Studies on Concrete Made of Recycled Materials for Sustainability

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Studies on Concrete Made of Recycled Materials for Sustainability

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by

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(Roll Number: 512CE1005)

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under the supervision of

Prof. Pradip Sarkar

and

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This is to certify that the work presented in this dissertation entitled *Studies on Concrete Made of Recycled Materials for Sustainability* by *Kirtikanta Sahoo*, Roll Number: 512CE1005, is a record of original research carried out by him under our supervision and guidance in partial fulfillment of the requirements of the degree of *Doctor of philosophy* in *Civil Engineering*. Neither this dissertation nor any part of it has been submitted for any degree or diploma to any institute or university in India or abroad.

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MY FAMILY

Declaration of Originality

I, Kirtikanta Sahoo, Roll Number: 512CE1005 hereby declare that this dissertation entitled *Studies on Concrete Made of Recycled Materials for Sustainability* represents my original work carried out as a doctoral student of NIT Rourkela and, to the best of my knowledge, it contains no material previously published or written by another person, nor any material presented for the award of any degree or diploma of NIT Rourkela or any other institution. Any contribution made to this research by others, with whom I have worked at NIT Rourkela or elsewhere, is explicitly acknowledged in the dissertation. Works of other authors cited in this dissertation have been duly acknowledged under the section "Bibliography". I have also submitted my original research records to the scrutiny committee for evaluation of my dissertation.

I am fully aware that in case of my non-compliance detected in the future, the Senate of NIT Rourkela may withdraw the degree awarded to me on the basis of the present dissertation.

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Abstract

Construction industry uses Portland cement which is known to be a heavy contributor to the CO_2 emissions and environmental damage. Incorporation of industrial wastes like demolished old concrete, silica fume (SF) and fly ash (FA) as supplementary cementing materials (SCMs) could result in a substantial reduction of the overall CO_2 footprint of the final concrete product. However, use of these supplementary materials in construction industry especially in the making of concrete is highly challenging. Significant research efforts are required to study the engineering properties of concrete incorporating such industrial wastes. Present research is an effort to study the properties of concrete incorporating industrial wastes such as demolished concrete, SF and FA.

Recycled coarse aggregate (RCA) concrete construction technique can be called as ‘green concrete’, as it minimizes the environmental hazard of the concrete waste disposal. Indian standard recommends target mean compressive strength of the conventional concrete in terms of water cement ratio (*w/c*). The behaviour of RCA concrete, prepared from two samples of parent concrete having different age groups, is investigated, to propose the relationship of compressive strength with water cement ratios, in the present study. Number of recycling may influence the mechanical properties of RCA concrete. The influence of age and number of recycling on the properties such as capillary water absorption, drying shrinkage strain, air content, flexural strength and tensile splitting strength of the RCA concrete are examined. While the compressive strength reduces with number of recycling gradually, the capillary water absorption increases abruptly, which leads to the conclusion that further recycling may not be advisable.

Previous studies show that the properties of RCA concrete are inferior in quality compared to NCA concrete. The improvement of properties of RCA concrete with the addition of two ureolytic-type bacteria, *Bacillus subtilis* and *Bacillus sphaericus* to enhance the properties of RCA concrete. The experimental investigations are carried out to evaluate the improvement of the compressive strength, capillary water absorption and drying shrinkage of RCA concrete incorporating bacteria. The compressive strengths of RCA concrete are found to be increased by about 20% and 35% at the cell concentrations of 10^6 cells/ml for the two bacteria. The capillary water absorption as well as drying shrinkage of RCA are reduced when bacteria is incorporated. The improvement of RCA concrete is

confirmed to be due to the bacterial mineral precipitation as observed from the microstructure studies such as EDX, SEM and XRD.

The mechanical properties, such as compressive, flexural and tensile splitting strength, of SF concrete considering the 10% additional quantity of cement as recommended by International codes, by partial replacement of slag cement on low to medium strength concrete, have not been investigated so far. The present study investigates the mechanical properties of medium strength SF concrete made as per this construction practice by partial replacement of slag cement. Effect of SF on compressive, flexural and tensile splitting strength of hardened concrete is examined. Seven concrete mixes are prepared using Portland slag cement (PSC) partially replaced with SF ranging from 0 to 30%. The mix proportions were obtained as per Indian standard IS: 10262-2009 with 10% extra cement when SF is used as per the above the construction practice. Optimum dosages of SF for maximum values of compressive strength, tensile splitting strength and flexural strength at 28 days are determined. Results of the present study are compared with similar results available in literature associated with Portland cement. Relationships, in the form of simplified equations, between compressive, tensile splitting and flexural strengths of SF concrete are proposed.

Several studies related to sustainable concrete construction have encouraged the usage of industrial waste products such as SF and FA. Design of structures, made using such SF and FA concrete, for an acceptable level of safety, requires the probabilistic descriptions of its mechanical properties. For this purpose, an extensive experimental programme was carried out on compressive strength, flexural strength and tensile splitting strength properties of SF and FA concrete. The probability distribution models are proposed based on the three goodness-of-fit tests such as Kolmogorov-Sminrov, Chi-square and log-likelihood tests. The proposed probability distributions are used to study performance of typical buildings made of SF and FA concrete through seismic fragility curves and reliability indices.

Key Words: Concrete, Recycled coarse aggregate, Ureolytic bacteria, Silica fume, Fly ash, Variability, Fragility.

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Notations

μ	Mean
A	Air content
A_1	Apparent Air Content
C	Drift Capacity
D	Drift Demand
f_{cc}	Compressive Strength
f_{fl}	Flexural Strength
f_{sp}	Tensile Splitting
$FR(X)$	Seismic Fragility
G	Aggregate Correction Factor
S	Coefficient of Capillary Water Absorption
Sa	Spectral Acceleration
Sd	Spectral Displacement
t	time of immersion
w/c	Water Cement ratio.
x	Random Variable
α	Shape Factor
ΔW	Cumulative amount of water absorbed
λ,β	Scale Factor
σ	Standard Deviation

Note: - The symbols and abbreviations other than above have been explained in the text.

Abbreviation

ACI	American Concrete Institute
CDF	Cumulative Distribution Function
CPA	Calcite Precipitation Agar.
CS	Chi-square
C-S-H	Hydrated Calcium Silicate
CT	Control Cubes
EDP	Engineering Demand parameter
EDX	Energy Dispersive X-ray Spectroscopy
FA	Fly Ash
FESEM	Field Emission Scanning Electron Microscopy
HSC	High Strength Concrete
HVFA	High Volume Fly Ash
IM	Intensity Measure
KS	Kolmogorov- Smirnov
LK	Log-likelihood
LS	Limit State
LRC	Lightly Reinforced Concrete
NCA	Natural Coarse Aggregate
OD	Optical density
PGA	Peak Ground Acceleration
PSC	Portland Slag Cement
PSDM	Probabilistic Seismic Demand Model
RC	Recycled Concrete
RCA	Recycled Coarse Aggregate.
SCM	Supplementary Cementing Materials
SEM	Scanning Electron Microscopy
SF	Silica Fume
TASC	Tubular Aerosol Suspension Chamber
UPV	Ultrasonic Pulse Velocity
XRD	X-Ray Diffraction

Chapter 1

Introduction

1.1 Background and Motivation

Most engineering constructions are not eco-friendly. Construction industry uses Portland cement which is known to be a heavy contributor to the CO_2 emissions and environmental damage. In India, amount of construction has rapidly increased since last two decades. Using various types of supplementary cementing materials (SCMs), especially SF and FA, as a cement replacement could result in a substantial reduction of the overall CO_2 footprint of the final concrete product. Lesser the quantity of Portland cement used in concrete production, lesser will be the impact of the concrete industry on the environment.

