete is capable of indefinitely withstanding stress only up to 70 per cent to 75 per cent of the strength under loads applied at 35 psi/sec.

2.3.6 Model for in-situ concrete strength in compression

Mirza et al (1979) presented the combined effect of volume, speed of loading on the concrete compressive strength in a structure, in the following model:

$$f_{cstrR} = f'_c\, r_{creal}\, r_{in-situ}\, r_R \qquad (2.11)$$

In which

r_{creal} = random variable relating real cylinder strength to design compressive strength;
$r_{in-situ}$ = random variable relating in-situ strength to real cylinder strength; and

r_R = random variable relating strengths at R psi/sec and 35 psi/sec.

The mean value for the in-situ compressive strength of concrete at a given rate of loading R psi/sec was found to be:

$$\bar{f}_{cstrR} = \bar{f}_{cstr35}\left[.89(1+.08\log R)\right] \tag{2.12}$$

in which

F_{cstr} 35 is the mean 28 days strength of concrete in a structure at normal rate of loading.

The coefficient of variation of the in-situ compressive strength of concrete at a given rate of loading can be calculated using the model of Eq. (2.11) and has been expressed as

$$V^2{}_{cstrR} = V^2{}_{Creal} + V^2{}_{In-situ} + V^2{}_{R} \tag{2.13}$$

The strength of concrete measured by control cylinders includes variations in the "real" concrete strength and the so-called in-test variations due to testing procedure. Thus,

$$V_{ccyl}^2 = V_{real}^2 + V_{in-test}^2 \tag{2.14}$$

Using Eqns. (2.13) and (2.14) and assuming variation of concrete in a structure with respect to the compressive strength of control cylinders, Vin-situ, of 10 per cent ***(Davis, 1976)*** an in-test variation, Vin-test, of 4 per cent and the variation due to the rate of loading effect, VR, to be negligible, the coefficient of variation of the in-situ strength of concrete at a given rate of loading, according to ***Mirza et al (1979)***, is given by

$$V_{cstrR}^2 = V_{ccyl}^2 + 0.084 \tag{2.15}$$

2.4 EFFECT OF FIBRES

2.4.1 Cracking characteristics

Achyutha and Sabapathi (2006) presented results of an experimental investigation on the effects of inclusion of steel fibres in conventionally reinforced concrete beams on their cracking characteristics. They concluded that load at first visible crack increases by 50 to 128% due to inclusion of steel fibres over the whole section of a reinforced concrete beam. But the increase is of the order of 30% only in the beams with fibres around the tension steel only, compared to beams without fibres and beams with extra reinforcement equivalent to the quantity of fibre around tension steel. The presence of fibres reduces the crack height by 25%. There is a reduction of 50 to 90% in the maximum crack width at working load depending upon the fibre aspect ratio. The effect of fibres on maximum or mean crack spacing has been found to be insignificant irrespective of the fibre aspect ratio and volume percentage. The general trend is that an increase in the fibre content and aspect ratio enhances the percentage increase in load at a specified crack width.

Batson and Romualdi (2000) in their paper presented that the first crack strength of concrete improves by mixing closely continuous steel fibres in it. These steel wires act as crack arresters preventing the advancing micro cracks by applying pinching forces at the crack tips and thus delaying the propagation of cracks. The existence of crack arrest mechanism in closely spaced wire reinforced concrete also suggests that the increase in strength of concrete is inversely proportional to the square root of the wire spacing.

2.4.2 Flexural strength and compressive strength

Banthia and Dubey (2000) used Residual strength test method (RSTM) to measure the flexural toughness of fibre-reinforced concrete in terms of its post peak residual strength. This method has the ability to identify the influence of different fibre characteristics such as type, length, configuration, volume fraction, geometry, and the modulus of elasticity. The results were based on two set of testing. Test of set 1 clearly stated that fibrillated polypropylene fibres provide a better toughness than monofilament polypropylene fibres. Test of set 2 noted that hooked-end steel fibres verified a better toughening strength than crimped steel fibres in fibre -reinforced concrete.

Corinaldesi et al (2003) studied fibre reinforced self compacting concrete by casting thin Precast elements. In their experiments they used homogeneously dispersed steel fibres instead of ordinary steel reinforcing mesh. Steel fibres were added to the concrete mixture at a dosage of 10% by mass of cement and water cement ratio was kept as .40. They carried out compression and flexure tests to assess the safety of thin precast elements. Compression tests according to Italian Standards UNI 6132-72 were carried out on cubic specimens .The specimens were loaded at a constant strain rate until failure. Tensile strength of concrete was measured by subjecting the concrete beam (100 x 100 x 500 mm) to flexure. Their results as shown in figure 2.1 indicated that compressive strength level of 40 MPa was achieved at 28 days of curing. However in case of flexure strength the contribution of the steel fibres did not appear to be evident due to low dosage of fibres employed.

Gunasekaran (2002) conducted tests to investigate the flexural strength and load defection behavior of light weight concrete beams (150 mm x 150 mm x 900 mm) made with sintered fly ash aggregates and regulated set cement and included steel fibre reinforcement. Three different aspect ratios of about 47, 50 and 63 were used for the fibres. It was found that beams containing fibres with an aspect ratio of 50 had the best flexural strength 34.5kg/cm², but the beams containing fibres with an aspect ratio of 62.5 had better ductility, although lower flexural strength, 25.3 kg/cm². In both cases the quantity of fly ash used was the same. Beam made with concrete having the same total quantity of fibre as before, but comprising 50 percent fibres with an aspect ratio of 62.5, possessed considerable ductility without any reduction in flexural strength. They have concluded that for equal quantities of fibre reinforcement, a blend of fibres having both long and short fibres result in greater structural benefits in concrete than identical fibres with a high aspect ratio, and low aspect ratio act as crack arrestors in the finite volume enclosed by the high aspect ratio fibres, the latter are primarily responsible for enhanced ductility of fibre reinforced concrete.

Nanni et al (1992) conducted an investigation on the use of newly developed aramid fibres for the reinforcement of Portland cement based concrete. The aramid fibres were produced in chopping a bundle made of epoxy-impregnated braided into aramid filaments. In this investigation, the behavior of reinforced concrete of aramid fibres was compared to steel fibres and polypropylene fibres. Beams of 100 x 100 x 350mm were tested under four point flexural loading. It was found that aramid fibres acts similar to steel fibres and is superior to polypropylene fibres. They concluded aramid fibres were lack in corrosion problems while having a higher performance than polypropylene fibres. However, the use of aramid fibres was not very economical.

Sahmaran and yaman(2002) carried out investigations on fresh and mechanical properties of fibre reinforced concrete self compacting concrete incorporating high volume flu ash concrete that does not meet the fineness requirement of ASTM C 618 . Fibre inclusion to concrete enhances the mechanical properties, while making the concrete less workable. A polycarboxylic based superplasticizer was used in combination with a viscosity modifying admixture. In mixes containing fly ash, 50% of cement by weight was replaced with fly ash. Two different types of steel fibres were used in combination keeping total fibre content constant at 60 kg/m^3 . Slump Flow time ,V funnel and air content tests were performed to assess the fresh properties of the concrete. Compressive, split tensile strength and ultrasonic pulse velocity of the concrete were determined for the hardened properties. It can be concluded that high volume coarse fly ash could successfully be used in producing fibre reinforced self compacting concrete. Even though there is some reduction in concrete strength because of the use of high volume coarse fly ash it is possible to achieve self compaction with considerable fibre inclusion.

Siddique R. reported the results of an experimental investigation to study the effects of replacement of cement (by mass) with three percentages of fly ash and the effect of addition of natural san fibres on the slump, Vebe time, compressive strength ,splitting tensile strength, flexural strength and impact strength of fly ash concrete .San fibres belong to the category of natural blast fibres This class of natural fibre is mostly grown in the Indian subcontinent, Brazil, eastern and southern Africa and some parts of the United States. A control mixture of proportions 1:1.4:2.19 with W/C ratio of .47 and superplasticizer/cementitious ratio of .015 was designed. Cement was replaced with three percentages (35%, 45%, and 55%) of class F fly ash. Three percentages of san fibres (.25%, .5%and .75%) having25mm length were used. The test results indicated that replacement of cement with fly ash increased the workability, decreased compressive strength, splitting tensile strength and flexural strength and had no significant effect on the impact strength of plain concrete. Addition of san fibres reduced the workability, did not significantly affect the compressive strength, increased the splitting tensile strength and flexural strength and enhanced the impact strength of fly ash concrete as the percentage of fibres increased.

Siddique R reported the results of an experimental investigation dealing with concrete incorporating high volumes of class F fly ash. Portland cement was replaced with three percentages (40%, 45% and 50%) of class F fly ash. Tests were performed for fresh concrete properties (Slump, unit weight and temperature). Compressive, splitting and flexural strengths, modulus of elasticity were determined up to 365 days of testing. Test results indicated that the use of high volumes of class F fly ash as a partial replacement of cement in concrete decreased its 28 day compressive, splitting tensile and flexural strengths, modulus of elasticity. However all these strength properties showed significant improvement at the ages of 91 and 365 days, which was most probably due to the Pozzolanic reaction of fly ash. Based on results it was concluded that Class F fly ash could be suitably used up to 50 % level of cement replacement in concrete for use in precast elements and reinforced cement concrete construction.

Song et al (2004) investigated the strength potential of nylon fibre reinforced concrete versus that of polypropylene fibre reinforced concrete at a fibre content of .6 kg/m^3. The strength properties studied includes compressive strength and splitting tensile strength. Their results indicated that compressive and splitting tensile strength of the nylon fibre reinforced concrete improved by 6.3% and 6.7% respectively which is attributed to the fact that nylon fibres have high tensile strength and better distribution through the concrete mass.

Soroushian et al(2000) studied "Optimum use of Pozzolanic materials in steel fiber reinforced concrete" . The effects on fresh and hardened material properties for fly ash caused by substituting cement with fly ash and silica fume in steel fiber reinforced concrete were studied experimentally. The percentage substitution of cement ranged from 0 to 40% and from 0 to 20% for silica fume. The workability of fresh fibrous mixtures was characterized by measuring the inverted slump cone time. The hardened material was tested at 28 days under compression and flexural loads. The development of compressive strength with time was also assessed in steel fiber reinforced concrete incorporating fly ash. The generated test data were used to decide the optimum ranges of cement substitution with fly ash or silica fume in steel fiber reinforced concrete for achieving desirable fresh mix and hardened material characteristics.

Bilodeau et al (2001) reported the results of investigations carried out to determine the various durability aspects of high volume fly ash concrete using eight fly ashes and two Portland cements from U.S sources. The durability aspects investigated included resistance to the repeated cycles of freezing and thawing, the resistance to the chloride ion penetration and determination of water permeability coefficient. Based on the rest results that regardless of type of fly ash and the cements used the air entrained high volume fly ash concrete exhibited excellent durability characteristics in the tests investigated

2.4.3 Effect of Both fly ash and fibres

Malhotra et al (2000) studied polypropylene fibre reinforced high volume fly ash concrete having a low water to cementitious material ratio , fly ash content greater than 50 percent of the cementitious material, and contents of fibres up to 5 kg/m ^{3} of concrete. The workability of the concrete is maintained by the use of high dosages of superplasticizers. By their studies it was concluded that polypropylene fibre reinforced high volume fly ash concrete has satisfactory workability and strength characteristics. It was also concluded that polypropylene fibre reinforced high volume fly ash concrete has very low permeability, low drying shrinkage, adequate ductility and toughness characteristics.

Patton and Whittaker (1983) investigated on the steel fibre content for dependence of modulus of elasticity and correlation changes on damage due to load. They found out that there was approximately 3.3 percent increase over the modulus of elasticity of plain concrete for every 1.0% increase in fibre content by volume. Furthermore, the investigation shows that degeneration of stiffness starts at approximately 30 percent of the ultimate load before the first visible crack appears.

Trottier et al (1994) investigated the toughness of fibre reinforced concrete by using different geometry of steel fibres, which include hooked end, crimped circular, crimped crescent and twin cone end steel fibres. One fibre volume fraction (40kg/m ^{3}) was used throughout the research. The tests include compressive strength test and flexural strength test, with measurement of deformation of specimen as the load applied. They found out that fibres brought significant improvement in the toughness and energy absorption capacity of concrete. Based on four fibre geometries, fibres with deformations only at end appear more effective than those with deformations over the entire length.

Zhang et al(1999) investigated the materials used ,mixture proportions ,mixing and shotcreting operation and properties of the fresh and hardened polypropylene fibre reinforced shotcrete incorporating silica fume and high volumes of fly ash. The polypropylene fibre reinforced high volume fly ash shotcrete produced had satisfactory workability, mechanical properties and resistance to freezing and thawing cycling. The shotcrete containing silica fume had negligible rebound compared with that without silica fume. The incorporation of fly ash and silica fume improved the workability of the fresh shotcrete and this resulted in lower operation pressure for the shotcreting. The use of polypropylene fibres upto .5% by the volume of the shotcrete did not affect significantly the compressive strength and the shotcrete incorporating both fly ash and silica fume bonded well to the base concrete. The fibre –reinforced shotcrete showed satisfactory performance after 300 cycles of freezing and thawing with a durability factor >80 even though the air contents were relatively low and the spacing factor was relatively high

Table 2.1 Statistical variations of compressive and tensile strengths (Mirza et al, 1979)

Property	Mean, psi	V
Concrete normal control		
Compressive Strength in structure loaded to failure in one hour		
f'$_C$=3000 psi	2760	.18
= 4000psi	3390	.18
= 5000psi	4028	.15
Tensile Strength in structure loaded to failure in one hour		
f'$_C$=3000 psi	306	.18
= 4000psi	339	.18
= 5000psi	366	.18

Table 2.2 Control standards for evaluating performance of testing program using the within-test coefficient of variation (ACI report, 1988)

Field control testing				
Control standards for within-test coefficient of variation				
Excellent	Very good	Good	Fair	Average
Below 1.5	1.5 to 2.0	2.0 to 3.0	3.0 to 4.0	Above 4.0

Table 2.3 Results of statistical analysis of cube strength of concrete (Ranganathan 1990)

Source	**Grade**	**Specified strength, MPa**	**Mean, MPa**	**Standard deviation , Mpa**	**COV per cent**	**Distribution**	**Quality Control**
IIT Kanpur	M15	15	24.03	5.76	23.96	Lognormal	Nominal Mix
IIT Kanpur	M20	20	29.16	5.49	18.83	Normal	Nominal Mix
IIT Kanpur	M25	25	30.28	3.77	12.45	Normal	Designed Mix
IIT Kanpur	M35	35	42.28	5.60	13.24	Normal	Designed Mix
REC Calicut	M15	15	22.67	5.01	22.10	Lognormal	Nominal Mix
IIT Bombay	M15	15	17.56	2.69	15.33	Lognormal	Designed Mix
IIT Bombay	M20	20	26.8	4.04	15.07	Normal, Lognormal	Designed Mix

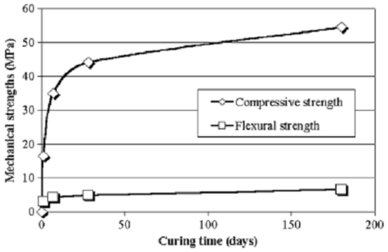


Fig 2.1 **Compressive and flexure strengths of concrete as a function of curing time (*Corinaldesi et al. (2003)***

CHAPTER 3 EXPERIMENTAL PROGRAMME

3.1 GENERAL

The main objective of test programme is to study strength characteristics of concrete with replacement of cement with high volume of fly ash and incorporation of Recron 3s fibres .The main parameters that were studied includes compressive strength and splitting tensile strength. The materials used for casting fibre concrete samples along with the tested results are described as under.

3.2 MATERIAL USED

3.2.1 Cement

Cement is a fine, grey powder. It is mixed with water and materials such as sand, gravel, and crushed stone to make concrete. The cement and water form a paste that binds the other materials together as the concrete hardens. The ordinary cement contains two basic ingredients namely argillaceous and calcareous. In argillaceous materials clay predominates and in calcareous materials calcium carbonate predominates. The Basic composition of cement is provided in table 3.1 .In the present work 53 grade Ultra Tech cement was used for casting cubes and cylinders for all concrete mixes. The cement was uniform colour i.e. grey with a light greenish shade and was free from any hard lumps. Summary of the results of various tests conducted on cement is provided in Table 3.2

3.2.2 Fine Aggregates

The material which passes through BIS test sieve no. 480 is termed as fine aggregate. Usually natural sand is used as a fine aggregate at places where natural sand is not available crushed stone is used as a fine aggregate.

The sand used for the experimental works was locally procured and conformed to grading zone III. Sieve Analysis of the Fine Aggregate was carried out in the laboratory as per IS 383-1970 and results are provided in Table 3.3 The sand was first sieved through 4.75mm sieve to remove any particle greater than 4.75 mm sieve and then was washed to remove the dust. The various results of testing carried out for fine aggregates is provided in table 3.4

3.2.3 Coarse aggregates

The material which is retained on BIS test sieve no. 480 is termed as a coarse aggregate. The broken stone is generally used as a coarse aggregate. The nature of work decides the maximum size of the coarse aggregate. Locally available coarse aggregate having the maximum size of 20 mm was used in the present work. Sieve analysis of coarse aggregates used was carried out and results are provided in table 3.5. The physical properties of coarse aggregates used are provided in table 3.6.

3.2.4 Fly ash

Fly ash is a fairly divided residue which results from the combustion of ground or powdered bituminous coal or sub bituminous coal like lignite. It is a by product of many thermal power stations. Fly ash resembles pozzolana i.e. a substance which although not cementitious itself contains constituents which combine with lime to form a material having cementitious properties. It is acidic in nature and its main constituents are silica, aluminium oxide and ferrous oxide. The fly ash used in the present study is taken from Guru Gobind Singh Thermal Plant, at Ropar. The Physical and chemical properties of the fly ash used are reported here for ready reference as obtained from GGS thermal plant. (See Table 3.7)

3.2.5 Water

Water is an important ingredient of concrete as it actively participates in the chemical reaction with cement. Since it helps to form the strength giving cement gel, the quantity and quality of water is required to be looked into very carefully. Potable water is generally considered satisfactory. In the present investigation, tap water was used for both mixing and curing purposes.

3.2.6 Fibres

Recron 3s fibre (CT-2024) is a fibre developed after extensive research at Reliance Technology Centre.CT-2024 is monofilament fibre designed specially to provide integral secondary reinforcement of concrete. Recron 3s fibres are Polyester staple fibres mainly used for mixing in concrete and mortar for improving certain properties of the concrete and mortar. The Fibres have special triangular shape for better anchoring with other ingredients of the mix. The fibres are made from polymerization of pure teraphthalic acid and Mono Ethylene Glycol using catalyst. Recron 3s fibres are available in 6mm and 12mm length. In our research work fibre dosage was fixed as .5% per bag of cement. The properties of Recron 3s Fibres are provided in Table 3.8 and figure of fibres is shown in 3.1

3.2.7Superplasticizer

Sika ViscoCrete -SC 001, the superplasticizer supplied by Sika India Pvt. Limited was used in our investigations. It is a third generation highly effective superplasticizer for concrete and mortar. It meets the requirements for superplasticizer according to EN934 -2, SIA 262, ASTM C 494-99/99a Type F and 9103-1999 (amended 2003). The dosage of the superplasticizer was fixed based on the requirements for workability. The technical data related to the superplasticizer used is provided in Table 3.9.

3.3 TESTS CARRIED OUT TO STUDY PROPERTIES OF FRESH CONCRETE:

3.3.1 Workability:

The term workability is hard to define precisely, and ***Newman (1965)*** has proposed that it can define in three separate properties:

1. *Compatibility*. This means the ‘ease and ability with which the concrete can be compacted and the air voids are removed’ ***(Murdock et. al, 1968)***.
2. *Mobility*. It terms as the ‘ease and ability with which the concrete can pour into the moulds, around the steel and be remolded’ ***(Murdock et. al, 1968)***.
3. *Stability*. It is the ability of the ‘concrete to stay a stable coherent homogenous mass while handling and vibration’ ***(Murdock et. al, 1968)*** without the constituents segregating.

Cement Association of Canada (2004) stated that workability could be defined as the ease of placing, consolidating, and finishing fresh mixed concrete and the degree to which it resists segregation. Concrete must be workable but the constituents should not be separate during transport and handling. Properties such as consistency, segregation, mobility, bleeding, and finish ability are related with workability'. Slump test is used to measure consistency of a concrete, which have a close indication to workability. A low slump concrete has a stiff consistency. Concrete will be difficult to place when the consistency is too dry and harsh. It may results that large aggregate particles may separate from the concrete mix. However, more workable mix does not necessarily means a more fluid mix. Segregation and honeycombing can occurred if the mix is too fluid.