The deposition of construction garbage which is increasingly accumulated due to various causes such as demolition of old construction is also an environmental concern [Topcu and Guncan 1995]. In India, the Central Pollution Control Board has assessed that the solid waste generation is about 48 million tonnes per annum of which 25% are from the construction industry. This scenario is not so different in the rest of the world. In order to decrease the construction waste, recycling of waste concrete as aggregate is beneficial and effective for preservation of natural resources [Khalaf and Venny 2004].

Usage of demolished concrete, SF and FA in construction industry is more holistic as it contributes to the ecological balance. However, use of these waste materials in construction industry especially in the making of concrete is highly challenging. Significant research efforts are required to study the engineering properties of concrete made of such industrial wastes. Present research is an effort to study the properties of concrete incorporating industrial wastes such as demolished concrete, SF and FA.

Demolished concrete can be used as recycled coarse aggregate (RCA) to make new concrete (RCA concrete) by partially or fully replacing the natural coarse aggregate (NCA). Various researchers have examined the physical and mechanical properties of RCA concrete and found that the mechanical strength of the RCA concrete is lower than that of conventional concrete with NCA. This is due to the highly porous nature of the

RCA compared to NCA and the amount of replacement of NCA [Rahal 2007]. The physical properties of the RCA depend on the amount of adhered mortar and its quality. Amount of adhered mortar depends on the process of crushing of parent concrete. Due to these reasons, RCA shows more porosity, more water absorption, low density and low strength as compared to the natural aggregate. Previous researchers reported that up to 25% reduction in compressive strength has been occurred due to above reasons [Amnon 2003; Elhakam *et al*. 2012; Tabsh and Abdelfatah 2009, McNeil and Kang 2013].

The relationship between the water-to-cement (*w*/*c*) ratio and the compressive strength is essential for the preliminary estimation of water and other constituent materials for mix design of concrete. Indian standard recommends such relationship for NCA concrete. This relationship may be different for RCA concrete depending on its age and number of recycling. Many studies [Rahal 2007; Amnon 2003; Tabsh and Abdelfatah 2009; Kou *et al*. 2011, Kou and poon 2009; and Padmini *et al*. 2009] are reported in literature that focuses on the behaviour, properties, and functional uses of RCA. However, no studies have been reported on the behaviour of RCA concrete with regard to above aspects. The present work is an attempt to study the relationship of w/c ratio with compressive strength considering age and number of recycling of RCA.

The rising tide of adoption of RCA for construction demands an investigation of methods to improve the quality of RCA concrete. Use of urease-producing bacteria can address the problems associated with RCA concrete to some extent. Such bacteria can precipitate $CaCO_3$ through urease activity [Pei *et al*. 2013; Pacheco-Torgal and Labrincha 2013; and Siddique and Chahal 2011] which catalyzes the hydrolysis of urea into ammonium and carbonate. First, urea is hydrolyzed intracellular to carbamate and ammonia. Carbamate spontaneously hydrolyzes to form additional ammonia and carbonic acid. These products subsequently form bicarbonate, ammonium, and hydroxide ions. These reactions increase the ambient pH, which in turn shifts the bicarbonate equilibrium, resulting in the formation of carbonate ions. This leads to accumulation of insoluble $CaCO_3$, which fills up the pores of the concrete and improves the impermeability and strength. Bacterial calcium carbonate mineralization using urease producing bacteria is proposed in the present study to improve the quality of RCA concrete.

Like all other pozzolanic materials, SF is capable of reacting with the calcium hydroxide, $Ca(OH)_2$ liberated during cement hydration to produce hydrated calcium silicate (C–S–H), which is accountable for the strength of hardened concrete. The high content of very fine amorphous spherical (100 nm average diameter) silicon dioxide particles (present more than 80%) is the main reason for high pozzolanic activity of SF. The SF can improve both chemical and physical properties, which transform the microstructure of concrete and hence reduce the permeability and increase the strength. Most of the previous studies on the SF concrete are conducted using Portland cement for high strength concrete applications. International codes [ACI 234R-96] recommend an additional 10% of cement when SF is used as partial replacement of cement in the construction practice. The mechanical properties of SF concrete considering the 10% additional quantity of cement as recommended by International codes, incorporating slag cement on low to medium strength concrete, have not been investigated so far. The present study investigates the mechanical properties of medium strength SF concrete made as per this construction practice using slag cement.

Randomness and variability of material properties can considerably affect structural performance and safety. In contradiction to reality this phenomenon is usually neglected, in conventional structural analysis and design that assume deterministic values of material properties. This assumption makes the analysis models less realistic and less satisfactory. With the advancement of computing facilities, the complex structural analyses including the probabilistic nature of the various parameters of the structure are not difficult and have become essential for its response against natural loads like earthquake, wind, etc. There are many studies [Campbell and Tobin 1967; Soroka 1968; Chmielewski and Konapka 1999; and Graybeal and Davis 2008] reported on the variability of compressive strength of concrete. The variability of compressive strength of concrete usually represented in literatures by a normal distribution if the coefficient of variation does not exceed 15-20%, although slight skewness may be present. However, when the coefficient of variation is high, the skewness is considerable [Campbell and Tobin 1967] and if the quality control is poor [Soroka 1968], a lognormal distribution is more rational to represent the tail areas of distribution than a normal distribution. A recent study [Chen *et al.* 2014] concludes that the variation in concrete compressive strength should be characterized using various statistical criteria and different distribution functions.

The inherent variability of cement and SF may not be similar in nature, as SF is a by-product in the carbothermic reduction of high-purity quartz with carbonaceous materials like coal, coke, wood-chips in the production of silicon and ferrosilicon alloys. Therefore, existing literatures on the variability of cement concrete may not be useful to describe the variability of concrete with SF. One of the focus of the present study is to describe the variability of concrete using SF by finding out a best fitted probability distribution matching the experimental data. An attempt has been made to study the seismic behaviour of typical RC structures through fragility analysis considering the variability of the SF concrete obtained from experiments.

FA, which is another material used to supplement cement popularly to produce concrete. A part of the present study is devoted to investigate the above described properties for FA concrete also.

1.2 Objectives

Based on a detailed literature review (presented in Chapter 2), the major objective of the present research work is identified as the investigation of properties of concrete made using various alternative materials (RCA, SF and FA) and its possible enhancement. Following are the sub-objectives to achieve the major goal.

i. To study the relationship of *w/c* ratio and compressive strength, the effects of age and number of recycling on the properties of RCA concrete.
ii. To study the enhancement of engineering properties of RCA concrete using bacteria.
iii. To investigate the mechanical properties of low to medium strength SF concrete incorporated with 10% additional cement quantity as per the construction practice.
iv. To describe the variability in the properties of both SF and FA concrete and its implications in the seismic behaviour of typical building structures through fragility analysis.

1.3 Scope

Following are the scopes and limitations of the present study

1. Present construction industry uses slag cement over ordinary Portland cement. 90% of the cement used in Indian construction industry are of slag cements. Present research, therefore, considers only slag cement for all the studies.
2. Only low to medium strength concrete are considered in the present study as the usage of this type of concrete is higher compared to high strength concrete.
3. Only two parameter probability distributions are considered for the description of variability of SF and FA concrete.

4. Only three statistical goodness of tests such as Kolmogorov-Sminrov, Chi-square and log-likelihood tests are used for evaluation of best –fit probability distribution models.

1.4 Methodology

In order to achieve the above objectives following step by step methodology is adopted:

1. Prepare RCA from demolished concrete, prepare test specimens and perform different tests to evaluate the effect of age and number of recycling on the properties of RCA concrete.
2. Culture of bacteria in the laboratory, incorporate them on the RCA concrete to enhance the properties.
3. Perform micro-structure analyses such as X-ray diffraction (XRD) and field emission scanning electron microscopy (FESEM) to relate the morphology and microstructure of bacterial concrete to its mechanical properties.
4. Design the mix proportion for SF and FA concrete and evaluate their mechanical properties.
5. Propose probability distribution models for the description of variability in mechanical properties of SF and FA based on goodness of fit tests
6. Study the behaviour of typical building structures through seismic fragility analysis using the proposed probability distribution models.

1.5 Novelty of the Present Work

This research is focussed on following important aspects which were not reported in any published literature:

(i) The effect of successive recycling of coarse aggregate on the properties of concrete has been carried out.

(ii) Although the construction industry have shifted from OPC to PSC in the concrete making worldwide, the research focus is still surprisingly limited to OPC to a great extent. Studies on RCA/bacterial/SF/FA concrete using PSC makes this research meaningful.

(iii) Design philosophy for concrete structure is moving towards performance-based design which requires probabilistic description of material properties, structural model and design loads. Although a large amount of literature exist in the domain of SF/FA concrete there is no probabilistic description of properties of such concrete reported in published literature. This research attempted to fill this gap with an extensive study.

(iv) This research is also demonstrated the importance of the probabilistic models through a case study of fragility and reliability analyses of building made of SF/FA concrete.

1.6 Organisation of the Thesis

This introductory chapter has presented the background, objective, scope and methodology of the present study.

Chapter 2 starts with review of various literature on RCA concrete and enhancement of its properties. Later this chapter reviews the literatures available on the study of enhancement of properties of concrete using bacteria. After that, it presents the review of the various studies carried on supplementing cement materials like SF and FA. Finally it discusses published literature on variability of the mechanical properties of concrete. Last part of this Chapter presents the experimental techniques used in the present study.

Chapter 3 presents the results of experiments on RCA concrete with emphasis on age and number of recycling of RCA. This Chapter also presents the results of RCA concrete incorporating bacteria. To study the effect of bacteria on the concrete specifically some of the tests are conducted on cement mortar. This Chapter presents the results of those tests also.

Chapter 4 presents the experimental results of the mechanical properties of SF and FA concrete.

Chapter 5 presents studies on variability of mechanical properties of SF and FA concrete obtained experimentally. Last part of this chapter discusses the behaviour of typical building structures through fragility analysis.

Finally, Chapter 6 presents summary and significant contributions of this research. It also presents future scope of this research work.

Chapter 2

Literature Review

2.1 General

Literature review for the present study is carried out broadly in the direction of concrete made of recycled materials for sustainability. The present study uses bacteria for the improvement of RCA concrete. The investigations are carried out in the present study to assess the mechanical properties of RCA concrete, SF concrete and FA concrete. The variability characteristics of the concrete made from SF and FA and its effect on fragility curves are also examined in this study. For the presentation purpose, the literature review is divided in six segments such as (i) studies in RCA concrete, (ii) studies on application of bacteria to improve the properties of normal concrete, (iii) studies on mechanical properties of SF and FA concrete (iv) studies of variability of normal concrete (v) studies on fragility curves (vi) review of experimental methods used in the present study.

2.2 Studies on RCA Concrete

Crushed concrete that results from the demolition of old structures is generated nowadays in large quantities. The current annual rate of generation of construction waste is 145 million tonnes worldwide [Revathi *et al.* 2013]. The area required for land-filling this amount of waste is enormous. Therefore, recycling of construction waste is vital, both to reduce the amount of open land needed for land-filling and to preserve the environment through resource conservation [Revathi *et al.* 2013, Pacheco-Torgal *et al*. 2013]. It has been widely reported that recycling reduces energy consumption, pollution, global warming, greenhouse gas emission as well as cost [Khalaf and Venny 2004; Pacheco-Torgal and Said 2011; Ameri and Behnood 2012; Vázquez 2013; Behnood *et al.* 2015; Pepe 2015 and Behnood *et al.* 2015]. This in turn is beneficial and effective for environmental preservation

Various researchers have examined about the physical and mechanical properties of the RCA and its influence when natural aggregate is replaced partially or fully by RCA to make concrete. It has been found that the mechanical strength of the RCA concrete is lower than that of conventional concrete. This is due to the highly porous nature of the RCA compared to natural aggregates and the amount of replacement against the natural aggregate [Rahal 2007, Brito and Saikia 2013].

The physical properties of the RCA depend mainly on the adhered mortar and generally RCA shows more porosity, more water absorption, low density and low strength as compared to the natural aggregate concrete. It is reported that up to 25% reduction in compressive strength has been occurred due to above reasons [Amnon 2003; Tabsh and Abdelfatah 2009; Elhakam *et al.* 2012; McNeil and Kang 2013].

Barbudo *et al.* (2013) studied the influence of the water reducing admixture on the mechanical performance of the recycled concrete. This study shows that use of plasticizers may improve the properties of recycled concrete. Rahal (2007) investigated the mechanical properties of recycled aggregate concrete in comparison with natural aggregate concrete.

Tabsh and Abdelfatah (2009) studied the behaviour of recycled aggregate and their mechanical properties. It is reported that the strength of recycled concrete can be 10–25% lower than that of natural aggregate concrete. It is reported that though the recycled aggregate are inferior to natural aggregate, their properties can be considered to be within the acceptable limits.

Kou *et al.* (2011) investigated the long term mechanical properties and pore size distribution of the recycled aggregate concrete. It is reported that after 5 years of curing, the recycled aggregate concrete had lower compressive strength and higher splitting tensile strength than that of the natural aggregate concrete.