Indications show that concrete mix with addition of fibres may have a stiff effect. Therefore, slump test is not recommended as the only test used to measure the workability of fibre-reinforced concrete. Thus compaction factor test is used to find out the workability of fibre reinforced concrete mix and is explained in the following section

3.3.2Compaction Factor Test:

This test is particularly used for concrete mixes of very low workability. The compacting factor was determined from the following equation:

$$\text{Compacting factor} = \frac{\text{Mass of partially compacted concrete } (M_1)}{\text{Mass of fully compacted concrete } (M_2)}$$

The values of compaction factor for control as well as fibre reinforced concrete mixes are provided in Tables 3.10 and 3.11 respectively. Compaction factor test apparatus is sown in figure 3.2

3.4 MIX DESIGN

Mix design is known as the selection of mix ingredients and their proportions required in a concrete mix. There are several methods of mix design used throughout the world. Eventually, all of these methods follow the same procedure and produce similar results. In India the most commonly used method for mix design is the Indian Standard Method. The mix design involves calculations of the amount of cement, fine aggregate and coarse aggregate in addition to other related parameters. The mix design calculations are dependent on the properties of the constituent materials.

In the present study for designing the mix target mean strength is not fixed rather water cement ratios were fixed. The various water cement ratios taken into consideration are 0.40, 0.42, 0.44, 0.46, 0.48. By fixing the water cement ratios all the required quantities were calculated as per Indian standards after making all the calculations and necessary adjustments the quantities of coarse aggregate, fine aggregates as obtained are detailed in Table 3.12(a). The details of the modifications made and quantities of constituent materials used to cast Fly ash Fibre concrete mixes are provided in Table 3.12(b)

3.5 PREPARATION OF SPECIMENS FOR TESTING

Cylindrical mould of size 150 mm×300 mm, and cubes of size150mm x150mmx150mm were used to prepare the concrete specimens for the determinations of compressive strength, split tensile strength of fibrous fly ash concrete. All specimens were prepared in accordance with Indian Standard Specifications IS: 516 -1959. All the moulds were cleaned and oiled properly. These were securely tightened to correct dimensions before casting. Care was taken that there are no gaps left from where there is any possibility of leakage out of slurry. Concrete cylinders, 150 mm×300 mm were tested for the determinations of split tensile strength and cubes 150mm x150mmx150mm were used for obtaining the compressive strength of control as well as fibrous fly ash concrete as per Indian Standard Specifications IS: 516-1959

3.6 BATCHING, MIXING AND CASTING OF SPECIMENS

Careful procedure was adopted in the batching, mixing and casting operations. The coarse aggregates and fine aggregates were weighed first with an accuracy of 0.5 grams. The concrete mixture was prepared by hand mixing on a watertight platform. The fly ash and cement were mixed dry to uniform colour separately. Superplasticizer as per requirement of workability (Medium) was added to required quantity of water separately in different containers. On the watertight platform, the coarse and fine aggregates were mixed thoroughly. To this mixture, the required quantity of cement, fly ash and fibres (Fly ash 50 % and 60% by weight of cement and fibres .5%) were added. These were mixed to uniform colour. Then water was added carefully so that no water was lost during mixing.

Eighteen clean and oiled moulds for each category were then placed on the vibrating table respectively and filled in three layers. Vibrations were stopped as soon as the cement slurry appeared on the top surface of the mould. The specimens were allowed to remain in the steel mould for the first 24 hours under ambient condition. After that, these were demoulded with care so that no edges were broken and were placed in the curing tank at the ambient temperature for curing. The ambient temperature for curing was $27 \pm 2\ ^0$ C.

3.7 TESTING OF SPECIMENS

The specimens, after a fixed curing period of 7 days, 14 days and 28 days were tested for compressive as well as split tensile strength on an automatic compression testing machine (3000kN capacity) (Plates - 3.1 and 3.2) as per procedure laid down in IS 516:1959. A uniform pace rate of 5.25 kN/sec was maintained through out the testing of each specimen.

3.8 RESULTS OF STRENGTH TESTS

The results of the compressive strengths of 150 mm cubes are listed in Tables 3.13 to 3.15 for each mix in the form of means, standard deviations and within-test coefficient of variations and results of split tensile strength tests are listed in table 3.16 to 3.18. The within-test coefficient of variation provided in the tables has been calculated using Eq.3.1 (ACI 214 – 1988)

$$V_{wt} = \frac{S_{wt}}{\mu} * 100 \qquad 3.1$$

where V_{wt} is within-test coefficient of variation expressed as per cent, and t is the average strength for the class of concrete in MPa and S_{wt} is the within-test standard deviation given by Eq.3.2.

$$S_{wt} = \frac{R}{d_2} \qquad 3.2$$

Where R is the average range for all tests of a class of concrete and is the difference between the highest and lowest strengths of the cubes or cylinders making up a test, and d_2 is a factor based on number of samples within the test. The value of d_2 equals 1.128 if two cubes are used in the test and it equals 1.693 for three cubes, 2.059 for four cubes and 2.398 for six cubes. The experimental data so generated was further probabilistically analysed and has been subsequently used in the evaluation of partial safety factors for different concrete strengths corresponding to varying reliability indices. The results of the analysis have been provided in chapter 4.

Table: 3.1 Composition limits of Portland cement

(Reference: Book concrete technology M.S Shetty)

Ingredient	Per cent ,Content
CaO(Lime)	60-67
SiO_2 (Silica)	17-25
Al_2O_3(Alumina)	3-8
Fe_2O_3 (Iron Oxide)	0.5-6
MgO(Magnesia)	0.1-4
Alkalies	.4-1.3
Sulphur	1-3

Table 3.2 Properties of cement

S.No.	Characteristics	Values obtained	Standard values
1.	Normal Consistency	33%	-
2.	Initial Setting time	2 hours 10 min	Not to be less than 30 minutes
3.	Final Setting time	6 hour 25 min	Not to be greater than 600 minutes
4.	Fineness	4%	<10%
5.	Specific gravity	3.15	-
Compressive strength(MPa)			
1.	3 days	33.2	33
2.	7 days	42.5	43
3.	28 days	53.5	53

Table 3.3 Sieve Analysis of fine aggregates

S.No.	**Sieve No.**	**Weight Retained (gms)**	**Percentage Retained%**	**Percentage Passing%**	**Cumulative % Retained**
1.	4.75mm	95	9.5	90.5	9.5
2.	2.36mm	42.5	4.25	86.25	13.75
3.	1.18mm	110.5	11.05	75.2	24.8
4.	600µm	128.5	12.85	62.35	37.65
5.	300µm	308.0	30.8	31.55	68.45
6.	150µm	281.0	28.1	3.45	96.55
7.	Pan	34.5	3.45	----	----
				ΣF=	250.5

Fineness Modulus of fine aggregates= ΣF/100

= 250.5/100= 2.505

Table 3.4 Physical properties of fine aggregates

S.NO.	**Characteristics**	**Value**
1.	Type	Natural Sand
2.	Specific Gravity	2.65
3.	Fineness Modulus	2.505
4.	Grading Zone	III

Table 3.5 Sieve Analysis of coarse aggregates

S.No.	Sieve No.	Weight Retained (gms)	Percentage Retained	Percentage Passing	Cumulative Retained
1.	80 mm	-----	0.00	100	0.00
2.	40 mm	-----	0.00	100	0.00
3.	20 mm	0	0.00	100	0.00
4.	12.5 mm	2186.5	72.883	27.117	72.883
5.	10 mm	674.5	22.483	4.634	95.336
6.	4.75 mm	139.0	4.633	0.01	99.999
7.	Pan	0	0.00	-----	------
				ΣC=	268.24

Fineness Modulus of coarse aggregates (20mm) = ΣC+500/100=268.24+500/100=7.68

Table 3.6 Properties of coarse aggregates

S.NO.	Characteristics	Value
1.	Type	Crushed
2.	Specific Gravity	2.61
3.	Water absorption	2.37%
4.	Fineness Modulus	7.68
5.	Maximum Size	20 mm

Table3.7 Chemical Properties of fly ash under consideration

Sr. No.	Properties	
1.	Silica (SiO_2)	45 to 89%
2.	Alumina (Al_2 O_3)	23 to 33%
3.	Ferric Oxide(Fe_2O_3)	0.6 to 0.4%
4.	Titanium (Ti O_2)	0.5 to 16%
5.	Calcium Oxide (CaO)	5 to 16%
6.	Magnesia (MgO)	1.5 TO 5%
7.	Sulphuric Anhydride as SO_3	2.5%
8.	Loss of ignition	1.0 to 2.0%

Table 3.8 Properties of Recron 3s fibres (CT 2024)

Reference: Chart provided by Reliance industries for Recron 3s fibres o n their website

S.No.	Specifications	Value
1.	Material	100%Virigin Polyester
2.	Filament Diameter	30-35 microns
3.	Cut Length	12mm
4.	Tensile strength	~600kg/cm^2
5.	Melting point	>250°C
6.	Dispersion	Excellent
7.	Acid resistance	Good
8.	Alkali resistance	Good
9.	Electrical Conductivity	Low
10.	Thermal Conductivity	Low

Table 3.9: Technical Data of Superplasticizer

Reference: Literature provided with Superplasticizer

S.NO.	Characteristics	Value
1.	Colour	Dark brown liquid
2.	Specific gravity	1.17
3.	Air Entrainment	Maximum 1%
4.	pH	7to8

Table 3.10: Compaction factor for the controlled mix

S.No.	W/C ratio	Compaction factor
1.	0.40	0.799
2.	0.42	0.879
3.	0.44	0.882
4.	0.46	0.881
5.	0.48	0.880

Table 3.11: Compaction factor for the mix with fibres and fly ash

S.No.	W/C ratio	Compaction factor
1.	0.40	0.800
2.	0.42	0.8702
3.	0.44	0.8703
4.	0.46	0.8721
5.	0.48	0.8723

3.12 (a) Mix proportions for various water cement ratios under consideration for control mix

W/C Ratio	Cement(kg/m^3)	Coarse Aggregates(kg/m^3)	Fine aggregates(kg/m^3)	Water (kg/m^3)	Ratio
0.40	476.25	1217.20	509.20	190.50	1:1.06:2.55
0.42	454.00	1224.00	515.60	190.50	1:1.13:2.69
0.44	433.00	1237.75	521.10	190.50	1:1.20:2.85
0.46	414.10	1250.00	526.00	190.50	1:1.27:3.01
0.48	396.87	1260.00	530.45	190.50	1:1.33:3.17

3.12(b) Mix proportions for various water cement ratios for 50% Fly ash

W/C Ratio	Fly ash (kg/m^3)	Cement (kg/m^3)	Coarse Aggregates (kg/m^3)	Fine aggregates (kg/m^3)	Water (kg/m^3)	Fibre %	Super Plasticizer (kg/m^3)
0.40	238.12	238.12	1217.20	509.20	190.50	0.5	0.95
0.42	227.00	227.00	1224.00	515.60	190.50	0.5	0.90
0.44	216.50	216.50	1237.75	521.10	190.50	0.5	0.80
0.46	207.05	207.05	1250.00	526.00	190.50	0.5	0.86
0.48	198.43	198.43	1260.00	530.45	190.50	0.5	0.79

3.12 (c) Mix proportions for various water cement ratios for 60% Fly ash

W/C Ratio	Fly ash (kg/m^3)	Cement (kg/m^3)	Coarse Aggregates (kg/m^3)	Fine aggregates (kg/m^3)	Water (kg/m^3)	Fibre %	Super Plasticizer (kg/m^3)
0.40	285.75	190.5	1217.20	509.20	190.50	0.5	0.95
0.42	272.40	181.60	1224.00	515.60	190.50	0.5	0.90
0.44	259.80	173.20	1237.75	521.10	190.50	0.5	0.80
0.46	248.47	165.65	1250.00	526.00	190.50	0.5	0.86
0.48	238.12	158.74	1260.00	530.45	190.50	0.5	0.79

Table 3.13 Compressive strength data for concrete mixes without replacement of cement by Fly ash

W/C Ratio	7 days curing			14 days curing			28 days curing		
	Mean MPa	S.D., MPa	Withintest COV	Men MPa	S.D., MPa	Withintest COV	Mean MPa	S.D., MPa	Witintest COV
0.40	29.0	1.26	1.43	39.0	1.27	1.07	40.0	1.67	1.47
0.42	28.0	1.27	2.97	38.0	1.26	3.29	39.5	1.32	1.49
0.44	27.3	1.75	1.53	37.3	2.03	2.23	38.6	2.98	1.52
0.46	26.6	1.32	3.13	34.6	1.46	2.41	37.6	0.83	1.57
0.48	24.3	0.96	3.45	33.3	0.73	2.50	35.3	1.32	1.67

Table 3.14 Compressive strength data for concrete mixes with 50% replacement of cement by Fly ash

W/C Ratio	7 days curing			14 days curing			28 days curing		
	Mean MPa	S.D., MPa	Withintest COV	Men MPa	S.D., MPa	Withintest COV	Mean MPa	S.D., MPa	Witintest COV
0.40	21.30	1.78	1.44	22.0	1.33	5.68	28.0	1.96	1.5
0.42	20.30	0.51	1.50	21.6	1.22	5.76	26.5	5.05	2.91
0.44	18.67	1.22	4.50	20.6	0.52	2.03	23.3	2.25	1.07
0.46	16.30	1.36	1.27	17.8	0.41	2.35	20.67	1.5	6.04
0.48	10.17	0.76	1.72	12.5	1.05	1.07	15.16	0.75	5.51

Table 3.15 Compressive strength data for concrete mixes with 60% replacement of cement by Fly ash

W/C Ratio	7 days curing			14 days curing			28 days curing		
	Mean MPa	S.D., MPa	With-intest COV	Men MPa	S.D., MPa	With-intest COV	Mean MPa	S.D., MPa	With intest COV
0.40	9.30	0.82	8.97	12.0	1.22	1.04	18.5	2.32	1.35
0.42	6.67	0.44	6.29	11.3	0.56	7.38	17.5	2.06	1.19
0.44	4.50	0.55	9.33	7.33	0.52	5.73	10.3	1.37	1.69
0.46	4.67	0.52	9.08	7.67	0.52	5.48	11.5	1.04	1.52
0.48	5.83	1.17	2.14	8.83	0.75	9.49	12.5	1.38	4.17

RESULTS OF SPLITTING TENSILE STRENGTH TESTS

Table 3.16 Splitting strength data for concrete mixes without replacement of cement by Fly ash

W/C Ratio	28 days curing		
	Mean MPa	S.D., MPa	With-intest COV
0.40	3.22	0.24	1.83
0.42	3.50	0.37	1.68
0.44	2.98	0.21	1.98
0.46	3.64	0.28	1.62
0.48	3.73	0.25	1.58

Table 3.17 Splitting strength data for concrete mixes with 50% replacement of cement by Fly ash

W/C Ratio	28 days curing		
	Mean MPa	S.D., MPa	With-intest COV
0.40	1.6	0.52	3.69
0.42	2.3	0.50	2.56
0.44	1.9	0.24	3.10
0.46	1.4	0.47	4.21
0.48	1.5	0.15	3.93

Table 3.18 Splitting strength data for concrete mixes with 60% replacement of cement by Fly ash

W/C Ratio	28 days curing		
	Mean MPa	S.D., MPa	With-intest COV
0.40	1.50	0.52	3.69
0.42	1.63	0.50	2.59
0.44	1.79	0.24	3.20
0.46	1.37	0.47	4.67
0.48	1.17	0.15	3.97

Figure 3.1 Recron 3s Fibres

Figure 3.2 Compaction factor Test Apparatus
(Source: Indian Standard, 2002)

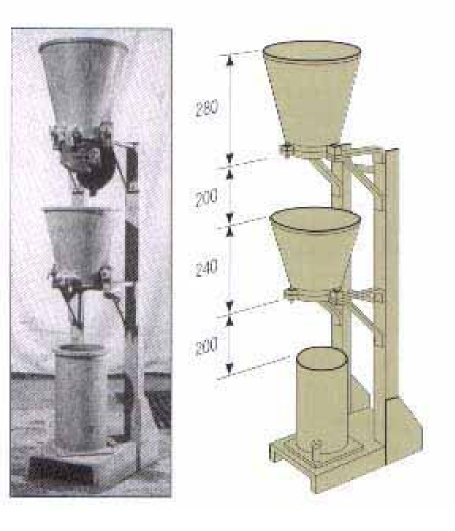


Plate 3.1 Photograph of automatic compression testing machine

Plate 3.2 Photograph Showing Splitting Tensile strength

CHAPTER 4 MATHEMATICAL MODEL FOR DEVELOPMENT OF RELIABILITY - BASED DESIGN CRITERIA

4.1 GENERAL

It is generally recognized that uncertainty exists in the results of design formulae for evaluating the resistance of structural elements. One of the factors contributing to this uncertainty is the variability of the strengths of the constituent materials. The other contributing factors are the variations in loads coming on the structural elements and cross- sectional dimensions. The mathematical model used in the succeeding section, used for the evaluation of partial safety factors corresponding to varying reliability indices, for concrete mixes, takes into account the variations in the water -cementitious ratio and percentage replacement of cement by Fly ash .The content of fibre used in all mixes is constant as detailed in previous chapter.

In general the development of a reliability-based design involves the following steps:

a) Collection and statistical analysis of the data on basic variables.

b) Defining the probability distributions of each variable – at least in terms of mean, standard deviation and conforming probability distribution.

c) Study of the resistance of elements and establishing their statistical characteristics.

d) Selection of the target reliability index, β, i.e . accepted or specified level of reliability.

e) Determination of the partial safety factors for the desired uniform reliability index, β, under all design situations within the scope of work.

The present study aims at finding partial safety factors using reliability based procedure for concrete mixes of desired strength at a desired curing age for a particular reliability index. An extensive data bank on the basic variables viz, compressive strength of concrete in terms of mean, standard deviation and within-test coefficient of variation corresponding to variations in water-cementitious material ratio, 50 and 60 per cent replacement of cement by Fly ash , curing age and fixed percentage of fibres etc. has been generated experimentally. This Experimental data has been used for the development of partial safety factors using reliability based procedure.

4.2 STATISTICAL STUDY OF COMPRESSIVE STRENGTH

4.2.1 Basic Variables and Failure Surface

In a structural design problem, several parameters viz, geometric properties of the section, physical properties of the materials and loads coming on the structure are subjected to random variations. If the coefficient of variation of a random variable is very small, the variations may be ignored and the parameter may be considered as deterministic. Thus the parameters in any design problem, which are to be considered as random variables, are initially fixed, and those are called basic variables. An equation that is developed for a particular failure condition shall be a function of these basic variables say $X_1, X_2, X_3,\ldots\ldots\ldots\ldots X_n$.

Consider a failure function $g(X_1, X_2, X_3,\ldots\ldots\ldots\ldots X_n)$. The margin of safety, M, for this failure function can be represented by

$$M = R - S \tag{4.1}$$

where R and S represent the internal resistance and external action respectively, which are also functions of the basic variables $X_1, X_2, X_3,\ldots\ldots\ldots\ldots X_n$.

Hence,

$$M = g(X_1, X_2, X_3,\ldots\ldots\ldots\ldots X_n) \tag{4.2}$$

When this failure function is reduced to zero i.e. $g(X_1, X_2, X_3 \ldots Xn) = 0$ it is called as failure surface. A degree of safety is insured by specifying a small value for the probability of reaching a particular failure state. The magnitude assigned depends on the serviceability consequences of reaching the particular failure state. If $f_x\ (x)$ is the probability density function of the jointly distributed basic variables $X_1, X_2, X_3,\ldots\ldots\ldots\ldots X_n$, then the probability of failure is

$$p_f = \iiint\limits_{g<0} \ldots\ldots \int f(x)dx \qquad 4.3$$

The multiple integral is to be evaluated over the region g<0. The failure surface equation divides the space into two regions, viz. (i) safe and (ii) unsafe or failure region.

In the present study the effect of variations in water-cementitious material ratio, 50 and 60 per cent replacement of cement by Fly ash and curing age etc. have been studied on the compressive strength of concrete. As the compressive strength of concrete (f_k) is directly related to the water cementitious material ratio $\left(\frac{w}{c+p}\right)$ the failure function can be written as $g\left(f_k + {}^{w}\!/_{c+p}\right)$ and is shown in Fig 4.1.