Kou and Poon (2009) studied the self-compacting concrete made from both recycled coarse and fine recycled aggregate. The different tests covering fresh, hardened and durability properties were investigated and the results show that both fine and coarse recycled aggregates can be used in self-compacting concrete. The similar observation was also made by Grdic *et al.* (2010).

Li (2009) has developed mix design for pervious recycled concrete with compressive strength and water seepage velocity as verification indexes. The Volume of voids is also tested for feasibility of new proposed mix design. Fathifazl *et al.* (2009) proposed a new method of mixture proportioning for concrete made with coarse recycled concrete aggregates. The new method was named as “equivalent mortar volume” in which the total mortar volume was kept constant.

Bairagi *et al.* (1990) proposed a method of mix design for recycled aggregate concrete from the available conventional methods. It has been suggested that the cement required was about 10% more in view of the inferior quality aggregate.

The adhered mortar forms a weak porous interface, which influences the strength and performance of RCA concrete [Ollivier *et al.* 1995; Prokopski and Halbiniak 2000; and Tam *et al*. 2005] and subsequently results in concrete with lower quality [Mehta and Aitcin 1990; Bentz and Garboczi 1991; Aitcin and Neville 1993; Alexander, 1996; Buch *et al.* 2000; Kwan *et al.* 1999]. This is considered to be one of the most significant differences between RCA and NCA concrete.

It has been reported that concrete made with 100% recycled aggregates is weaker than concrete made with natural aggregates at the same water to cement ratio (*w*/*c*) and same cement type. Many published literature [Amnon, 2003; Tabsh and Abdelfatah, 2009; Elhakam *et al.* 2012 and McNeil and Kang, 2013] reported that RCA concrete with no NCA reduces the compressive strength by a maximum of 25% in comparison with NCA concrete. A similar trend was observed in the case of tensile splitting strength and flexural strength [Silva *et al*. 2015].

Wardeh *et al.* (2014) carried out an experimental program on RCA concrete according to the mix design method given in Eurocode 2. Sriravindrarajah *et al.* (2012) proposed a mix design for pervious concrete and developed an empirical relationship between porosity, compressive strength and water permeability. Brito and Alves (2010) studied the correlation of mechanical properties, density and water absorption of RCA concrete.

Lauritzen (1993) and Dhir *et al.* (1999) reported that RCA concrete requires more water for the same workability as compared to NCA concrete. Hansen, 1986; found that density, compressive strength and modulus of elasticity of RCA concrete are relatively lesser than that of the parent concrete. RCA concrete results in higher permeability, rate of carbonation and risk of reinforcement corrosion than NCA concrete for a given w/c ratio.

Gayarre *et al.* (2015) studied the variation of w/c ratio of some mechanical properties of concrete. The results showed a significant decrease in mechanical properties with an increase of *w*/*c* ratio when natural aggregates are completely replaced by recycled aggregates. Gayarre *et al.* (2014) investigated the effect of different curing conditions on the compressive strength of RCA concrete and showed compressive strength of RCA concrete is reduced up to 20% when cured in open-air conditions.

There are several techniques available in the literature [Achtemichuk 2009; Berndt 2009; González-Fonteboa *et al.* 2009; Kou and Poon 2012 and Limbachiya *et al.* 2012] to enhance the properties of RCA concrete such as partial replacement of cement with SF and FA, addition of nanoparticles, etc. However, use of bacteria to enhance the properties of RCA concrete is not attempted by any previous researchers. Similar studies on NCA concrete are also found to be very limited.

2.3 Studies on Bacterial Concrete

Bio-mineralization has been used for many years in several engineering applications. One encouraging bio-mimetic process in nature is the conversion of sand to sandstone by soil thriving bacteria [Dick *et al.* 2006]. Later it was found that this conversion was done by *Bacillus pasteurii*, which precipitate calcite that acts as a binding material for the limestone. Introducing a calcite precipitating bacteria can thus meet the need to improve the strength. The improvement of soil bearing capacity by microbial calcite precipitation is reported by [Whiffin *et al.* 2007].

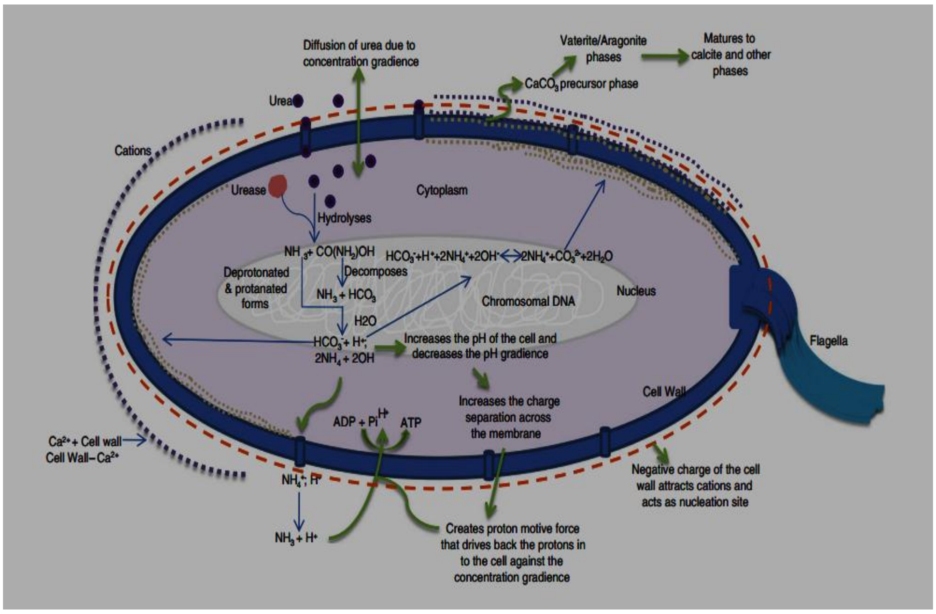


Figure 2.1: Mechanism of calcite precipitation by bacterium [Sarayu *et al.* 2014]

Microbial mineral precipitation using ureolytic bacteria was reported to improve the overall behaviour of concrete including strength and durability [Bachmeier *et al.* 2002; Muynck *et al.* 2008; Achal *et al.* 2009; Sung-Jin *et al.* 2010; Siddique and Chahal 2011; Majumdar *et al.* 2012; Grabiec *et al.* 2012; Pacheco-Torgal and Labrincha 2013;

Vekariya *et al.* 2013; Achal *et al.* 2013; Sujatha *et al.* 2014]. Bacteria can be used externally as a healing agent on hardened concrete for sulphate treatment [Wiktor *et al.* 2011]. The microbially induced precipitation can resist the carbonation and chloride ingress in concrete [Muynck *et al.* 2008, Pacheco-Torgal *et al.* 2015]. Bio-mineralization has also been used as an alternative and environmental friendly crack repair technique [Bang *et al.* 2001; Muynck *et al.* 2007; Achal *et al.* 2011 and Xu *et al.* 2014]. *Bacillus subtilis* bacteria can precipitate $CaCO_3$ through urease activity [Siddique and Chahal. 2011; Pei *et al.* 2013 and Pacheco-Torgal and Labrincha 2013] which catalyses the hydrolysis of urea into ammonium and carbonate. Fig. 2.1 shows the schematic diagram of the mechanism of calcite precipitation. A brief discussions on the mechanism of bio-mineralisation is described in Appendix A (Section A.2)

Pacheco-Torgal and Labrincha (2013) summarises some bacteria are capable of naturally precipitating calcium carbonates. The precipitation is due to several activities of bacteria and fungi such as photosynthesis, ammonification, denitrification, sulphate reduction and anaerobic sulphide oxidation [Castainer *et al.* 2000 and Riding, 2000]. From a majority of the experiments reported in literature, it is seen that bacteria of the genus Bacillus are used as an agent for the biological production of calcium carbonate based minerals.