It is convenient to have a mathematical function (probability distribution function or PDF) to describe the random variables viz, compressive strength of concrete and water-cementitious material ratio. The choice of a distribution function has been based upon the accuracy and ease with which it can represent a given set of empirical data. The compressive strength data generated experimentally has been analysed using normal-probability distribution function. The normal distribution is the most extensively used, as many natural phenomenon seem too governed by this distribution. This probability distribution function as used in the present study is briefly explained below

(a) Normal distribution

If a random phenomenon (variable) arises because of several factors, which act in an additive manner to result in the phenomenon, then the model arising out of such a situation will be a normal distribution ***(Ranganathan, 1990)***. This distribution is known as Gaussian distribution. The PDF of normal variate is given by

$$f_x(x) = \frac{1}{\sigma\sqrt{2\Pi}} \exp\left[-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right] \qquad 4.4$$

where the parameter σ and μ represents the mean and standard of the distribution, respectively. The normal distribution can also be designated as N (μ, σ). A normal distribution with parameters μ = 0 and σ = 1 is called a standard normal distribution and is designated as N (0, 1). Thus, the PDF of standard normal variate is given by

$$f_U(u) = \frac{1}{\sqrt{2\Pi}} \exp\left[-\frac{1}{2}u^2\right] \qquad 4.5$$

The cumulative probability of standard normal variate that is $\Phi(u) = F_u(u) = P(U \le u)$ is shown along with the PDF in Fig 4.2 ***(Ranganathan 1990)***

From the Fig. 4.2 cumulative probability can be expressed as

$$\Phi(u_1) = p_1 \qquad 4.6$$

Conversely the value of u_1 at a cumulative probability p_1 is given by

$$u_1 = \Phi^{-1}(p_1) \qquad 4.7$$

4.2.2 Reliability Index

If the failure function is represented by Eq. (4.1) then the basic variables can be normalized using the relationship

$$Z_i = \frac{Xi - \mu i}{\sigma i} \qquad \text{i=1,2,……………..n} \qquad 4.8$$

In the z coordinate system, the failure surface is a function of z_i. Equating the failure function to zero, the failure surface equation is obtained which can be written in the normalised coordinate system. This failure surface divides the design sample space into two regions namely safe and unsafe or failure regions.

Because of the normalization of the basic variable μ_{z_1} =0 and σ_Z =1 the z coordinate system has a rotational symmetry with respect to the standard deviation and the origin O will usually lie in the safe region. From the 2-D representation shown in Fig. 4.3. It is evident that as the failure surface $g(Z_1,Z_2)$ moves away from the origin . The reliability $g(Z)>0$ increases and as it moves closer to the origin , reliability decreases. Hence the position of failure surface with respect to the origin in normalized coordinate system determines the magnitude of reliability.

Hasofer and Lind (1974) defined the reliability index β as the shortest distance from the origin to failure surface in the normalized coordinate system. The point D on the failure surface in the Fig. 4.3 is called the design point. This point is called the checkpoint for the safety of structure. The reliability index β is related to the failure surface and not to the failure functions. The reliability index, $\beta=\frac{\mu_M}{\sigma_M}$ defined by ***Cornell (1969)***, will coincide with the value obtained by Hasofer and Lind when the failure surface is a linear function of basic variables. Thus for the Hasofer-Lind method the relation between reliability index and probability of failure as described by Eq. (4.9) can be used provided the failure function is a linear function of the normally distributed basic variables

$$p_f = 1\times10^{-\beta} \qquad 4.9$$

Thus, the reliability index β defined by ***Cornell (1969)*** can be obtained for a non linear function by expanding the function about the design point D. This means that the non linear surface is approximated by its tangent plane at the design point D. For a nonlinear surface, the shortest distance of the origin to the failure surface is not unique as in case of a linear failure surface. ***Shinozuka (1983)*** has proved that the point D on the failure surface with minimum distance to the origin (normalized coordinate system) is the most probable failure point. Thus, the tangent plane at the design point D may be taken to approximate the value of β. If the failure is concave towards the origin, the approximation will be on the safer side, while for the surface which is convex towards the origin it will be on unsafe side.

4.2.3 Mathematical Model

The checking of reliability of structure elements involves the evaluation of reliability index β or in other words evaluating the safety corresponding to a particular reliability index. The reliability-based design procedure is an inverse problem, wherein a design has to be achieved which shall ensure a certain level of reliability. A probabilistic procedure has

been developed using the first order second moment approach using Hasofer Linds method for finalizing Partial Safety Factors. The procedure involved the calculation of design parameters for the given target β. Different levels of reliability yield different surfaces, amounting to different designs. Hence, in the proposed procedure the values of the variables, which will ensure that the design will have a failure surface that complies with a required reliability index βare determined

The limit state design methodology, stipulated in IS: 456 – 2000, considers a partial safety factor of 1.5 for concrete and 1.15 for steel, when assessing the strength of a structure element. These partial safety factors account for any possibility of variation in the strength of material, deviation of sectional dimensions, lack of accuracy of the calculated procedure, and risk of life and economical consequences. In the present study, independent partial safety factors for variation in strength of fibrous fly ash concrete have been determined. A step-by-step procedure for computing the partial safety factors taking in to account the concrete strength variation has described below.

1. The relationship between water-cementitious material ratio and experimentally generated compressive strength (in terms of means and standard deviations) obtained for three curing ages of 7, 14 and 28 days and experimentally generated values of splitting tensile strength after 28 days have been shown in Figs. 4.4 to 4.7. The mathematical expressions for different cases are given below

(a) Compressive Strength

For concrete mixes without replacement of cement by fly ash, the relationship is represented in Figure 4.4 and the regression equations at different curing ages are

7 Days curing

$$f_{ck} = 13.33\left(\frac{w}{c}\right)^{-0.8605}$$

$$R^2 = 0.9102$$

14 Days curing

$$f_{ck} = 17.66\left(\frac{w}{c}\right)^{-0.8775}$$

$$R^2 = 0.9365$$

28 Days curing

$$f_{ck} = 22.62\left(\frac{w}{c}\right)^{-0.6857}$$

$$R^2 = 0.8927$$

For concrete mixes with 50 per cent replacement of cement by Fly ash, the relationship presented in Fig. 4.5 and the regression equations at different curing ages are

7 Days curing

$$f_{ck} = 1.38\left(\frac{w}{c+p}\right)^{-3.049}$$

$$R^2 = 0.8009$$

14 Days curing

$$f_{ck} = 2.02\left(\frac{w}{c+p}\right)^{-2.718}$$

$$R^2 = 0.8389$$

28 Days curing

$$f_{ck} = 22.62\left(\frac{w}{c+p}\right)^{-2.886}$$

$$R^2 = 0.9065$$

For concrete mixes with 60 per cent replacement of cement by Fly ash fume, the relationship presented in Fig. 4.6 and the regression equations at different curing ages are

7 Days curing

14 Days curing

28 Days curing

$$f_{ck} = 0.21\left(\frac{w}{c+p}\right)^{-4.045}$$
$$R^2 = 0.6550$$

$$f_{ck} = 2.02\left(\frac{w}{c+p}\right)^{-2.253}$$
$$R^2 = 0.6174$$

$$f_{ck} = 22.62\left(\frac{w}{c+p}\right)^{-3.055}$$
$$R^2 = 0.6601$$

The relation between compressive strength and water-cementitious material ratio can be expressed in the form:

$$f_{ck} = \frac{1}{A}\left(\frac{W}{C}\right)^{B} \qquad\qquad f_{ck} = \frac{1}{A}\left(\frac{W}{C+p}\right)^{B}$$

(b) Splitting Tensile Strength

For concrete mixes without replacement of cement by Fly ash, the relationship is presented in Fig. 4.7(a), for concrete mixes with 50% replacement of cement the relationship is presented in Fig 4.7(b), for concrete mixes with 60% replacement of cement the relationship is presented in Fig 4.7(c) and the regression equations at 28 days of curing are:

0% Fly Ash

$$f_{ck} = 6.38\left(\frac{w}{c}\right)^{-0.7180}$$

$$R^2 = 0.8798$$

50% Fly Ash

$$f_{ck} = 0.84\left(\frac{w}{c+p}\right)^{-0.7369}$$

$$R^2 = 0.1864$$

60% Fly Ash

$$f_{ck} = 0.55\left(\frac{w}{c+p}\right)^{-1.203}$$

$$R^2 = 0.6601$$

The relation between compressive strength and water-cementitious material ratio can be expressed in the form:

$$f_{ck} = \frac{1}{A}\left(\frac{W}{C}\right)^{B} \qquad\qquad f_{ck} = \frac{1}{A}\left(\frac{W}{C+p}\right)^{B}$$

Where A and B are the constants, w is the water content, c is the cement content and p is the amount of Fly ash

2. Using the with-in-test coefficient of variation for compressive strength as provided in the Table 3.13 to 3.20, the revised coefficients of variation and standard deviation have been calculated using the equations as below

$$\delta_{CSrev} = \left[\left(\delta_{CS}\right)^2 + \left(\delta_{\mathrm{CST}}\right)^2\right]^{1/2} \qquad 4.10$$

$$\sigma_{\mathrm{CSrev}} = \mu_{CS}(\delta_{CSrev}) \qquad 4.11$$

where δ_{CSrev} and σ_{CSrev} are the revised values of coefficient of variation and standard deviation respectively; δ_{CS} and μ_{CS} are the experimentally obtained values of coefficient of variation and means of compressive strengths, respectively, and δ_{CST} is the with-in-test coefficient of variation.

Consider the coefficient of variation of water-cementitious ratio at site to be 5 percent. Thus

$$\delta_{WCR} = 0.05 \tag{4.12}$$

The standard deviation values for water-cementitious ratio σ_{WCR} have been obtained by multiplying the actual values by coefficient of variation. The values of coefficients A and B along with the means, revised coefficients of variation and standard deviations of compressive strengths for different water cementitious material ratios and curing ages have been tabulated in the Tables from Table 4.1 to 4.2

3. The governing equation of the design values has been formulated in the form **(Kumar, 2002)**

$$g(Z) = 0$$

$$(\mu_{CSrev} + \sigma_{CSrev} z_{cs}) - \frac{1}{A}(\mu_{Wcr} + \sigma_{Wcr} z_{wcr})^{B} = 0 \tag{4.13}$$

where $\mu_{CS\,rev}$ and σ_{CSrev} are the revised means and standard deviations of compressive strength and μ_{WCR} and σ_{WCR} are the revised means and standard deviations of water-cementitious ratio. The values of $\alpha_1 and \alpha_2$ have been computed using Hasofer Lind's method for different cases using normal distribution function.

(a) Normal Distribution Function

The values of $\alpha_1 and \alpha_2$ have been evaluated using the equations

$$\alpha_1 = \left(-\frac{1}{K}\right)\left(\sigma_{\mathrm{CSrev}} A\right) \tag{4.14}$$

$$\alpha_2 = \left(-\frac{1}{K}\right)\left(-\sigma_{\mathrm{WCR}} B\right) \tag{4.15}$$

$$\alpha_1^{\,2} + \alpha_2^{\,2} = 1 \tag{4.16}$$

Using these values of $\alpha_1 and \alpha_2$, the design of compressive strength and water cementitious material ratio for varying reliability indices (β= 1.0, 1.3, 2.0, 3.0) have been obtained by Eqns. (4.17) as below:

$$CS^* = \mu_{CS} - \alpha_1 \beta \sigma_{CSrev} \qquad 4.17$$

The characteristic values corresponding to 95 per cent confidence level for compressive strength have been found by using the equations as below:

$$CS^* = \mu_{CS} - 1.64\sigma_{CSrev} \qquad 4.18$$

4.2.4 Evaluation of Partial Safety Factors

The probabilistic procedure for design of reinforced concrete rectangular beams is based on philosophy of partial safety factors for variations in concrete strength. Partial safety factor is defined with respect to a particular value of the variable. It is defined with respect to the mean value as ratio of the design value to the mean value as given below

$$\gamma_{ci} = \frac{x_i^*}{\mu_i}$$

In the present study partial safety factor with respect to the characteristic value or the characteristic strength of concrete (the value of the strength of 150mm cubes, after 28 days of curing below which not more than 5 per cent of the test results are expected to fall) is defined as

$$\gamma_{ci} = \frac{x_i^*}{x_{ki}}$$

where x_{ki} is the characteristic value of compressive strength.

From the design and characteristic values of compressive strength evolved in the preceding section the partial safety factors (PSF) with respect to the characteristic values for different curing ages and different replacement ratios of cement by fly ash and reliability indices have been evaluated. Partial safety factors for compressive strength and splitting strength are being tabulated in tables 4.7 to 4.9 and 4.10 to 4.12 respectively.

Table 4.1(a) Parameters for concrete mixes without fly ash

Curing Period	**7 Days**	**14 Days**	**28 days**
Parameters	A=0.075 B= –0.8605	A=0.056 B= –0.8775	A=0.044 B= –0.6357

Table 4.1(b) Revised compressive strength parameters for concrete mixes without fly Ash

W/C Ratio	**7 days curing**			**14 days curing**			**28 days curing**		
	Mean MPa	**Rev CoV per cent**	**Rev SD, MPa**	**Mean MPa**	**Rev CoV per cent**	**Rev SD, MPa**	**Mean MPa**	**Rev CoV per cent**	**Rev SD, MPa**
0.40	29.0	4.20	1.22	39.0	3.2	1.24	40.0	4.3	1.72
0.42	28.0	5.02	1.40	38.0	4.7	1.78	39.50	3.6	1.42
0.44	27.3	6.50	1.77	37.3	5.5	2.05	38.67	7.8	3.01
0.46	26.6	5.80	1.54	34.6	4.6	1.59	37.6	2.7	1.01
0.48	24.3	5.00	1.22	33.3	3.3	1.09	35.3	4.5	1.41

Table 4.2(a) Parameters for concrete mixes having 50% fly ash

Curing Period	**7 Days**	**14 Days**	**28 days**
Parameters	A=0.7217 B= −3.049	A=0.4950 B= −2.718	A=0.4811 B= −2.886

Table 4.2(b) Revised Compressive strength data for concrete mixes with 50% replacement of cement by fly ash

W/(C+p) Ratio	**7 days curing**			**14 days curing**			**28 days curing**		
	Mean MPa	**Rev CoV per cent**	**Rev SD, MPa**	**Mean MPa**	**Rev CoV per cent**	**Rev SD, MPa**	**Mean MPa**	**Rev CoV per cent**	**Rev SD, MPa**
0.40	21.30	8.4	1.79	22.0	8.26	1.81	28.0	7.2	2.01
0.42	20.30	2.9	0.59	21.6	7.60	1.64	26.50	8.1	5.08
0.44	18.67	7.9	1.48	20.6	3.60	0.74	23.30	9.7	2.26
0.46	16.30	8.4	1.37	17.8	3.10	0.55	20.67	9.5	1.96
0.48	10.17	7.6	.779	12.5	8.10	1.01	15.16	7.0	1.06

Table 4.3(a) Parameters for concrete mixes having 60% fly ash

Curing Period	**7 Days**	**14 Days**	**28 days**
Parameters	A=4.753 B= –4.045	A=0.8650 B= –2.532	A=0.8942 B= –3.051

Table 4.3(b) Revised Compressive strength data for concrete mixes with 60% replacement of cement by Fly ash

W/C+p Ratio	**7 days curing**			**14 days curing**			**28 days curing**		
	Mean MPa	**Rev CoV per cent**	**Rev SD, MPa**	**Mean MPa**	**Rev CoV per cent**	**Rev SD, MPa**	**Mean MPa**	**Rev CoV per cent**	**Rev SD, MPa**
0.40	9.30	6.05	1.12	12.0	10.0	1.20	18.5	12.6	2.33
0.42	6.67	7.50	0.60	11.3	7.60	0.79	17.5	11.8	2.07
0.44	4.50	5.05	0.67	7.33	9.50	0.66	10.3	13.3	1.37
0.46	4.67	5.65	0.65	7.67	8.00	0.62	11.5	10.0	1.15
0.48	5.83	7.5	1.22	8.83	12.4	1.09	12.5	12.0	1.50

Table 4.4 (a) Parameters for concrete mixes without fly ash (Splitting Tensile Strength)

Curing Period	28 Days
Parameters	A=0.1567 B= –0.7180

Table 4.4(b) Revised Splitting tensile strength parameters for concrete mixes without fly Ash

W/C Ratio	28 days curing		
	Mean MPa	Rev CoV per cent	Rev SD, MPa
0.40	3.22	19.8	0.63
0.42	3.50	19.7	0.69
0.44	3.90	21.2	0.62
0.46	3.64	17.9	0.65
0.48	3.73	16.8	0.62

Table 4.5(a) Parameters for concrete mixes with 50% fly ash(Splitting tensile strength)

Curing Period	**28 Days**
Parameters	A=1.19 B= –0.7369

Table 4.5(b) Revised Splitting Tensile Strength parameters for concrete mixes with 50% fly ash

W/C+p Ratio	**28 days curing**		
	Mean MPa	**Rev CoV per cent**	**Rev SD, MPa**
0.40	1.60	20.8	0.78
0.42	2.30	21.5	0.75
0.44	1.90	23.0	0.63
0.46	1.40	17.9	1.092
0.48	1.50	16.8	0.684

Table 4.6(a) Parameters for concrete mixes with 60% fly ash

Curing Period	**28 Days**
Parameters	A=1.809 B=−1.203

Table 4.6(b) Revised Splitting Tensile Strength parameters for concrete mixes with 60% fly ash

W/C+p Ratio	**28 days curing**		
	Mean MPa	**Rev CoV per cent**	**Rev SD, MPa**
0.40	1.50	21.6	0.46
0.42	1.63	25.0	0.41
0.44	1.79	23.0	0.44
0.46	1.37	35.0	0.48
0.48	1.17	36.0	0.42

Table 4.7 Partial Safety Factor (PSF) for concrete with 0% fly ash (Normal Distribution)

W/C Ratio	After7 Days				After 14 days				After 28 days			
	β=1	**β=1.3**	**β=2**	**β=3**	**β=1**	**β=1.3**	**β=2**	**β=3**	**β=1**	**β=1.3**	**β=2**	**β=3**
0.40	1.12	1.14	1.16	1.18	1.08	1.09	1.12	1.15	1.33	1.13	1.16	1.21
0.42	1.12	1.16	1.19	1.25	1.13	1.14	1.18	1.23	1.1	1.11	1.13	1.17
0.44	1.19	1.21	1.26	1.33	1.15	1.17	1.22	1.28	1.23	1.26	1.32	1.41
0.46	1.16	1.18	1.23	1.29	1.13	1.14	1.18	1.23	1.07	1.08	1.10	1.13
0.48	1.14	1.15	1.19	1.25	1.09	1.10	1.12	1.16	1.11	1.12	1.15	1.19

Table 4.8 Partial Safety Factor (PSF) for concrete with 50% fly ash (Normal Distribution)

W/C+P Ratio	After7 Days				After 14 days				After 28 days			
	β=1	**β=1.3**	**β=2**	**β=3**	**β=1**	**β=1.3**	**β=2**	**β=3**	**β=1**	**β=1.3**	**β=2**	**β=3**
0.40	1.25	1.28	1.35	1.45	1.25	1.27	1.34	1.44	1.22	1.24	1.29	1.37
0.42	1.08	1.08	1.11	1.14	1.22	1.25	1.31	1.4	1.73	1.82	2.01	2.3
0.44	1.24	1.26	1.33	1.42	1.01	1.11	1.13	1.17	1.3	1.34	1.42	1.53
0.46	1.25	1.28	1.35	1.45	1.09	1.09	1.11	1.15	1.29	1.33	1.41	1.52
0.48	1.22	1.24	1.32	1.38	1.24	1.27	1.34	1.43	1.2	1.23	1.28	1.36

Table 4.9 Partial Safety Factor (PSF) for concrete with 60% f fly Ash (Normal Distribution)

W/C+P Ratio	After7 Days				After 14 days				After 28 days			
	β=1	**β=1.3**	**β=2**	**B=3**	**β=1**	**β=1.3**	**β=2**	**β=3**	**β=1**	**β=1.3**	**β=2**	**β=3**
0.40	1.39	1.94	1.55	1.69	1.32	1.34	1.43	1.55	1.42	1.47	1.58	1.73
0.42	1.27	1.31	1.38	1.48	1.2	1.23	1.28	1.36	1.38	1.43	1.53	1.68
0.44	1.68	1.58	1.72	1.92	1.28	1.31	1.38	1.49	1.44	1.49	1.62	1.78
0.46	1.47	1.53	1.66	1.84	1.24	1.27	1.34	1.43	1.32	1.35	1.43	1.49
0.48	1.84	1.93	2.14	2.47	1.41	1.45	1.56	1.71	1.39	1.44	1.47	1.58