Also, bacteria is found to be used in previous studies [Zhong and Yao 2008] for healing cracks due to the precipitation of calcium carbonate. Ramachandran *et al.* (2001) reported that the durability of the concrete was enhanced with an increase in bacterial concentration.

Chahal *et al.* (2012) investigated the influence of the ureolytic bacteria (Sporosarcina pasteurii) on the compressive strength, water absorption and chloride permeability of concrete incorporating SF and FA. A cell concentration of 10^5 cells/ml was found to be the optimum dose of bacteria to enhance the compressive strength and reduce the permeability of NCA concrete. [Kim *et al.* 2013] investigated the characteristics of microbiological precipitation of calcium carbonate on normal and lightweight concrete by two types of bacteria, *Sporosarcina pasteurii* and *Bacillus sphaericus*. It is observed that *Bacillus sphaericus* precipitated thicker calcium carbonate crystals than *Sporosarcina pasteurii*.

The quality of RCA concrete may be improved by bacterial mineralization, it is proposed to study the mechanical properties of RCA concrete incorporating two bacteria, namely *Bacillus subtilis* and *Bacillus Sphaericus*.

2.4 Studies on SF and FA Concrete

Usage of substitute minerals in concrete helps the conservation of raw materials, reduces CO_2 emissions and ultimately helps to a cleaner environment. Increased use of supplementary cementing materials in place of cement in concrete structures worldwide contributes to sustainability in construction. SF, a by-product of silicon metal, and FA, a by-product of thermal power stations are the two globally available supplementary cementitious materials which possess pozzolanic properties [Yeginobal *et al.* 1997; Bilodeau and Malhotra 2000; and Ramazan *et al.* 2001].

SF like all other pozzolanic materials is capable of reacting with the calcium hydroxide, $Ca(OH)_2$ liberated during cement hydration to produce hydrated calcium silicate (C–S–H), which is accountable for the strength of hardened concrete. The high content of very fine amorphous spherical (100 nm average diameter) silicon dioxide particles (present more than 80%) is the main reason for high pozzolanic activity of SF [Bayasi and Zhou 1993]. There are a number of studies on the improvement of compressive strength of hardened concrete using SF available in published literature [Yogendran *et al.* 1987; Detwiler and Mehta 1989; Goldman and Bentur 1993; Hooton 1993; Khedr and Abou-Zeid 1994; Khatri et al. 1995; Sabir 1995; Zhou *et al.* 1995; Xie *et al.*1995; Iravani 1996; Neville 1996; Cetin and Carrasquillo 1998; Toutanji and Bayasi 1999; Mazloom *et al.* 2004 and Atis *et al.* 2005]. The SF can improve both chemical and physical properties, which transform the microstructure of concrete and hence reduce the permeability and increase strength [Elahi *et al.* 2010].

The durability and abrasion resistance of the SF concrete are also reported to be improved [Mehta 1985; Laplante *et al.* 1991; Malhotra and Mehta 1996; Müller 2004 and Behnood and Ziari 2008]. The resistance of concrete against acid and sulphate attack enhances with the addition of SF [Akoz *et al.* 1995; Turker *et al.* 1997; and Akoz *et al.* 1999]. It is well known that SF improves the bond between the paste and aggregate [Al-Khaja 1994; Khatri *et al.* 1997 and Alexander and Magee 1999]. Due to many advantages of SF it is being used as the most common mineral admixture for high-strength concrete (HSC) [Khayat and Aitcin 1992; and Poon *et al.* 2006].

However, all of the above studies are based on Portland cement. Development of PSC, where ground granulated blast furnace slag is incorporated in Portland cement to react with liberated $Ca(OH)_2$, made all these publications unusable for present construction. 95% of the cement used worldwide is PSC [www.indiancementreview.com]. Use of SF to enhance strength of concrete made of PSC has not got adequate research attention. This may be due to the uncertainty on the competence of SF to enhance strength of concrete in presence of slag. SF is an amorphous active form of silica, which is more active than slag [Didamony *et al.* 1996]. Also, higher specific surface area of SF compared to granulated slag [Cheng and Fildman 1985; Malhotra *et al.* 1987; and Sharara *et al.* 1994] makes SF chemically more active. Therefore, use of SF may significantly increase the mechanical properties even in PSC concrete.

FA is a by-product of coal-fired power plants, be suitable to pozzolanic materials. The demand of electric power is improved with the development of industry, and power stations now yield much FA annually. Maximum of them are dumped, while environmental situations will not permit the dumping of large amount of waste FA, which will growth every year.

FA contains of finely divided ashes produced by burning pulverized coal in power stations, and can be characterized as a normal type of pozzolana to yield high strength and high performance concrete. To achieve the sustainable development of concrete industry, high-volume fly ash (HVFA) concrete, which has normally 50–60% of FA as the total cementitious materials content, is broadly used. The combination of HVFA in concrete has many benefits, such as reducing the water demand, improving the workability, minimizing cracking due to thermal and drying shrinkage, and enhancing durability to reinforcement corrosion, sulfate attack, and alkali-silica expansion.

Dissimilar from cement, the foremost chemical components of FA are Al_2O_3, SiO_2 and Fe_2O_3. The mineral constituents of FA comprise a major vitreous phase and some minor crystalline phases (quartz, mullite, hematite and magnetite). Through the hydration of cement-FA composite binder, FA can react with $Ca(OH)_2$ and yield calcium silicate hydrate (C-S-H) gel and calcium aluminate hydrate (CAH) explicitly pozzolanic reaction which are active in creating denser matrix leading to higher strength and better durability. But the pozzolanic reaction of FA is relatively slow at early ages, so it mainly behaves as a micro aggregate to fill the pore structure of concrete, making a physical effect. At late ages, FA starts to make greater chemical effects and recover the properties of concrete. Furthermore, due to the exothermic hydration procedure of cement and low thermal conductivity of concrete, incorporation of FA in concrete obviously reduces the hydration heat to prevent concrete cracking. Concretes containing large amounts of FA were initially developed for mass concrete applications to reduce the heat of hydration. An efficient method to control the temperature rise of massive concrete structures is replacing a large part of cement by mineral admixtures such as FA whose hydration heat is much smaller than that of cement.

However, the addition of FA decreases the early strength of concrete. The chemical compositions, morphology, and the fineness of FA are the fore most reasons inducing the strength development rate. Due to the decrease in water requirement and the increase in reactivity of FA, the mortar or concrete strength significantly increases. The present study is an attempt to find out the optimum percentage of FA for obtaining the mechanical properties of FA concrete.

2.5 Studies on Variability of Concrete Properties

The construction field utilize the majority of such materials, by incorporating in concrete as supplementary cementing materials, and contribute to the sustainability. Such supplementary materials are FA, SF, metakaolin and ground granulated blast furnace slag [Radonjanin *et al.* 2013], used due to their pozzolanic activity. SF is very operative in design and development of concrete [Siddique 2011]. The incorporation of SF concrete in the construction sector is gaining popularity in the recent years, which requires the design and assessment of safety of these structures. Randomness and variability of material properties can considerably affect structural performance and safety. In contradiction to reality this phenomenon is usually neglected, in conventional structural analysis and design that assume deterministic values of material properties. This assumption makes the analysis models less realistic and less satisfactory. With the advancement of computing facilities, the complex structural analyses including the probabilistic nature of the various parameters of the structure are not difficult and have become essential for its response against natural loads like earthquake, wind, etc.

There are many studies [Campbell and Tobin 1967; Soroka 1968; Chmielewski and Konapka 1999; and Graybeal and Davis 2008] reported on the variability of compressive strength of concrete. The variability of compressive strength of concrete usually represented in literatures by a normal distribution if the coefficient of variation does not exceed 15-20%, although slight skewness may be present. However, when the coefficient of variation is high, the skewness is considerable [Campbell and Tobin 1967] and if the quality control is poor [Soroka 1968], a lognormal distribution is more rational to represent the tail areas of distribution than a normal distribution. A recent study [Chen *et* *al*. 2014] concludes that the variation in concrete compressive strength should be characterized using various statistical criteria and different distribution functions.

The inherent variability of cement and SF may not be similar in nature as SF is a by-product in the carbothermic reduction of high-purity quartz with carbonaceous materials like coal, coke, wood-chips in the production of silicon and ferrosilicon alloys. Therefore, existing literatures on the variability of cement concrete may not be useful to describe the variability of concrete with SF and FA.

There are a number of published literature on the risk assessment of structures made of traditional concrete and different methods to do so. Hwang and Jaw (1990) proposed a procedure to calculate fragility curves taking into account uncertainties in ground-motion and structure.

Singhal and Kiremidjian (1996) developed fragility curves for low, mid, and high rise RC frames that were designed using seismic provisions. Non-linear time history analyses were performed for stochastically generated frame models, with randomly paired simulated ground motion records. Structural demand versus seismic intensity relationships were determined from so-called stripe analyses. The structural demand at each seismic intensity level was assessed using ground motions scaled to that particular intensity level and was represented by a lognormal probability density function. The lognormal model of demand was then utilized to compute fragility estimates (for the performance limits considered) at that particular level. Finally, fragility curves were represented by lognormal cumulative distribution functions that were fit to individual fragility estimates, computed at several seismic intensity levels.

Singhal and Kiremidjian (1998) later presented a Bayesian method for updating the fragility curves which they had developed earlier for low-rise RC frames and estimating confidence bounds on those fragility curves, by using the observed building damage data from the 1994 Northridge earthquake.

Mosalam *et al.* (1997) studied on behaviour of low-rise Lightly Reinforced Concrete (LRC) frames with and without masonry infill walls using fragility curves. Adaptive nonlinear static pushover analyses were performed for the frame models. Monte Carlo simulation was used to generate the frame models considering uncertainties in material properties. Idealised single-degree-of-freedom (SDOF) systems developed from the pushover analysis results were employed in further analyses.

Shinozuka *et al.* (2000) developed empirical and analytical fragility curves for bridges. The observed bridge damage data from the 1998 Kobe earthquake was used for developing empirical fragility curves. Analytical fragility curves were developed from nonlinear time history analyses of stochastically generated models of two bridges, taking into account the uncertainty in material properties. Both fragility curves were represented by lognormal distribution functions with the distribution parameters estimated using the maximum likelihood method. Confidence intervals for the distribution parameters were also provided.

Porter *et al.* (2001) proposed an assembly-based vulnerability framework for assessing the seismic vulnerability of buildings. The proposed approach differs from usual fragility analysis discussed in literature. This approach accounts for the detailed structural and non-structural design of buildings. This is probabilistic analysis that considers the uncertainty associated with ground motion, structural response, assembly fragility, repair cost, repair duration and loss due to downtime. It is reported that the effectiveness of alternative retrofit scheme can be examined using this approach.

Ellingwood (2001) highlighted the importance of the probabilistic analysis of building response in understanding the perspective of building behaviour. This paper outlined a relatively simple procedure for evaluating earthquake risk based on seismic fragility curve and seismic hazard curve. This study shows the importance of inherent randomness and modelling uncertainty in forecasting building performance through a building fragility assessment of a steel frame.

Erberik and Elnashai (2004) studied the performance of mid-rise-flat-slab RC building with masonry infill walls using fragility curves as per the same methodology adopted by Singhal and Kiremidjian (1996). Uncertainties are considered by stochastically generated building models paired with each ground motion records rather than random sampling. Nonlinear static pushover analyses were carried out to identify performance limits for developing fragility curves.

Kim and Shinozuka (2004) developed fragility curves of two sample bridges before and after column retrofit for southern California region. Monte Carlo simulation was performed to study nonlinear dynamic responses of the bridges. Peak ground acceleration (PGA) was considered as intensity measure for developing fragility curves which is represented by lognormal distribution function with two parameters. It was found that the fragility curves after column retrofit with steel jacketing shows excellent improvement (less fragile) compared to those before retrofit.

Rossetto and Elnashai (2005) developed fragility curves for low-rise code designed RC frames with masonry infill walls for Italy region. Structural demand versus seismic intensity relationships was determined using the methodology given by Erberik and Elnashai (2004). Capacity spectrum method with adaptive pushover analysis was employed for estimating drift demand. A response surface equation was fit to the demand versus intensity data.

Ramamoorthy *et al.* (2006) developed fragility curves for low-rise RC frames. Cloud analysis was carried out based on nonlinear time history analyses to develop the structural demand. A bilinear function was used here to represent the median demand instead of a linear function given in Cornell *et al.* (2002).