Table 4.10 Partial Safety Factor (PSF) for concrete with 0% fly Ash (Normal Distribution) Splitting Tensile Strength

W/C Ratio	After 28 days			
	β=1	**β=1.3**	**β=2**	**β=3**
0.40	1.76	1.85	2.05	2.34
0.42	1.77	1.86	2.06	2.36
0.44	1.83	1.92	1.17	2.47
0.46	1.66	1.74	1.97	2.17
0.48	1.61	1.67	1.83	2.06

Table 4.11 Partial Safety Factor (PSF) for concrete with 50% fly Ash (Normal Distribution)

W/C+P Ratio	After 28 days			
	β=1	β=1.3	β=2	β=3
0.40	1.58	1.33	1.07	1.27
0.42	2.91	3.12	3.62	4.28
0.44	2.97	3.20	3.73	4.47
0.46	3.97	4.20	3.9	5.00
0.48	5.77	6.33	7.58	6.34

Table 4.12 Partial Safety Factor (PSF) for concrete with 60%fly Ash (Normal Distribution)

W/C+P Ratio	After 28 days			
	β=1	β=1.3	β=2	β=3
0.40	2.64	2.83	3.27	3.89
0.42	2.14	2.27	2.57	3.01
0.44	2.10	2.22	2.51	2.93
0.46	3.03	3.26	3.82	4.60
0.48	3.31	3.5	4.18	5.06

Fig. 4.1 Failure surface, failure and safe regions *(Ranganathan, 1990)*

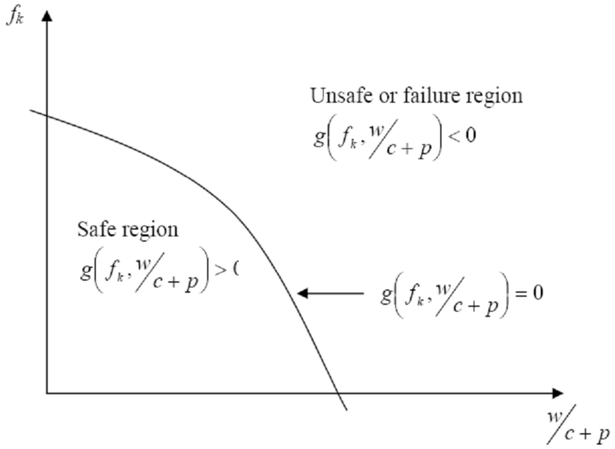


Fig 4.2 Standard normal density function *(Ranganathan, 1990)*

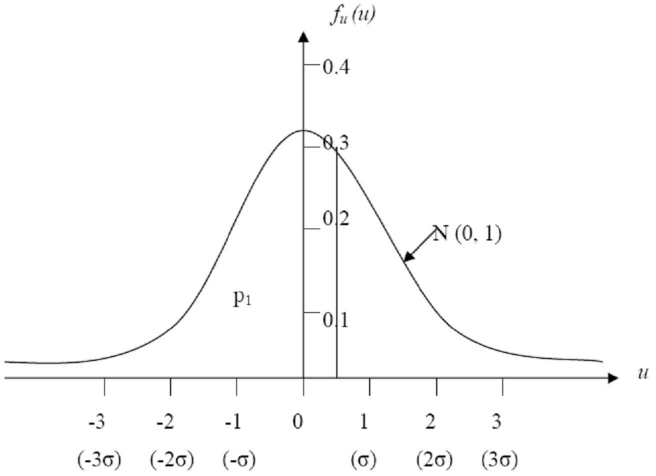
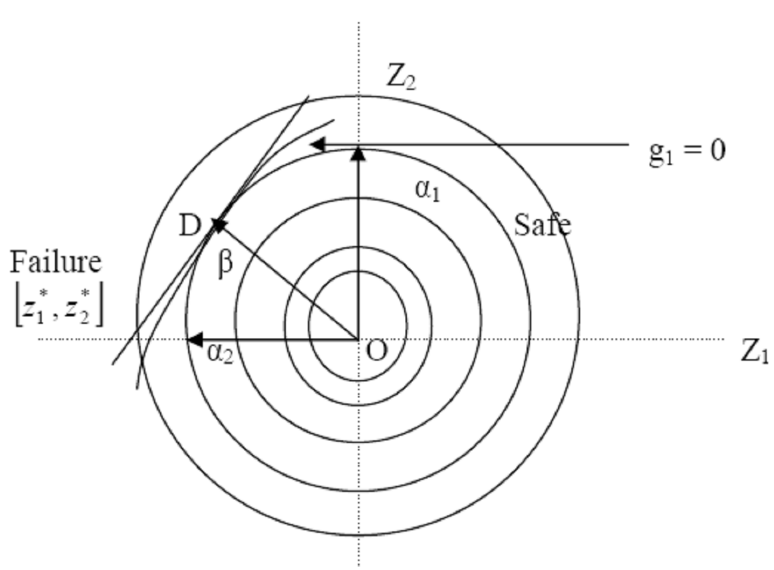
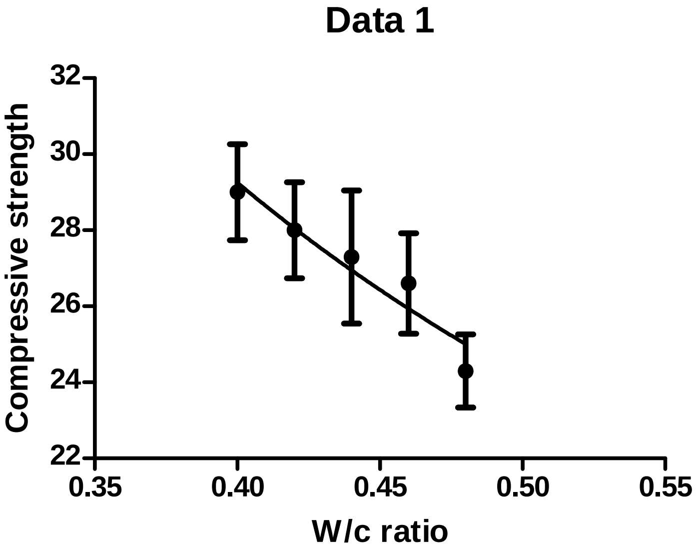
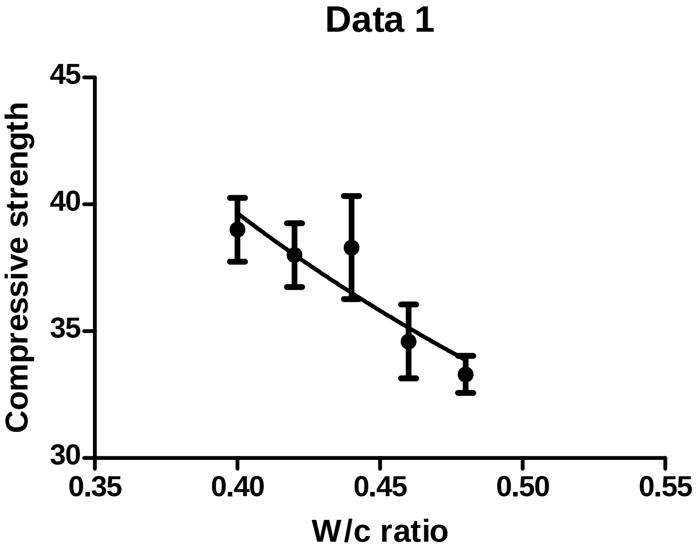


Fig. 4.3 Formulation of safety analysis in normalized coordinates *(Ranganathan, 1990)*

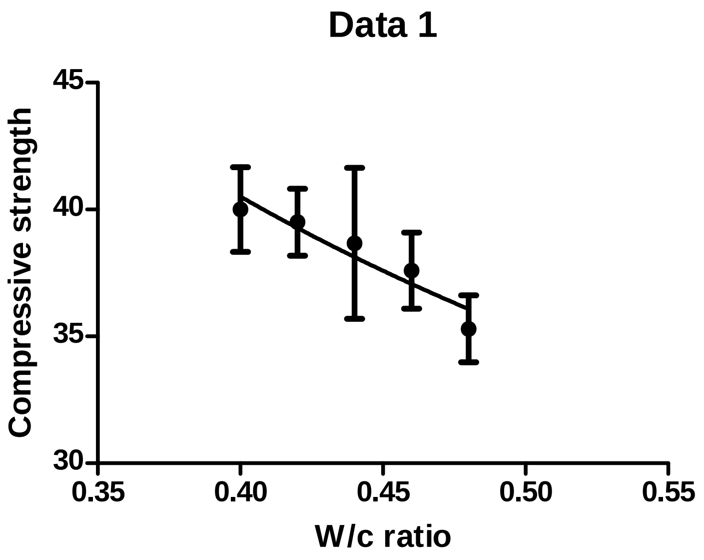

Z2
g1 = 0
α1
Safe
D
β
Failure
[z1*, z2*]
α2
O
Z1



(a) Normalised Compressive Strength vs water cement ratio curves for 7 days

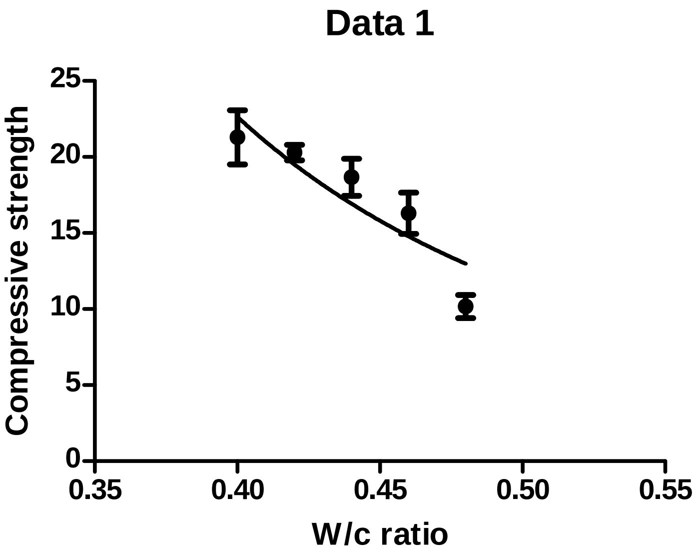


(b)Normalised Compressive Strength vs water cement ratio curves for 14 days

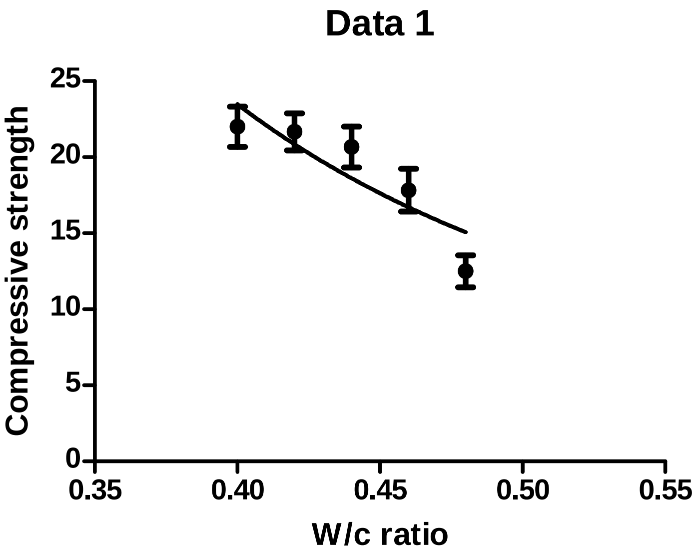


(c) Normalised Compressive Strength vs water cement ratio curves for 28 days

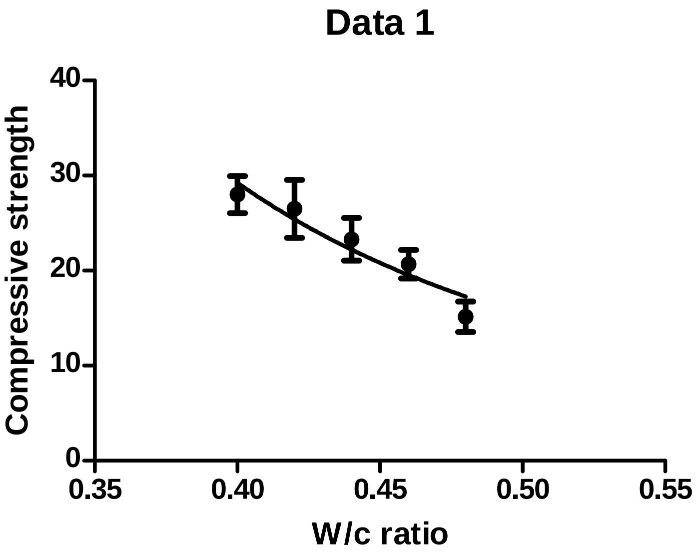
Fig 4.4 a,b,c Normalised Compressive Strength vs. water cement ratio curves for 7, 14, 28 day testing without fly ash



(a)Normalised Compressive Strength vs water cement ratio curves for 7 days (fly ash 50%)

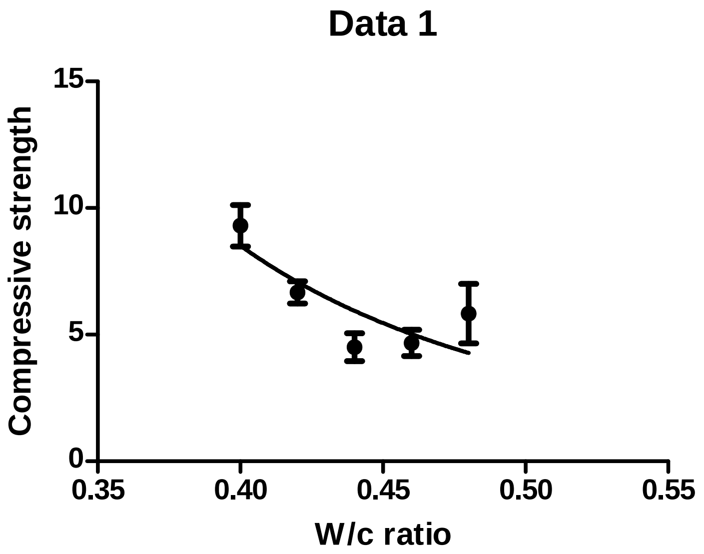


(b)Normalised Compressive Strength vs water cement ratio curves for 14 days (fly ash 50%)

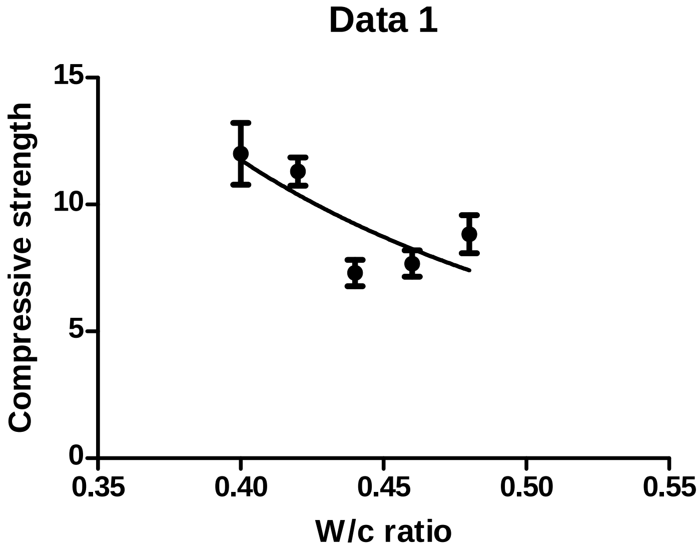


(c)Normalised Compressive Strength vs water cement ratio curves for 28 days (fly ash 50%)

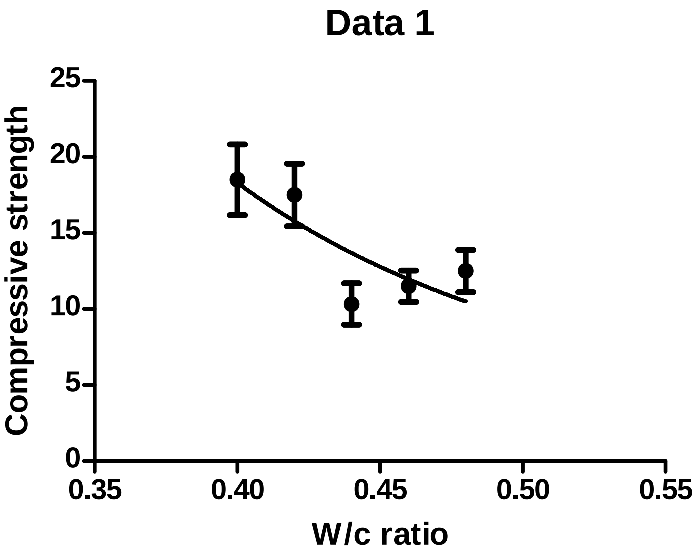
Fig 4.5 a,b,c Normalised Compressive Strength vs. water cement ratio curves for 7, 14, 28 day testing with fly ash 50%



(a)Normalised Compressive Strength vs. water cement ratio curves for 7 days (fly ash 60%)



(b)Normalised Compressive Strength vs. water cement ratio curves for 14 days (fly ash 60%)



(c) Normalised Compressive Strength vs. water cement ratio curves for 28 days (fly ash 60%)

Fig 4.6 a,b,cNormalised Compressive Strength vs. water cement ratio curves for 7, 14, 28 day testing with fly ash 60%

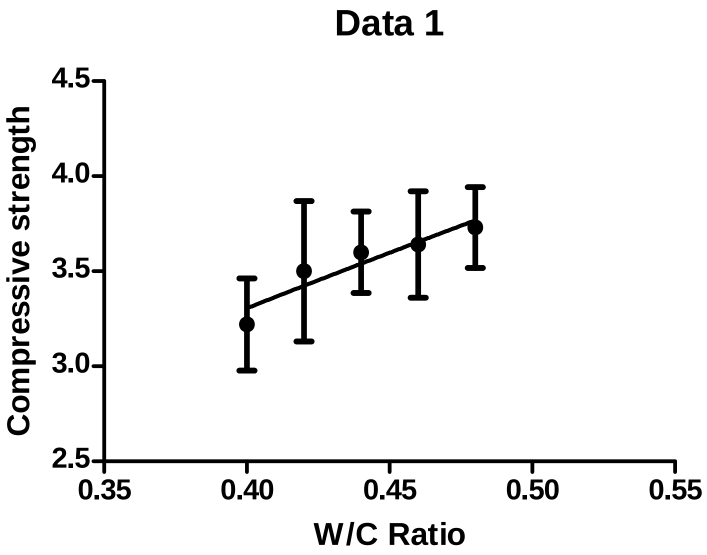


Fig 4.7 (a) Splitting Tensile Strength vs. water cement ratio curves for 28 days testing without fly ash

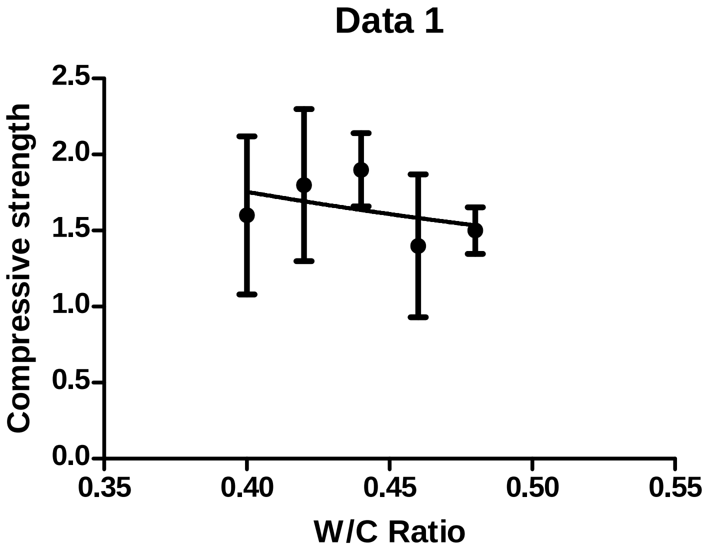


Fig 4.7(b) Splitting Tensile Strength vs. water cement ratio curves for 28 days testing with fly ash (50%)

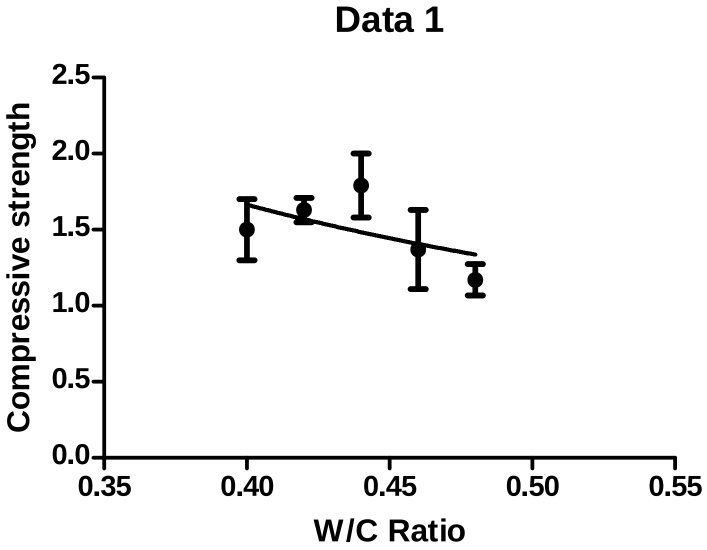


Fig 4.7 (c) Splitting Tensile Strength vs. water cement ratio curves for 28 days testing with fly ash (60%)

CHAPTER 5 RESULTS AND DISCUSSIONS

5.1 GENERAL

This chapter is divided into two sections. First section deals with the effect of variations in parameters on properties of concrete with and with out Fly ash. In the second section the effect of various parameters on the partial safety factors for concrete strengths obtained by the proposed procedure has been discussed .

5.2 PROPERTIES OF CONCRETE

The focus in this section is on understanding the effect of various parameters on workability of fresh concrete and statistical analysis of experimentally generated compressive strength data.

5.2.1 Workability

The variations in Compaction factor of concrete mix with water-cementitious material ratio, for different percentage replacements of cement by Fly ash have been presented in Tables3.10 and 3.11. The concrete mixes using 50 and 60 per cent Fly ash, have shown lesser workability as compared to concrete mixes without Fly ash, for the same water contents. This is attributed to the fact that the fly ash used in the present study is slightly coarser than cement. Also the addition of fibres to the concrete system reduces the workability of concrete.

5.2.2 Statistical properties of Compressive Strength

The statistical analysis of the experimentally generated data and its effect on various parameters is discussed below

(a) Coefficient of variation

The frequency analysis of coefficient of variation (ratio of standard deviation to mean value) of experimentally generated compressive strength data for the concrete mixes with different percentage of Fly ash at different curing ages is presented in Table 5.1. The values of the coefficient of variation have been calculated from the data given in Tables 3.13 to 3.15. From Table 5.1 it is observed that most of the concrete mixes without Fly ash have a coefficient of variation of compressive strength more than 5 per cent at 7 days curing age, but it decreases to 3 to 4 per cent as the curing age increases. It is also observed that concrete with 50 per cent Fly ash have coefficient of variation 4 to 5 per cent for 7 days and less than 3 per cent for 14 and 28 days. A similar trend is observed for concrete with 60 per cent replacement of cement by Fly ash having coefficient of variation lying between 3 to 5 per cent at 7 days and nearly 3 per cent for 14 and 28 days. Thus for all the concrete mixes, with and without Fly ash , the with-in-test coefficient of variation is less than permissible value of 6.0 per cent as suggested by ACI report (1988), which indicates that the testing conditions have not affected the results of compressive strengths.

(b) With-in-test coefficient of variation

The frequency analysis of with-in-test coefficient of variation of experimentally generated compressive strength data for the concrete mixes with different percentages of fly ash is presented in the Table 5.2. The values of with-in-test coefficient of variation have been listed in Tables 3.13 to 3.15. From Table 5.2 it is observed that most of the concrete mixes with and without Fly ash have a with-in-test coefficient of variation of compressive strength between 2 to 4 per cent. However, at 14 and 28days curing most of the concrete mixes have a within test coefficient of variation lying between 1.5 to 6.0 per cent. It is also seen that none of the concrete mixes with Fly ash and very few without Fly ash have a with-in-test coefficient of variation between 1.5 to 2.0 per cent at curing ages of 14 and 28 days, indicating improvement in homogeneity of hardened concrete at higher curing ages. Thus for all the concrete mixes, with and without Fly ash , the with-in-test coefficient of variation is less than permissible value of 4.0 per cent as suggested by ACI report (1988), which indicates that the testing conditions have not affected the results of compressive strengths

For Splitting tensile results it is observed that the concrete mixes have a within test coefficient of variation lying between 1.5 to 5.0 per cent. It is also seen that none of the concrete mixes with Fly ash and very few without Fly ash have a with-in-test coefficient of variation between 1.5 to 2.0 per cent at curing ages of 14 and 28 days, indicating improvement in homogeneity of hardened concrete at higher curing ages. Thus for all the concrete mixes, with and without Fly ash , the with-in-test coefficient of variation is less than permissible value of 4.0 per cent as suggested by ACI report (1988), which indicates that the testing conditions have not affected the results of compressive strengths

5.2.3 Compressive Strength

Variation of compressive strength with water different water cement ratios are being represented in figures 5.1 to 5.8**.**

(a) 7 days curing

The relationship of compressive strength with water cement ratios for 7 days with 50% and 60% replacement of cement is shown in figure 5.1 and 5.4.From figures it is clear that for 50% replacement of cement the compressive strength reduces as the water cement ratio increases on the other hand for 60% Fly ash the compressive strength reduces up to water cement ratio 0.40 to 0.48 but it show little bit increase for water cement ratio 0.48. This is attributed to the fact that fly ash does not contribute to strength development during early days of curing

(b) 14 days curing

The relationship of compressive strength with water cement ratios for 14 days with 50% and 60% replacement of cement is shown in figure 5.2 and 5.5 .From figures it is clear that for 50% replacement of cement the compressive strength reduces as the water cement ratio increases this strength is more than 7 days strength on the other hand for 60% Fly ash the compressive strength reduces up to water cement ratio 0.40 to 0.46 but it show little bit increase for water cement ratio 0.48. This is attributed to the fact that fly ash does not contribute to strength development during early days of curing

(c) 28 days curing

28 days testing also shows there is reduction in compressive strength for 50% replacement of cement by fly ash as shown in figure 5.3 and 5.6 but for 60% fly ash there is a abrupt decrease in compressive strength corresponding to water cement ratio 0.44 after that it increases little bit for water cement ratio 0.46 and 0.48. This is attributed to the fact that fly ash does not contribute to strength development during early days of curing

5.2.4 Splitting Tensile strength

Splitting tensile strength is tested for only 28 days. The relationship of compressive strength with water cement ratios is represented in figures 5.7 and 5.8

(a) 28 days curing

From figures 5.7 and 5.8 for 50% replacement of cement by fly ash there is reduction in splitting tensile strength as water cement ratio increases similar trend is observed for 60% replacement of cement by fly ash.

5.3 PARTIAL SAFETY FACTORS

Partial safety factor is generally defined as the ratio of design variable to any reference value. In the present study, partial safety factors with respect to the characteristic value or the characteristic strength of concrete have been evaluated. It is observed that partial safety factor increases with increase in reliability index. It is also seen that this decrease is exponential as the reliability index increases from 1.0 to 3.0. The increase in partial safety factor for compressive strength is insignificant as the reliability index increases from 1.0 to 1.3. However, it is of the order of 3 to 6 per cent as the reliability index increases from 2.0 to 3.0.

(a) Compressive Strength

5.3.1 Effect of percentage of Fly ash

The relationship of partial safety factor and compressive strength with different percentage replacement of cement by fly ash corresponding to reliability index β of 1.0, 1.3, 2.0 and 3.0 have been presented in Fig 5.9 to 5.16. In general it is observed that concrete mixes designed for higher replacement of fly ash give lower partial safety factors.

(a) At 7 days curing

The variation of partial safety factor with 7 days compressive strength of concrete at different reliability indices has been shown in Figs. 5.9 and 5.12 .It is observed that for β = 1.0 to 3, the partial safety factor increases with increase in strength of concrete for 50 per cent replacement level, but for 60 per cent Fly ash replacement, the partial safety factors decreases with increase in strength of concrete. This is attributed to the fact that with increase in percentage of the fly ash the increase in strength at early ages is minimal.

(b) At 14 days curing

The variation of partial safety factor with 14 days compressive strength of concrete at different curing ages has been shown in Figs. 5.10 and 5.13 it is observed that for β = 1.0 to 3, the partial safety factor increases as the compressive strength of concrete increases for 50 per cent replacement level but for 60 per cent Fly ash replacement, the partial safety factors decreases with increase in strength of concrete. The partial safety factor is less for 50 per cent replacement level as compared to 60 per cent replacement level. This is attributed to the fact that with increase in percentage of the fly ash the increase in strength at early ages is minimal.

(c) At 28 days curing

The variation of partial safety factor with 28 days compressive strength of concrete at different curing ages has been shown in Figs. 5.11 and 5.14 it is observed that for β = 1.0, the partial safety factor decreases as the compressive strength of concrete mixes increases at both replacement levels, but is more for 60 percent replacement. This is attributed to the fact that with increase in percentage of the fly ash the increase in strength at early ages is minimal also this may be attributed to the value of characteristic strength of concrete as well.

(b) Splitting Tensile Strength

The relationship of partial safety factor and compressive strength with different percentage replacement of cement by fly ash corresponding to reliability index β of 1.0, 1.3, 2.0 and 3.0 have been presented in Fig 5.9 to 5.16. In general it is observed that concrete mixes designed for higher replacement of fly ash give lower partial safety factors.

(a) At 28 days curing

The variation of partial safety factor with 28 days compressive strength of concrete at different curing ages has been shown in Figs. 5.15 and 5.16 it is observed that for β =

1.0 to 3.0 the partial safety factor decreases as the compressive strength of concrete mixes increases at both replacement levels, but is more for 60 percent replacement. This variation may be attributed to the value of characteristic strength of concrete as well

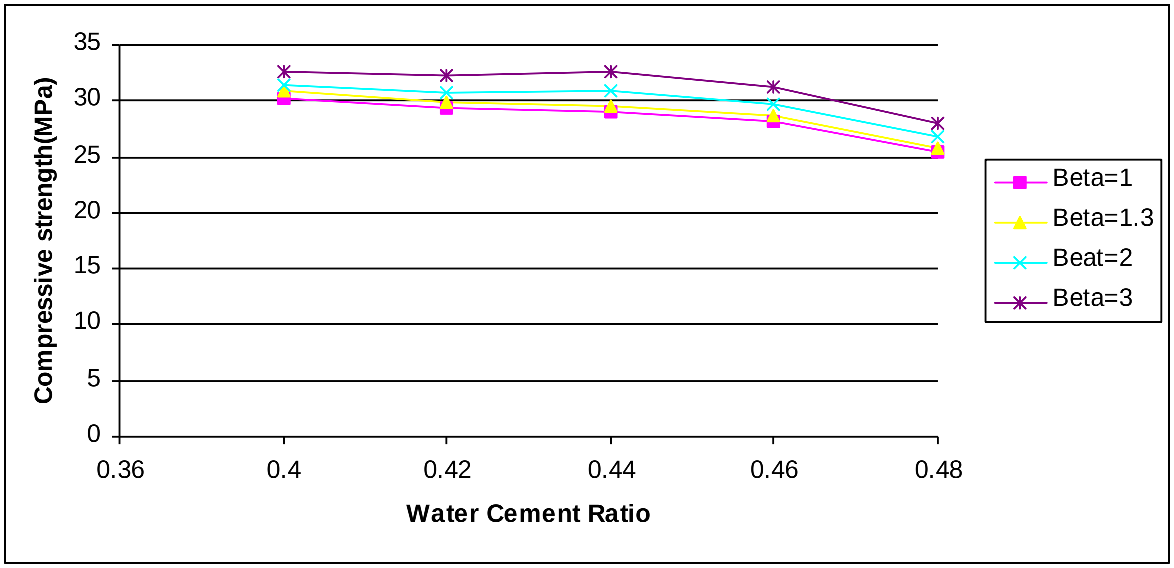
Table 5.1 Coefficient of variation of compressive strength for concrete mixes at different curing ages

Coefficient of Variation	**Number of mixes**								
	Curing period 7 Days			**Curing period 14 days**			**Curing period 28 days**		
	Replacement of cement by Fly ash								
	0 per cent	**50 per cent**	**60per cent**	**0 per cent**	**50 per cent**	**60 per cent**	**0per cent**	**50 per cent**	**60 per cent**
<3.0	----	----	----	----	----	----	2	1	3
3.0-4.0	----	----	1	----	----	1	1	2	----
4.0-5.0	1	3	2	1	3	2	----	----	----

>5.0	2	----	----	2	----	----	----	----	----

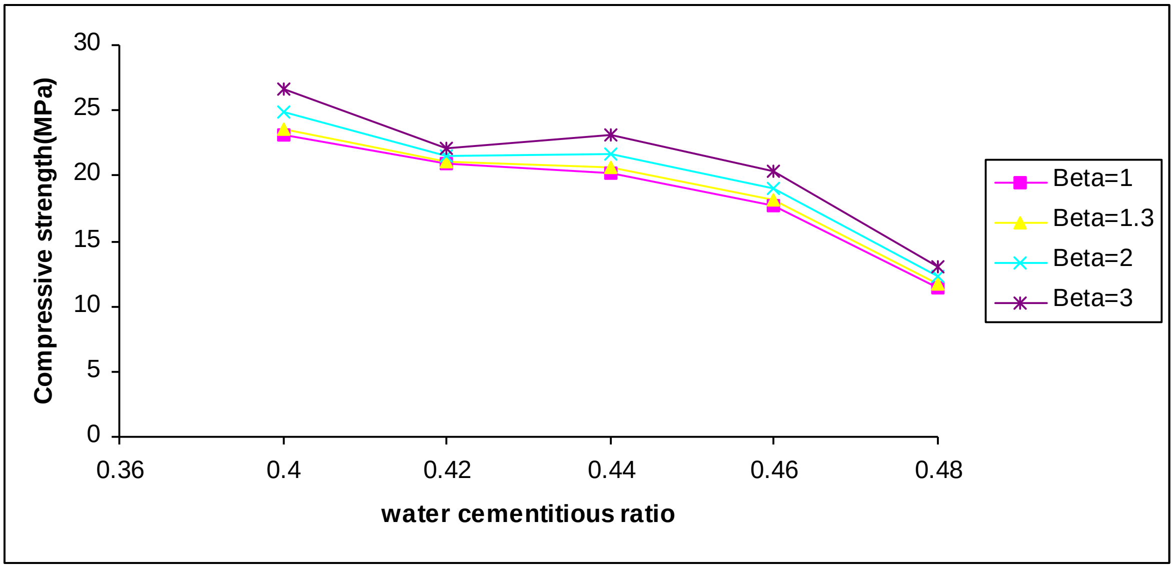
Table 5.2 with in test Coefficient of variation of compressive strength for concrete mixes at different curing ages

With in test Coefficient of Variation	**Number of mixes**								
	Curing period 7 Days			**Curing period 14 days**			**Curing period 28 days**		
	Replacement of cement by Fly ash								
	0 per cent	**50 per cent**	**60per cent**	**0 per cent**	**50 per cent**	**60 per cent**	**0per cent**	**50 per cent**	**60 per cent**
<1.5	----	----	----	----	----	1	----	----	1
1.5-2.0	----	----	---	1	3	----	3	2	2
2.0-3.0	1	1	1	2	----	2	----	1	----
>3.0	2	2	2	----	----	----	2	----	----



Normal Distribution

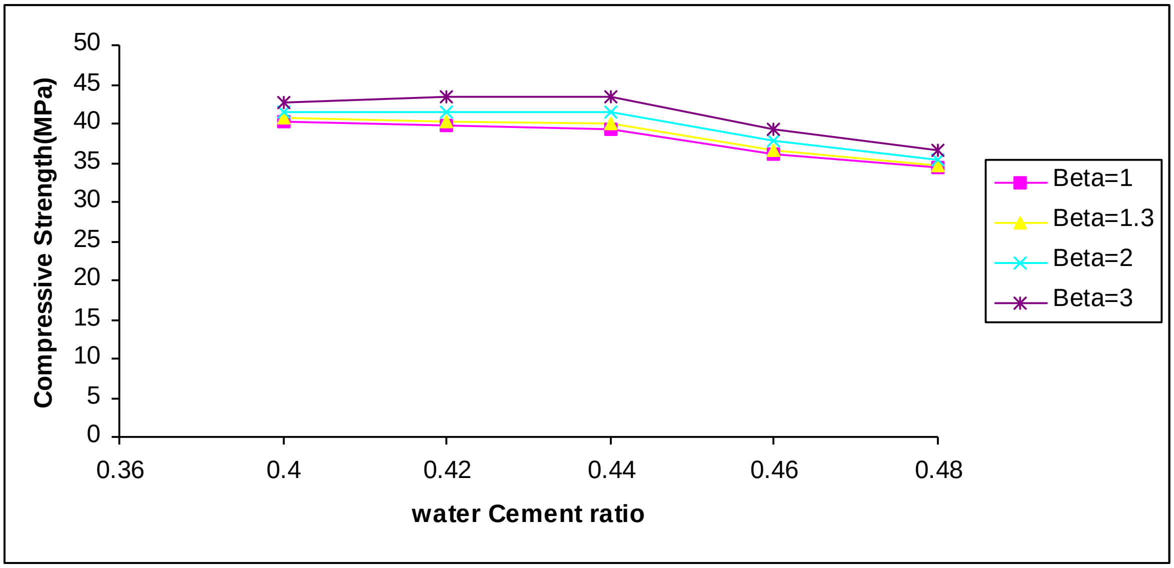
Without Replacement of cement b y Fly ash



Normal Distribution

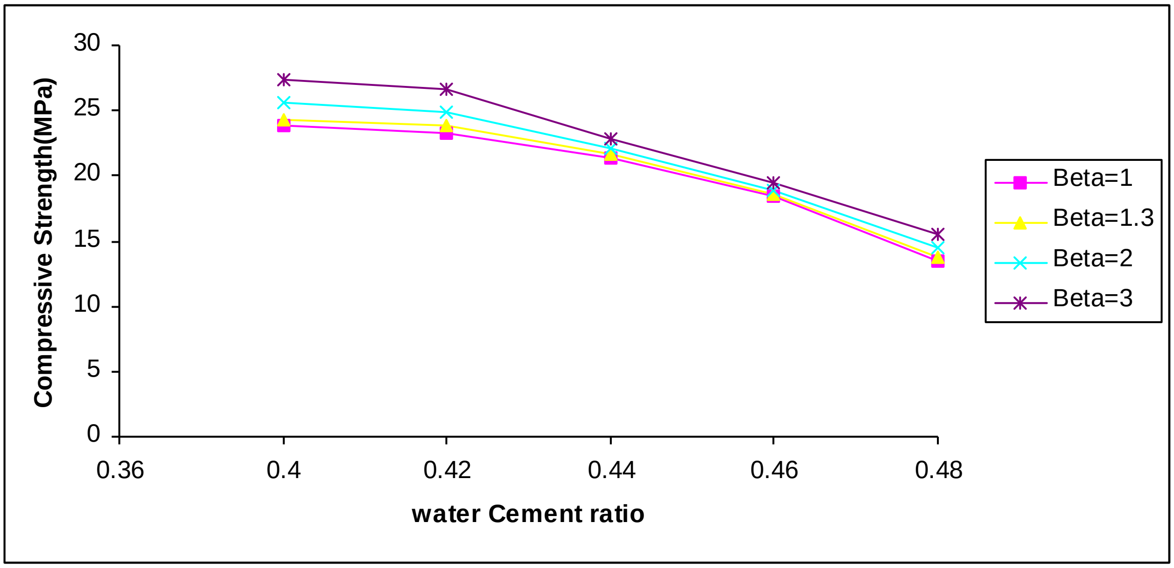
With 50% Replacement of cement by Fly ash

Fig 5.1 Variation of 7 days Compressive strength with water cementitious material ratio (Fly ash 50%)