Guneyisi and Altay (2008) developed fragility curves for high-rise RC office building retrofitted with fluid viscous dampers for Istanbul region. Three different scheme of viscous dampers (effective damping ratios as 10%, 15% and 20%) were used. For fragility analysis, a suit of 240 artificially generated ground motions compatible with the design spectrum was used to represent the variability in ground motions. Nonlinear dynamic responses of the structures before and after retrofit were studied. Slight, moderate, major, and collapse damage states were considered to express the condition of damage. The fragility curves, represented by lognormal distribution functions with two parameters, developed in terms of peak ground acceleration (PGA), spectral acceleration (S_a) and spectral displacement (S_d). Comparing the fragility curves this study concludes that viscous damper is an excellent retrofit scheme that improves the performance of buildings considerably.

Celik and Ellingwood (2010) studied the effects of uncertainties in material, structural properties and modelling parameters for gravity load designed RC frames. It was found that damping, concrete strength, and joint cracking have the greatest impact on the response statistics. However, the uncertainty in ground motion dominated the overall uncertainty in structural response. The study concluded that fragility curves developed using median (or mean) values of structural parameters may be sufficient for earthquake damage and loss estimation in moderate seismic regions.

Fragility based seismic vulnerability of buildings with consideration of soft storey and quality of construction was demonstrated by Rajeev and Tesfamariam (2012) on three-, five-, and nine-storey RC frames designed prior to 1970s.

A detailed literature review revealed no published literature that deals with risk assessment of concrete structures made of RCA, SF and FA.

2.6 Experimental Methods as per Indian Standards

All the experimental work conducted in this research confirming to Indian standard only. This section briefly describe the methods used for conducting experimental program.

2.6.1 Compressive Strength

The compressive strength of specimens is determined after 7 and 28 days of curing respectively with surface dried condition as per Indian Standard IS: 516-1959. Both moulds size $150 \times 150 \times 150 mm$ and $100 \times 100 \times 100 mm$ are used for evaluation of compressive strength. Three specimens are tested for typical category, and the mean compressive strength of three specimens is considered as the compressive strength of the specified category.

Figure 2.2: Compressive testing machine

2.6.2 Tensile Splitting Strength

Tensile splitting strength of concrete was found out as per IS: 516-1959. Cylinders of size $150 \times 300\ mm$ and $100 \times 200\ mm$ are used for getting tensile splitting strength of concrete throughout experiment.

2.6.3 Flexural Strength

Flexural strength of concrete was found out as per IS: 516-1959. Prisms of size $100 \times 100 \times 500\ mm$ was taken for the experiment.

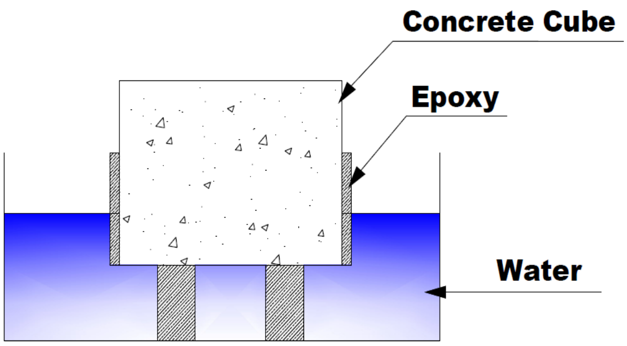
Figure 2.3: Flexural testing machine

2.6.4 Capillary Water Absorption

In the present study, capillary action through the concrete is found out by mass method using the concrete cubes of size in mm. After casting and successive 28 days curing, the cubes are allowed to dry in an oven at 105^{0}C until a gain of constant weight. One dimensional water flow is maintained for the measurement of capillary action by coating the cube with epoxy resins, except the top and bottom surfaces. The cubes are immersed in the water, and a minimum depth of immersion of 5 mm above the base of the cube is maintained. A gap of approximately 2 mm is maintained between the immersed face and the bottom of the water for good contact with water. The duration of immersions are 0.5, 1, 2, 4, 6, 24, 48, 72, and 96 hours respectively. The capillary water absorption is measured by recording the respective weights of the cubes after successive immersion. Capillary action is calculated using the following relation as a function of time.

$$\Delta W = S \times \sqrt{t} \tag{2.1}$$

where, ΔW is the cumulative amount of water absorbed per unit area (gm/mm^2) during the time of immersion (t) and *S* is the coefficient of capillary water absorption



(a) Schematic view of Capillary water absorption

(b) Mortar Specimens (c) Concrete Specimens

Figure 2.4: Capillary water absorption test set up

2.6.5 Drying Shrinkage

Drying shrinkage test is used to measure the shrinkage of concrete by determining the change in length of concrete specimens due to changes in moisture content. Initial drying shrinkage for the RCA concrete is measured as per Indian Standard IS: 1199-1959. The concrete prism is de-moulded after 24 hrs and left in the moist air for seven days. At the end of moist curing, the specimens are put in a water tank at 27^0C for 20 days. After the completion of curing, the length of the specimen is measured (to an accuracy of 0.005mm) in wet condition. This length is termed as the original wet measurement. Then the specimens are kept in the oven at 50^0C for a period of 44 hrs. After this period of heating, it is allowed to cool for at least four hours. The reading which is taken after cooling is taken as dry measurements. Dry shrinkage is measured as the difference between the original wet measurement and final dry measurement.

Figure 2.5: Drying shrinkage test

2.6.6 Air Content

Pressure method is used for measuring air content in freshly mixed concrete as per Indian Standard IS: 1199-1959. Freshly mixed concrete is kept inside the bowl after successive tamping. Required test pressure which is slightly more than 0.02 kg/cm^2 is applied through hand pump after adding water. At this instant, the corresponding initial height of water is measured on the graduated precision bore tube or gauge glass of the standpipe. Then test pressure is released gradually, and final water height is measured. The air content 'A' is calculated as

$$A = A_1 - G \tag{2.2}$$

where A is the apparent air content in percentage by volume of concrete, and it is equal to the difference between initial water height and final water height. 'G' represents the aggregate correction factor, in percentage by volume of concrete which is obtained as per IS: 1199-1959.

Figure 2.6: Air content machine

2.6.7 SEM and EDX

In order to understand the effect of bacteria in the microstructure of mortar various techniques such as spectroscopy, X-ray powder diffraction (XRD), Field Emission Scanning Electron Microscope (FESEM) and Scanning Electron Microscope (SEM) are used in the present study.