Normal Distribution

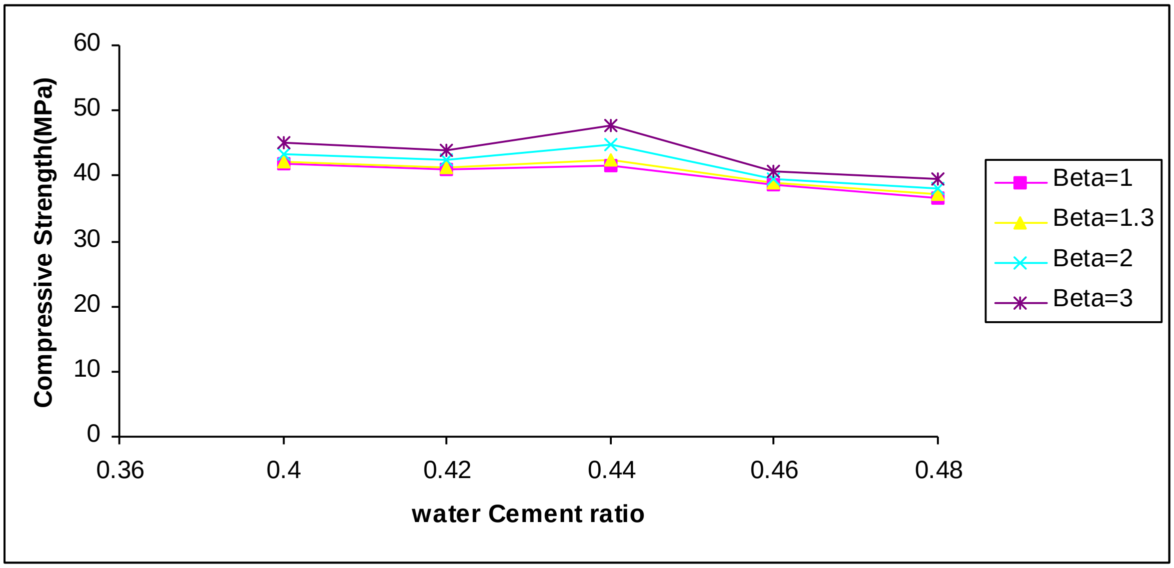
Without Replacement of cement by Fly ash



Normal Distribution

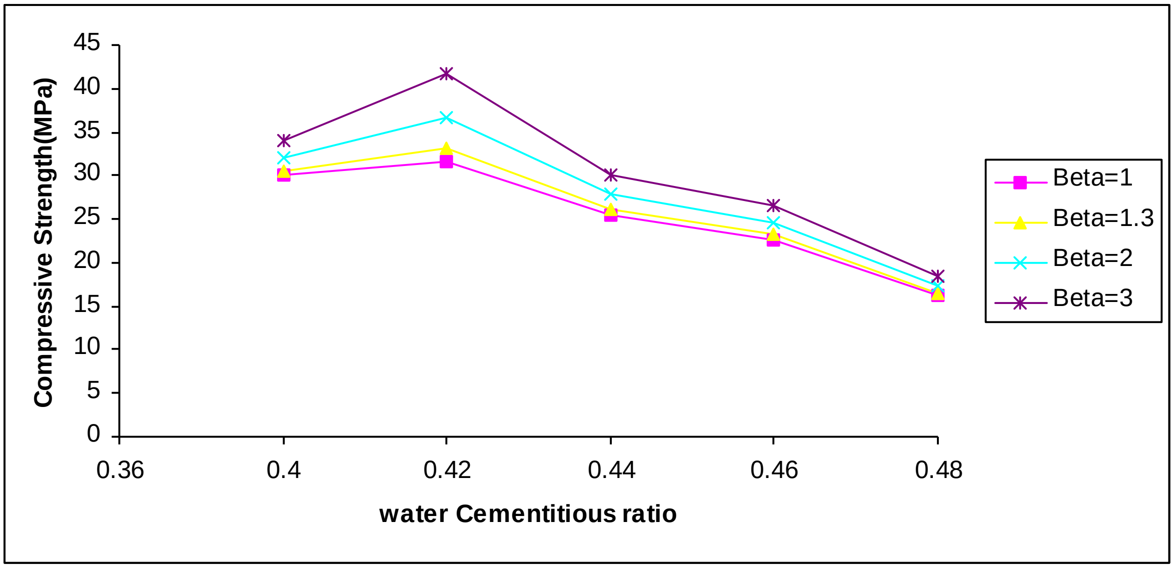
With 50% Replacement of cement by Fly ash

Fig 5.2Variation of 14 days Compressive strength with water cementitious material ratio (Fly ash 50%)



Normal Distribution

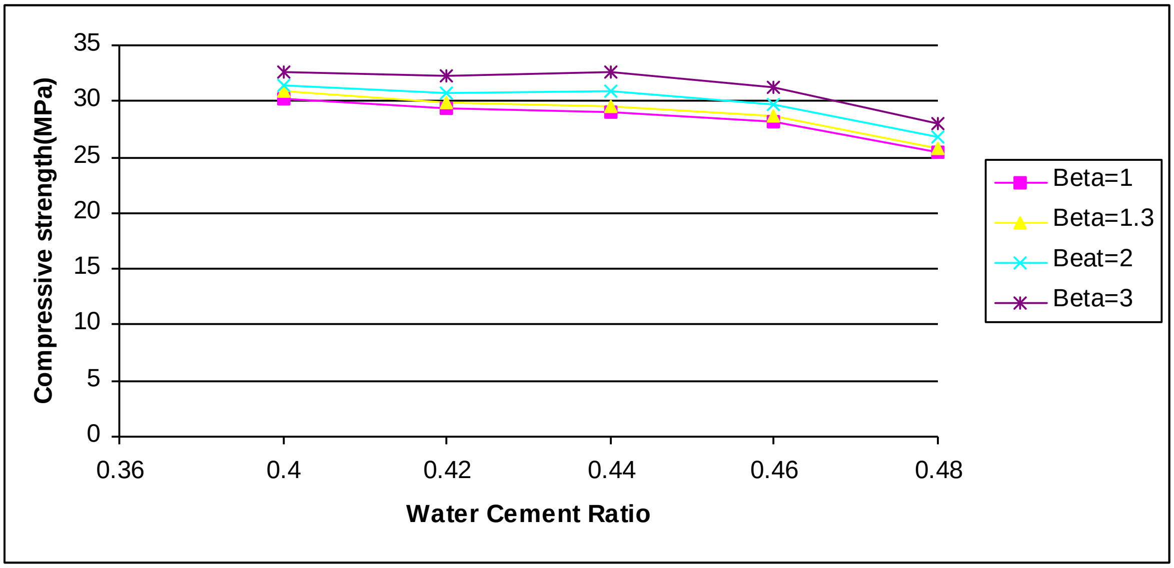
Without Replacement of cement by Fly ash



Normal Distribution

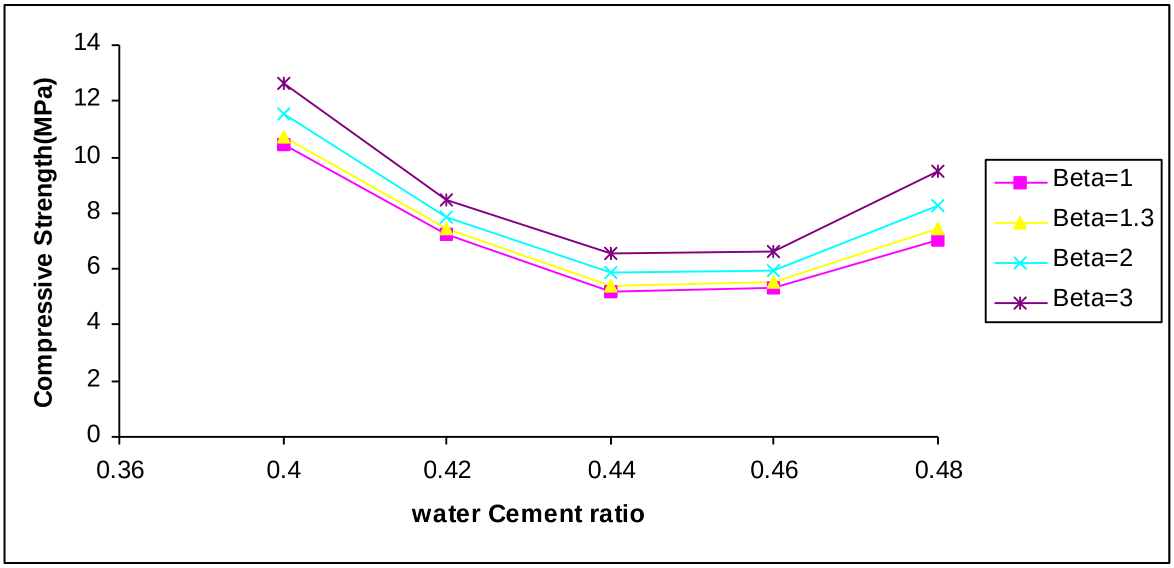
With 50% Replacement of cement by Fly ash

Fig 5.3 Variation of 28 days Compressive strength with water cementitious material ratio (Fly ash 50%)



Normal Distribution

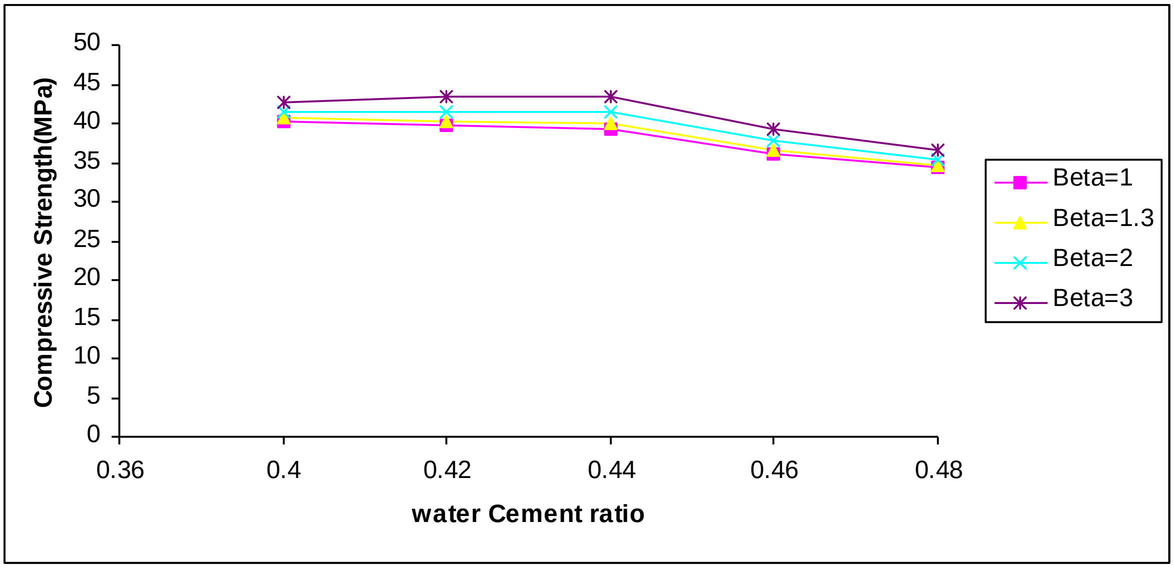
Without Replacement of cement by Fly ash



Normal Distribution

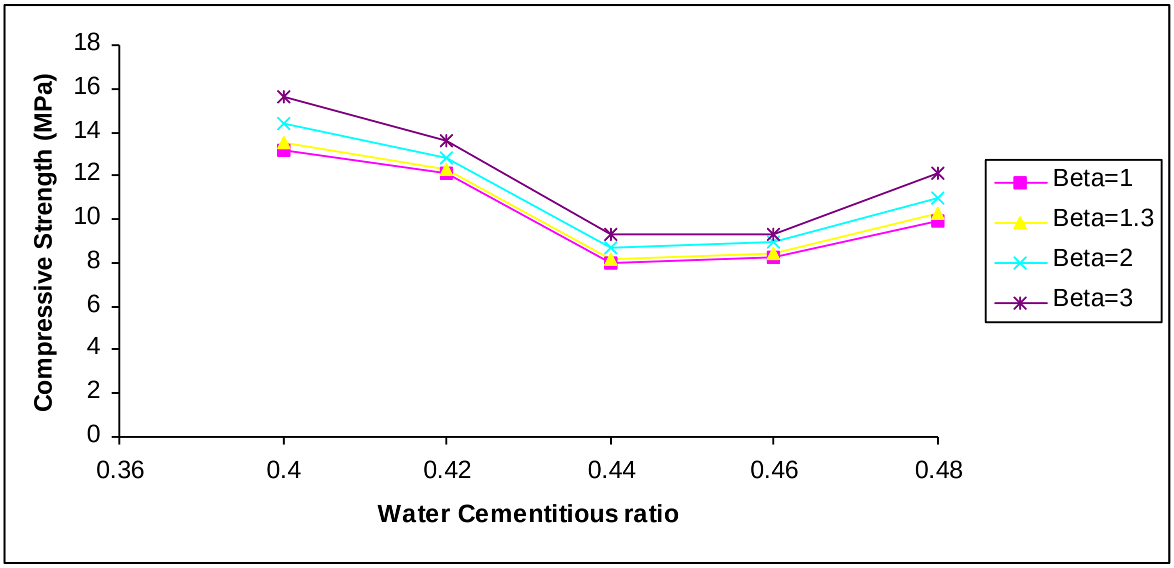
With 60% Replacement of cement by Fly ash

Fig 5.4 Variation of 7 days Compressive strength with water cementitious material ratio (Fly ash 60%)



Normal Distribution

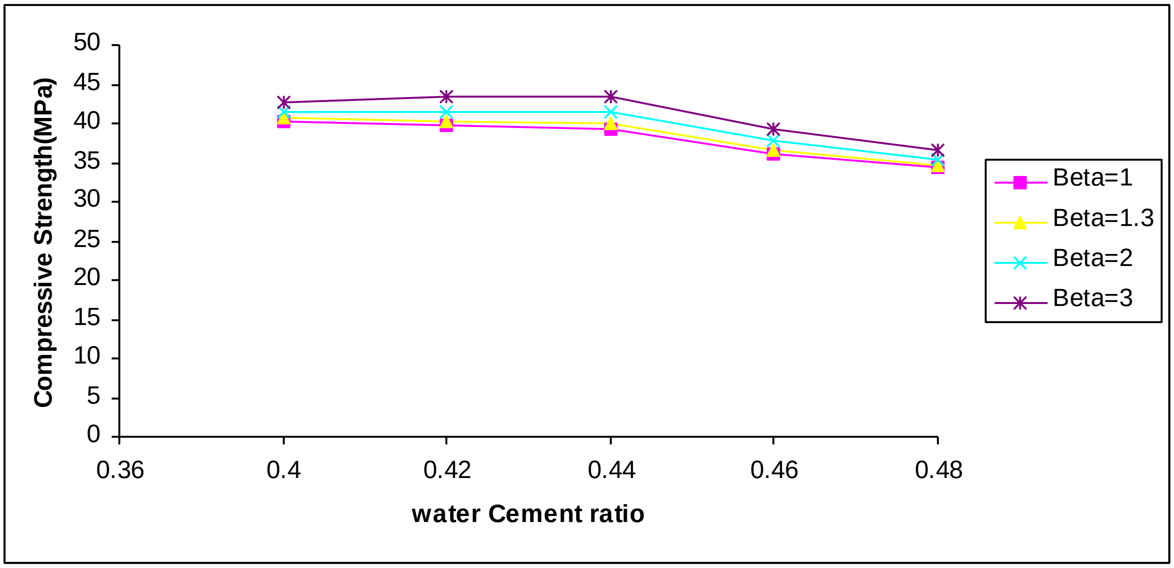
Without Replacement of cement by Fly ash



Normal Distribution

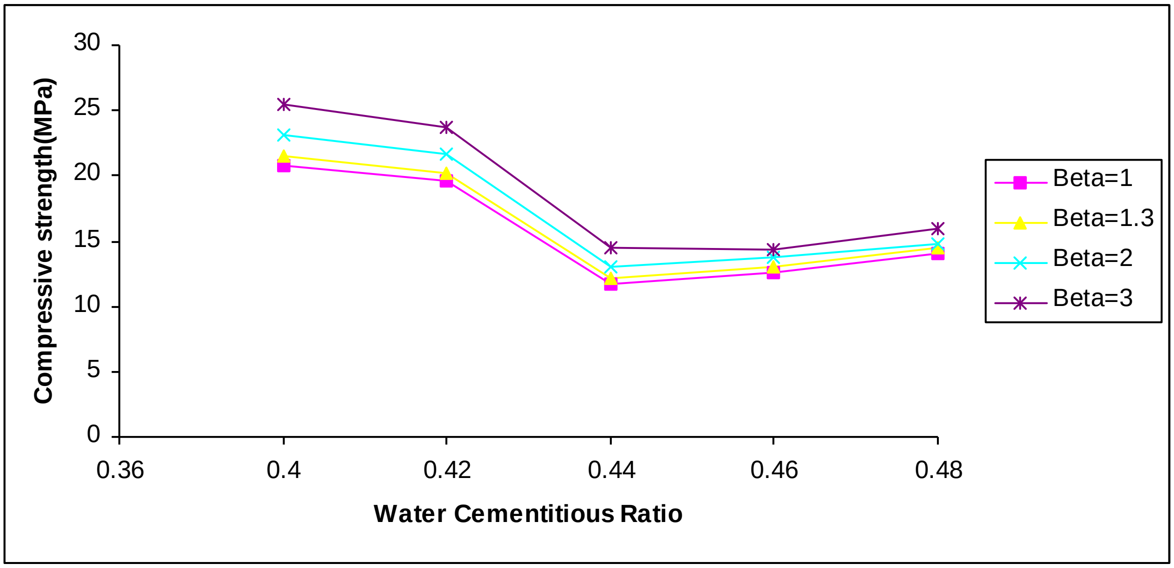
With 60 %Replacement of cement by Fly ash

Fig 5.5 Variation of 14 days Compressive strength with water cementitious material ratio (Fly ash 60%)



Normal Distribution

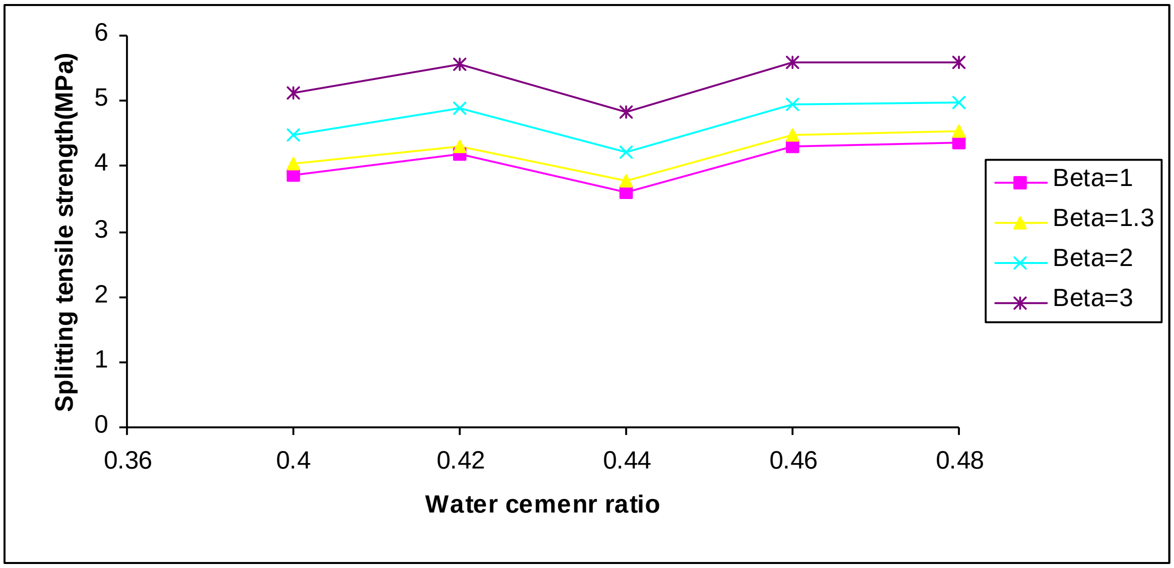
Without Replacement of cement by Fly ash



Normal Distribution

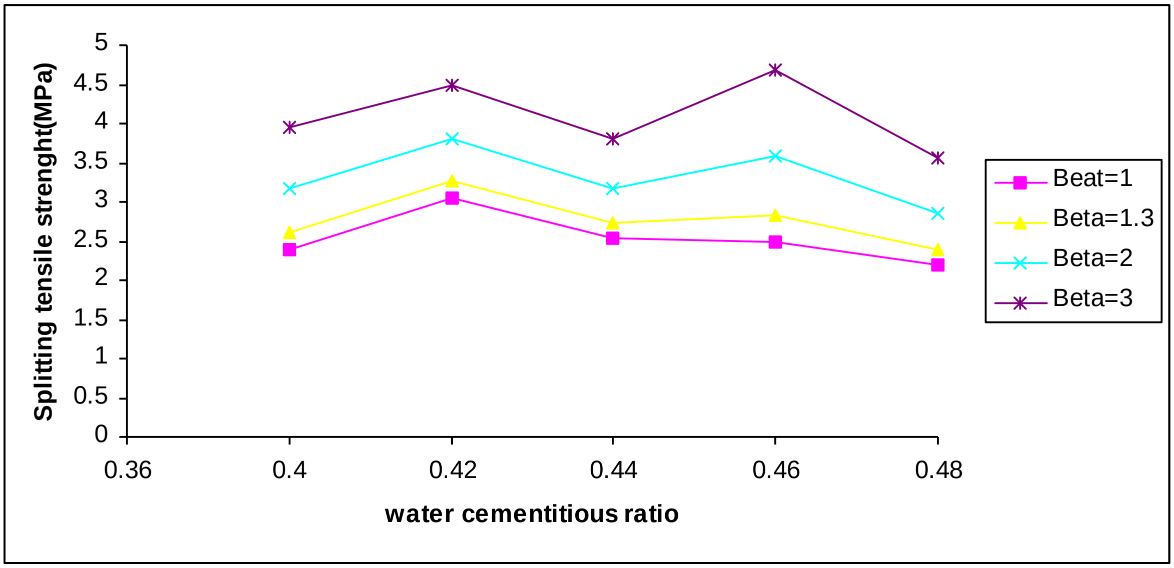
With 60 %Replacement of cement by Fly ash

Fig 5.6 Variation of 28 days Compressive strength with water cementitious material ratio (Fly ash 60%)



Normal Distribution

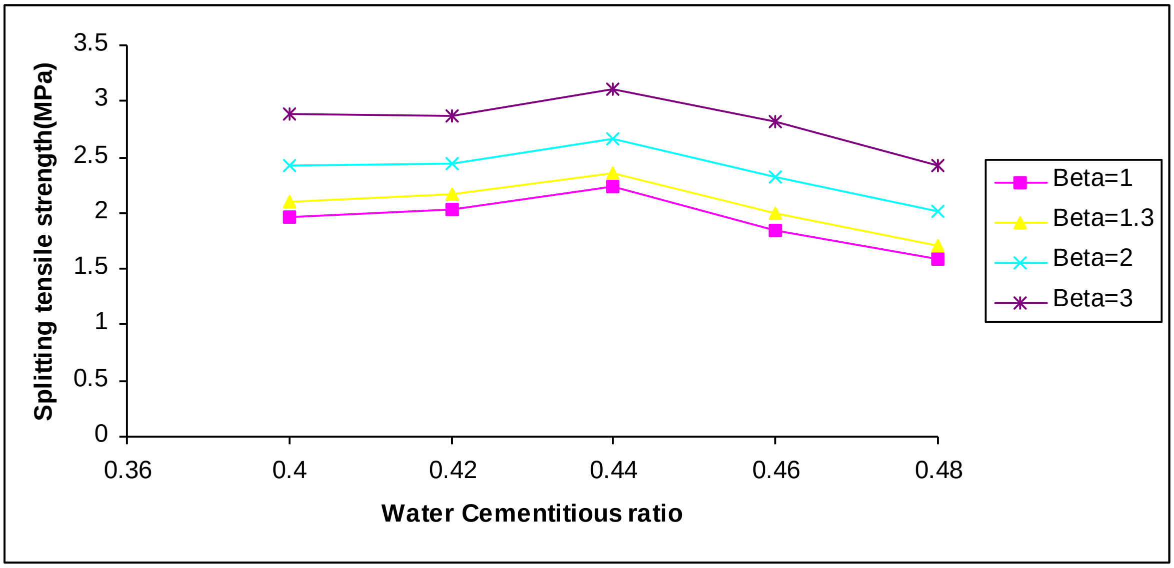
Without Replacement of cement by Fly ash



Normal Distribution

With 50% Replacement of cement by Fly ash

Fig 5.7 Variation of 28 days Splitting Tensile strength with water cementitious material ratio (Fly ash 50%)

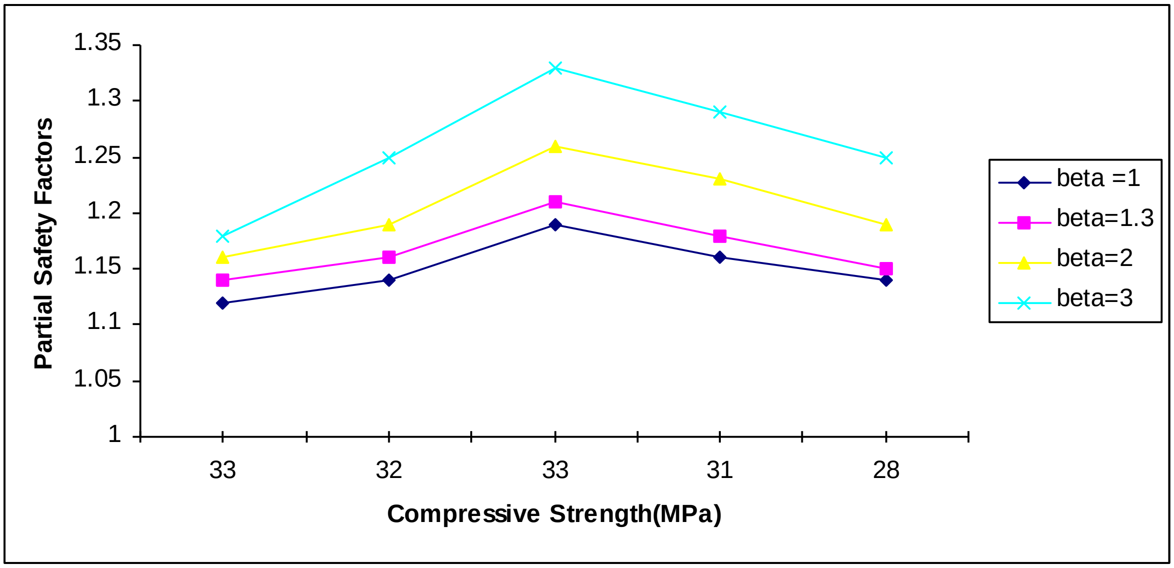


Normal Distribution

With 60% Replacement of cement by Fly ash

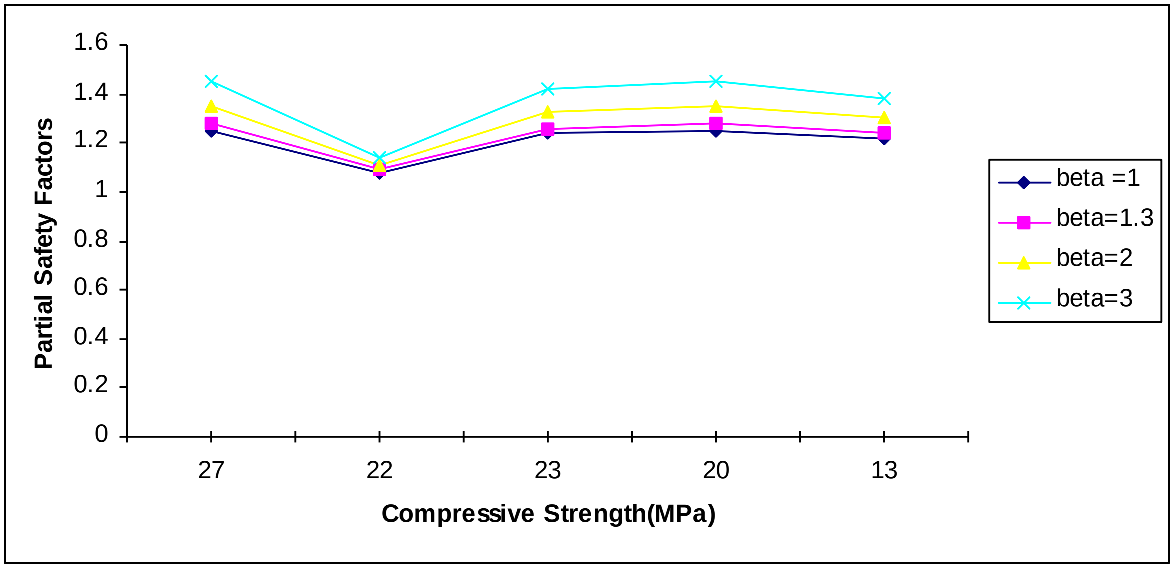
Fig 5.8 Variation of 28 days Splitting Tensile strength with water cementitious material ratio (Fly ash 60%)

Partial Safety factors for compressive strength of concrete



Normal Distribution

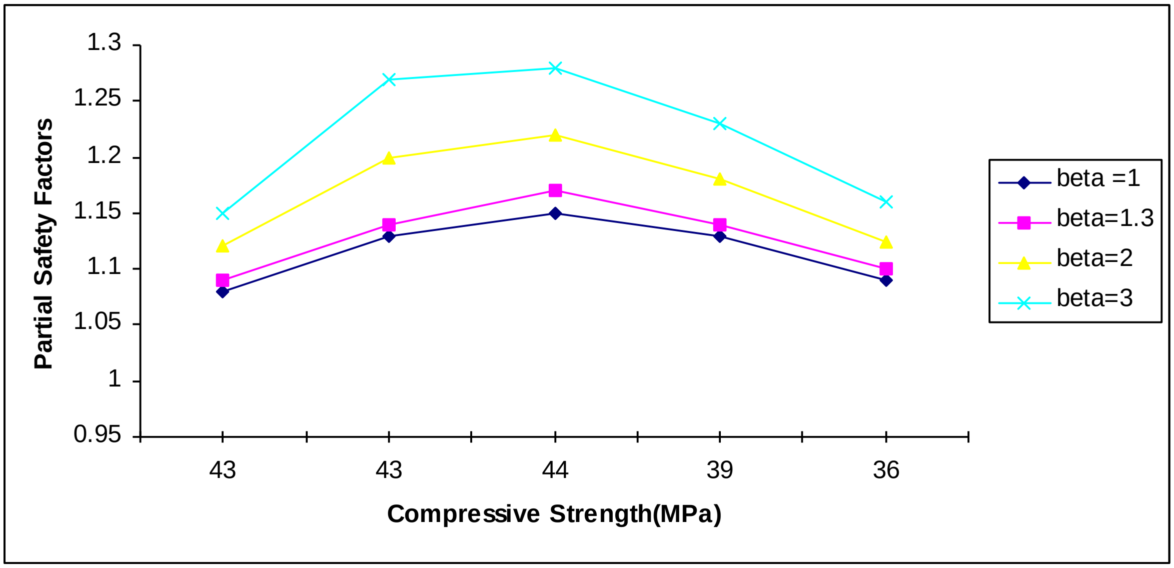
Without Replacement of cement by Fly ash



Normal Distribution

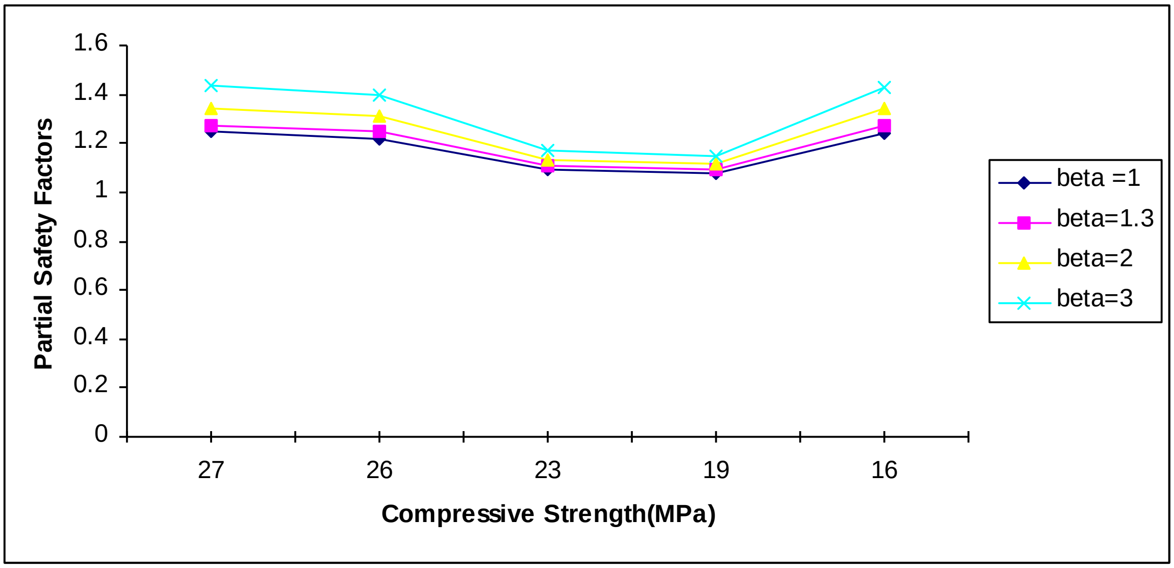
With 50% Replacement of cement by Fly ash

Fig 5.9 Variation of 7days Partial Safety factors with water cementitious material ratio (Fly ash 50%)



Normal Distribution

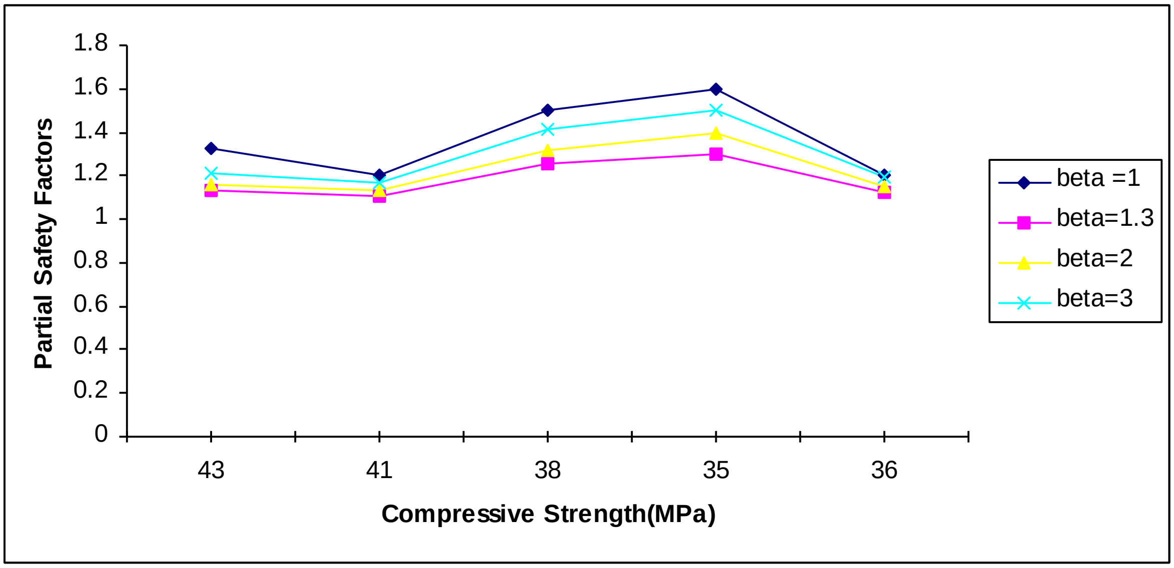
Without Replacement of cement by Fly ash



Normal Distribution

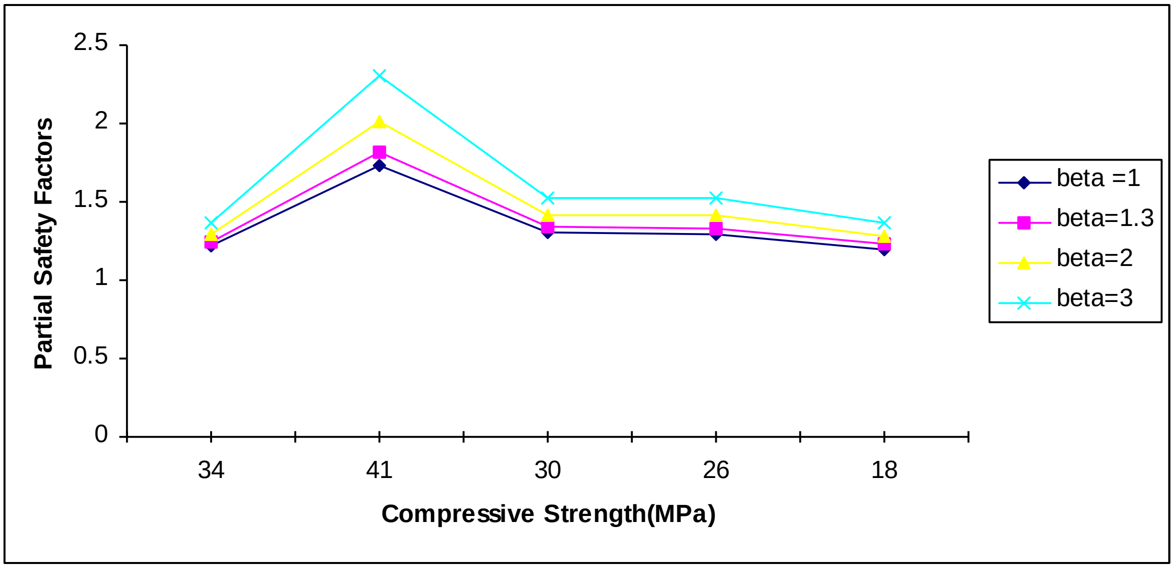
With 50% Replacement of cement by Fly ash

Fig 5.10 Variation of 14 days Partial Safety factors with water cementitious material ratio (Fly ash 50%)



Normal Distribution

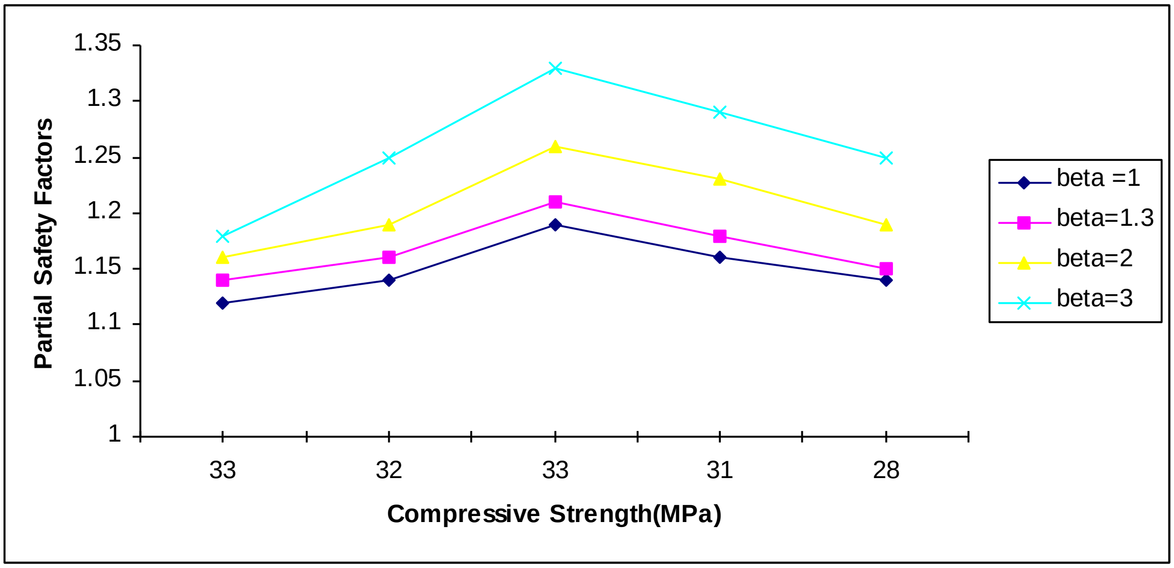
With 0% Replacement of cement by Fly ash



Normal Distribution

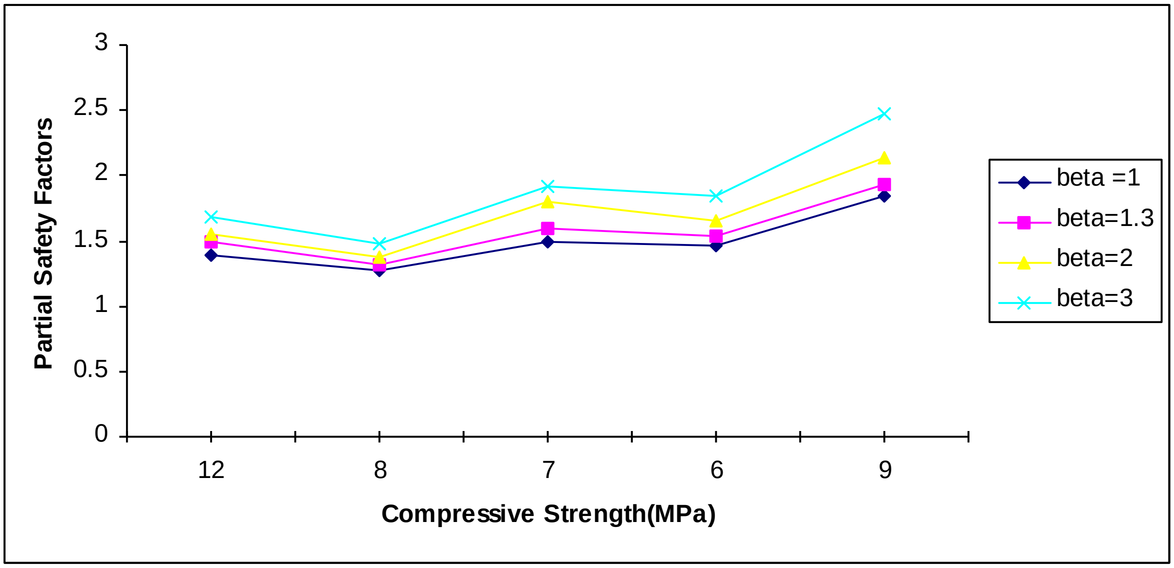
With 50% Replacement of cement by Fly ash

Fig 5.11 Variation of 28 days Partial Safety factors with water cementitious material ratio (Fly ash 50%)



Normal Distribution

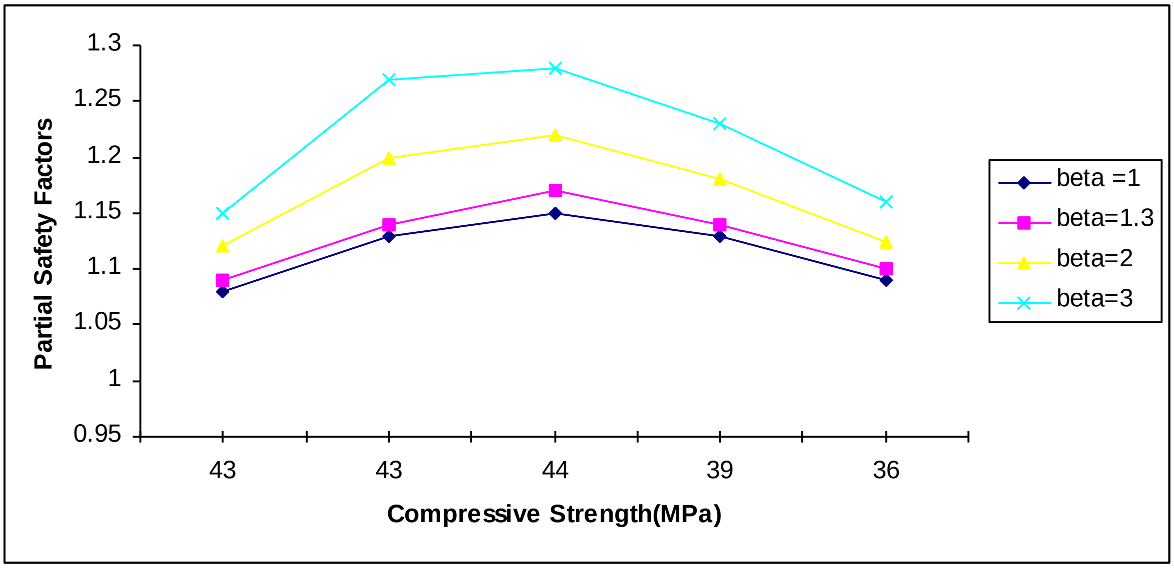
With 0% Replacement of cement by Fly ash



Normal Distribution

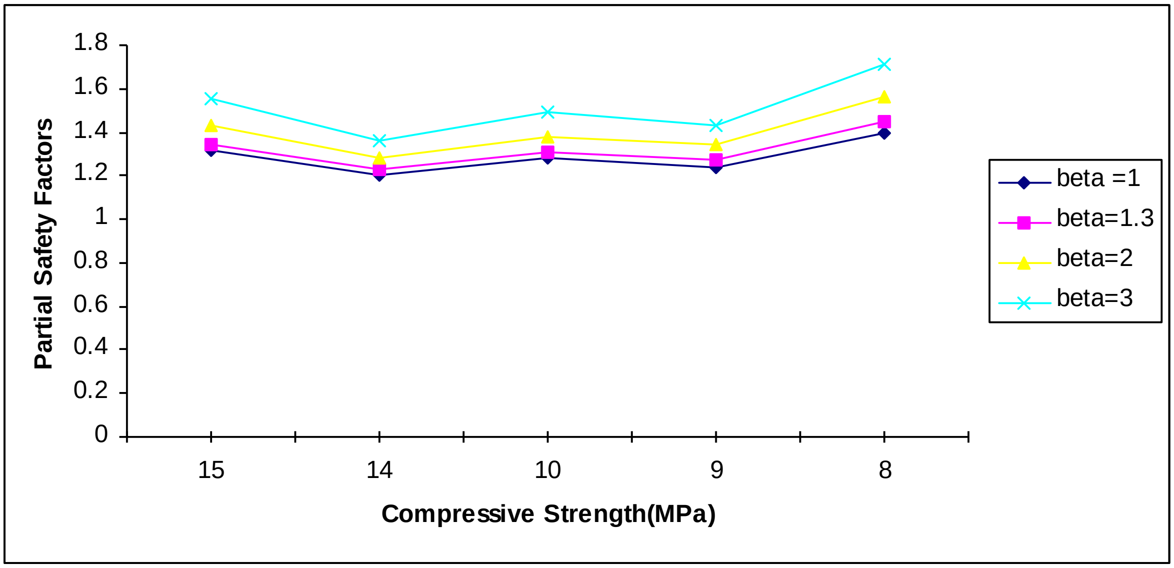
With 60% Replacement of cement by Fly ash

Fig 5.12 Variation of 7 days Partial Safety factors with water cementitious material ratio (Fly ash 60%)



Normal Distribution

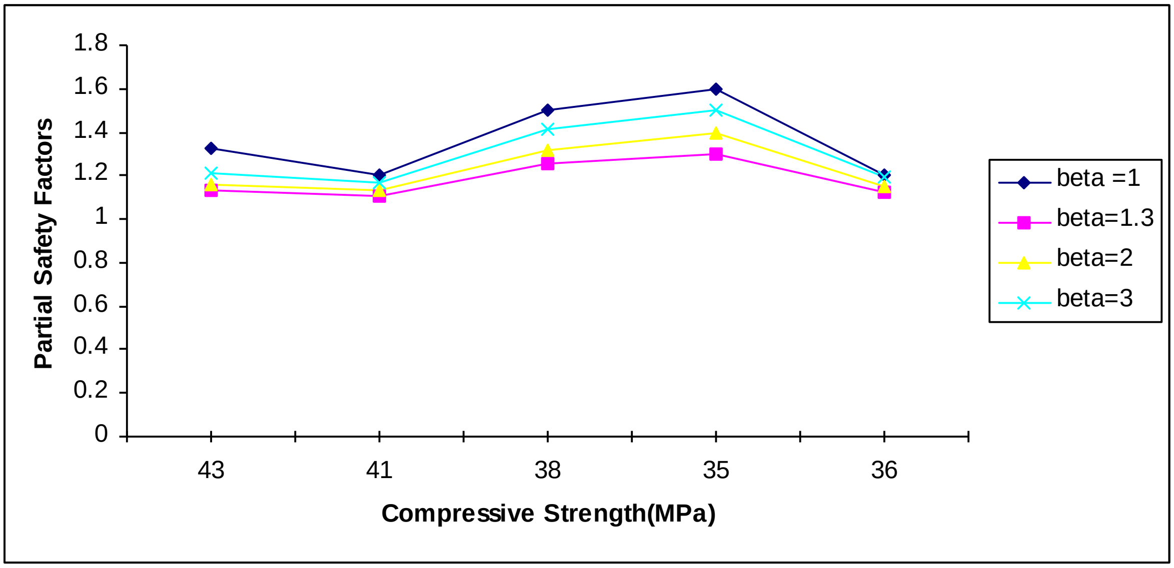
With 0% Replacement of cement by Fly ash



Normal Distribution

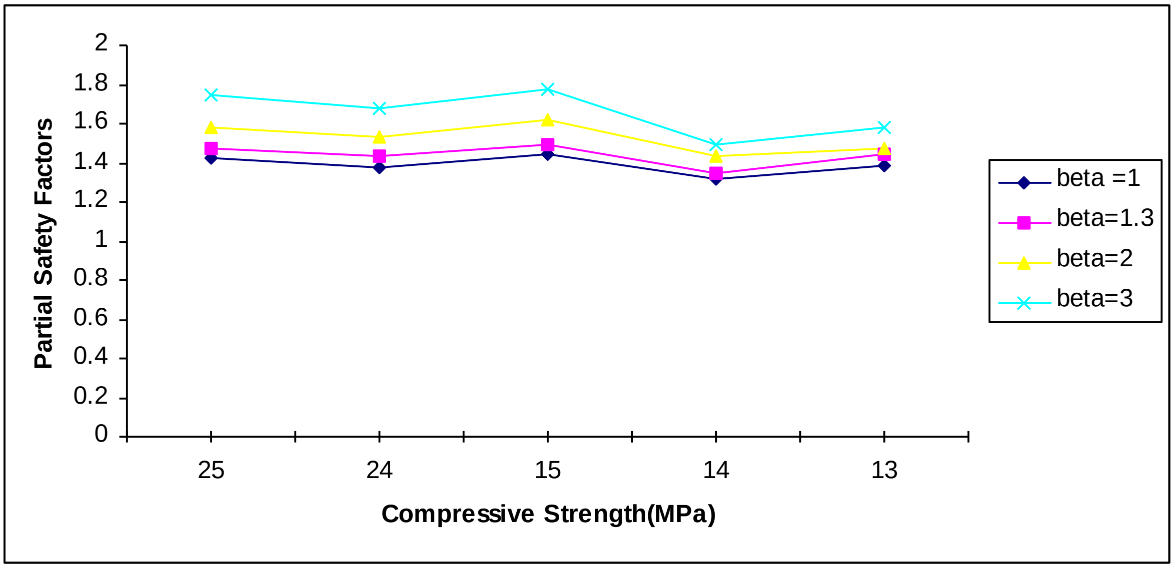
With 60% Replacement of cement by Fly ash

Fig 5.13 Variation of 14 days Partial Safety factors with water cementitious material ratio (Fly ash 60%)



Normal Distribution

With 0% Replacement of cement by Fly ash

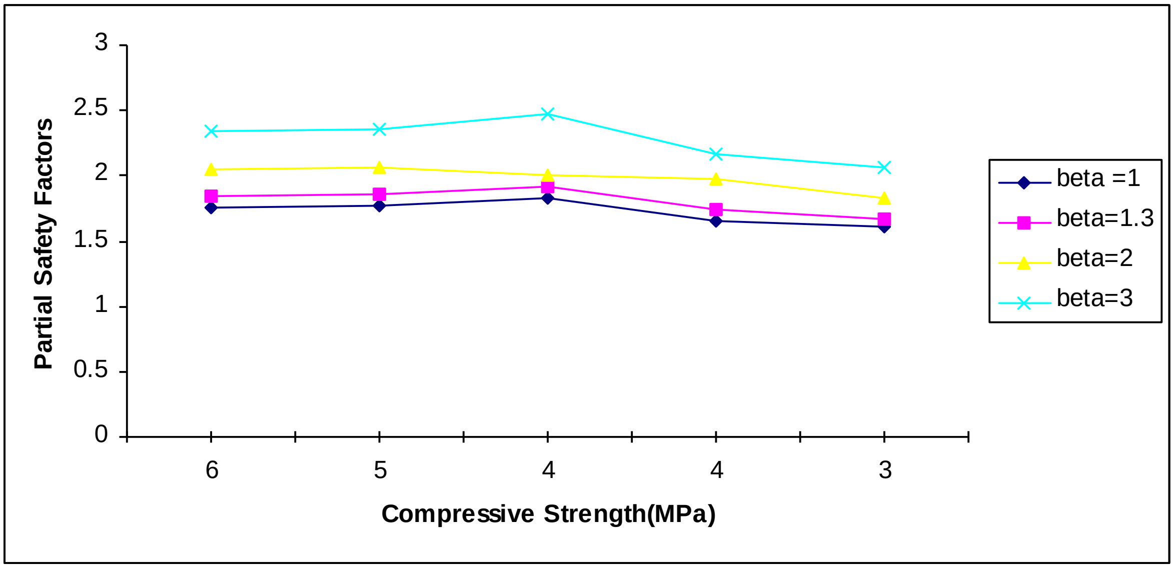


Normal Distribution

With 60% Replacement of cement by Fly ash

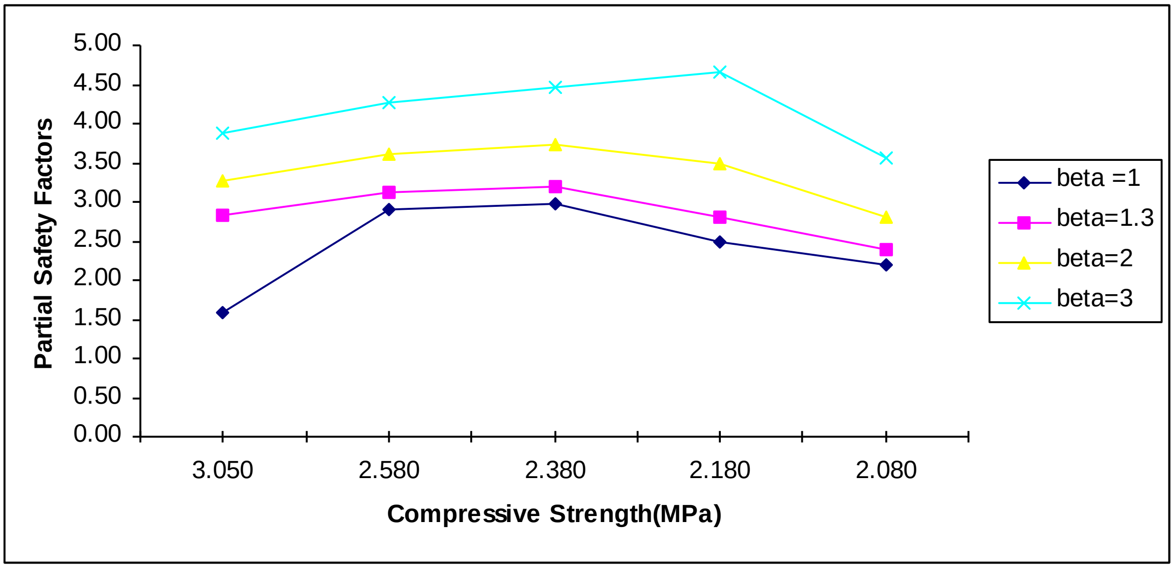
Fig 5.14 Variation of 28 days Partial Safety factors with water cementitious material ratio (Fly ash 60%)

Partial Safety Factors for Splitting Tensile strength of concrete



Normal Distribution

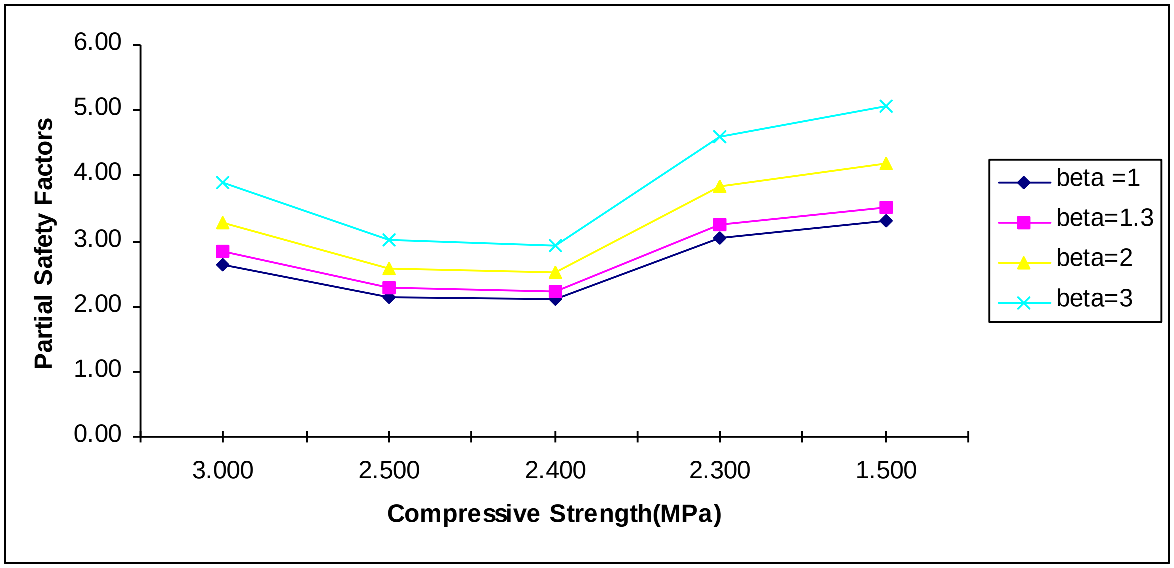
With 0% Replacement of cement by Fly ash



Normal Distribution

With 50% Replacement of cement by Fly ash

Fig 5.15 Variation of 28 days Partial Safety factors with water cementitious material ratio (Fly ash 50%)



Normal Distribution

With 60% Replacement of cement by Fly ash

Fig 5.16Variation of 28 days Partial Safety factors with water cementitious material ratio (Fly ash 60%)

CHAPTER 6 CONCLUSIONS

6.1 GENERAL

Partial safety factors for variation in the material strengths have been computed, in the present work, from the analysis of the experimental data generated. On the basis of the present study, following conclusions are drawn.

6.2 STATISTICAL PROPERTIES OF COMPRESSIVE STRENGTH

1) With-in-test coefficient of variation being less than the acceptance value of 3.0 per cent [ACI (1998)] indicates that the testing conditions were uniform. It implies that the testing conditions do not affect the results of compressive strength significantly.

2) The 7 and 28 days compressive strength of most of the concrete mixes with or without Fly ash have coefficient of variation lying between 3.0 to 5.0 per cent, whereas coefficient of variation for concrete mixes cured for 28 days is less than 3 per cent, also the coefficient of variation is less for mixes with Fly ash .

6.3 STRENGTH CHARACTERISTICS OF FIBROUS FLY ASH CONCRETE

(a) Compressive strength

1) Concrete mixes with 50 per cent and 60 per cent replacement of cement by fly ash designed for 7,14,28 days have less strength than concrete mixes without fly ash and fibres.

2) As the w/(C+p) ratio increases, the compressive strength of concrete mixes with 50 and 60 per cent fly ash decreases
3) Concrete mixes proportioned for higher reliability index require lower w/(C+p) ratio to ensure reduced probability of failure .

(b) Splitting Tensile Strength

1) Concrete mixes with 50 per cent and 60 per cent replacement of cement by fly ash designed for 28 days have less strength than concrete mixes without fly ash and fibres.
2) As the w/(C+p) ratio increases, the compressive strength of concrete mixes with 50 and 60 per cent fly ash decreases

6.4 PARTIAL SAFETY FACTORS

1) The partial safety factor increased with the increase in percentage of Fly ash in concrete, also signifying that partial safety factor increased with increase in compressive strength.
2) The concrete mixes with respect to 7 days compressive strength give higher partial safety factors than the concrete mixes proportioned with respect to 14 and 28 days.
3) Concrete mixes with Fly ash provide higher partial safety factors.
4) Partial safety factors vary exponentially with increase in reliability index .
5) The partial safety factors computed can be used in design of elements using concrete with fibres and fly ash.

SCOPE FOR FURTHER WORK

1) In the present study only up to 60 per cent replacement of cement by Fly ash has been considered. The other percentages i.e. 70 and 85 per cent need investigation.

2) In the present study the normal-distribution is considered for the strength of concrete mixes. The other probability distributions like lognormal-distribution etc. for strength of concrete also need investigations.
3) In this study only zone-A aggregates have been used. The other proportion of aggregates i.e. CA-I 50 per cent and CA-II 50 per cent, CA-II 50 per cent and CAIII (passing 4.75mm sieve and retained on 2.36mm sieve) 50 per cent also need investigations.

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