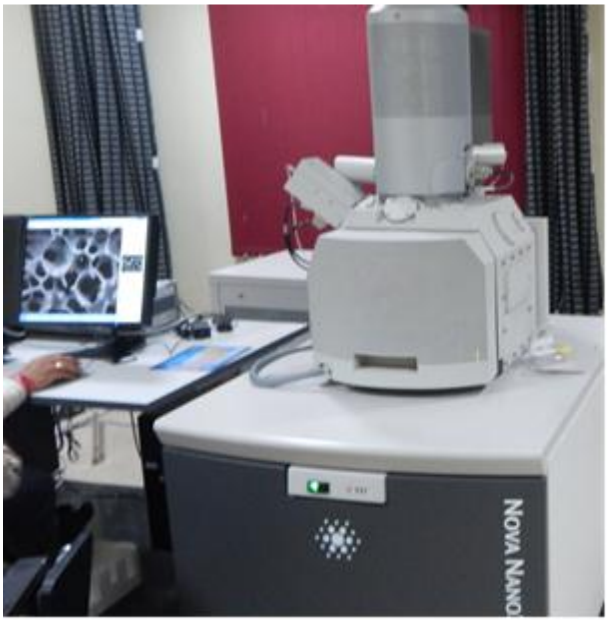


Figure 2.7: FESEM (Nova Nano SEM/FEI)

The deposition of calcium carbonate precipitation is investigated by scanning electron microscope (SEM) and X-ray diffraction (XRD). SEM micrographs are obtained using Jeol JSM-6480 LV SEM apparatus. Samples taken from the inner core of the concrete cube are dried at room temperature, and coated with platinum (JFC-1200 fine coat). During the examination, the accelerating voltage is maintained at a range of 20 kV. Mineral components of the isolates are further characterised by Energy Dispersive X-ray Spectroscopy (EDX) analysis.

2.6.8 XRD Spectroscopy

Ultimate X-ray diffractometer with a Cu anode (40 kV and 30 mA) and scanning from 200 to 800 is used for XRD spectra. The samples are taken from the inner core area, well crushed and ground before mounting on a glass fibre filter using a tubular aerosol suspension chamber (TASC). The components of the sample are identified by comparing them with standards established by the international centre for diffraction data.

Figure 2.8: Multipurpose X-ray diffraction system (Rigaku ULTIMA IV)

2.7 Summary

This chapter presents the review of literature in six segments comprising studies in RCA concrete, studies on application of bacteria to improve concrete, studies on mechanical properties of SF and FA concrete, studies of variability of normal concrete, studies on fragility curves and review of experimental methods used in the present study

Following are the important observations drawn out of the literature review.

(a) The relationship between the *w/c* ratio and compressive strength is essential for the preliminary estimation of compressive strength. Indian standard recommends such relationship for natural aggregate concrete. This relationship may be different for RCA concrete depending on the aspects such as age of recycled aggregate and number of recycling. Many studies are reported in literature that focus on the behaviour, properties, and functional uses of RCA. However, no study reported on the behaviour of RCA concrete with regard to above aspects. The present work is an attempt to study the relationship of *w/c* ratio with compressive strength considering age and successive recycling of RCA. Further, the properties like capillary water absorption, drying shrinkage, air content, flexural strength and tensile splitting strength of RCA concrete are also investigated considering the age and successive recycling of RCA.

(b) It is found from an extensive review of literature that there is hardly any research on the use of bacteria to improve the properties of RCA concrete. Therefore, the present study focus on the improvement of RCA concrete (made by 100% replacement of natural coarse aggregates) by adding two bacteria, *Bacillus subtilis* and *Bacillus sphaericus* bacteria, which are widely available and most efficient in calcite production in an alkaline environment.

(c) An extensive literature review revealed that majority of the published literatures, if not all, present studies on Portland cement. However, production of Portland cement in the recent times is reduced considerably and the construction industry uses slag cement mostly. This is the motivation for a detailed study on the concrete made with slag cement partially replaced by SF or FA. Most of the previous studies on SF concrete considered the partial replacement of cement keeping the total weight of cementitious material, fine and coarse aggregate as constant. The main purpose of these studies was to evaluate the effect of SF on the behaviour of concrete. However, in practical constructions, SF concrete is prepared as per relevant codes and standards. Many international design codes recommend extra cement of 10% while mineral admixture is used as partial replacement of cement. The behaviour of SF concrete prepared with the above practical aspects is not investigated yet. Also, almost all the previous studies on SF concrete are concentrated in high-strength concrete. No attempt has been carried out using SF as a replacement of cement for low/medium grade concretes (viz. M_{20}, M_{25}). The present experimental investigation is also focussed on the behaviour of medium grade concrete designed as per Indian Standard IS: 10262:2009 using slag cement partially replaced by SF.

(d) Performance-based analysis requires probability distributions of the constituent materials in the structure. Although the variability of mechanical properties related to normal concrete are reported in literature, the same for concrete made of partial replacement of mineral supplements such as SF and FA etc. are not available in literature. Extensive literature review revealed that studies on the variability of mechanical properties of concrete made with partial replacement of SF or FA is not available.

In the present study different two-parameter probability distribution functions are considered for the description of three important mechanical properties of SF and FA concrete: compressive strength, flexural strength and tensile splitting strength. Best fitted probability distribution functions are derived conducting various statistical goodness-of-fit tests.

Chapter 3

Concrete using Recycled Coarse Aggregate

3.1 Introduction

Recycling of old demolished concrete into aggregate is relatively simple process which involves breaking, removing, and crushing of existing concrete into a material with a specified size and quality. Previous research has proved that such RCA could successfully be used as substitute of natural coarse aggregates to produce normal and high strength concrete. The use of such recycled materials offers multiple environmental advantages by offering potential diversion of useful materials from the waste streams, reducing the energy investment in processing virgin materials, conserving natural resources, and allaying pollution.

RCA have lower specific gravity and higher water absorption capacity compared to natural aggregates due to the adhered mortar with RCA. The properties of concrete made with RCA are strongly dependent on the quality of the primary concrete crushed. The first part of this chapter deals with number of recycling and the age of the RCA, and its effects on the mechanical properties of RCA concrete.

Various researchers have reported that the physical and mechanical properties of RCA concrete are lower than that of conventional concrete with NCA. Second part of this chapter presents the experimental results of enhancement of mechanical properties of RCA concrete using bacteria.

In order to understand the effect of bacterial mineral precipitation on the properties of RCA concrete better, further studies were carried out on cement mortar only. The last part of this chapter presents the results of the properties of cement mortar with bacteria.

3.2 Behaviour of RCA Concrete

Behaviour of the RCA concrete is studied experimentally with a special emphasis on the age and number of recycling of RCA. This following sections present the details of materials used and the results of experimental study in this regard.

3.2.1 Materials and Mixture Proportion

RCA were collected from two sources: (a) demolished concrete wall (unused) constructed for drainage purpose (three years old) and (b) crushed concrete cubes and beams from structural engineering laboratory (aged about zero to one year old). While the Source (a) can be considered as non-load-bearing type with a design characteristic strength of 20 MPa the Source (b) was a mixture of concrete with different design characteristic strength (ranging from 25 MPa to 30 MPa) and all of them were undergone a loading (direct compression or combined shear-bending) up to failure. The exposure condition for both of the sources may be considered as normal. The RCA samples are collected and grouped in two categories according to the source/age of the parent concrete. A total of 24 mixtures are considered for the investigation. The materials used, mixture proportions are explained in the following sections. The relevant experimental procedure and test setup are discussed in Chapter 2.

Portland slag cement conforming to IS: 455-1989 code is used as the binder. It is to be noted here that the majority of the previous literature have used OPC in the preparation of RCA concrete. However, there are few papers (Myung-Kue, 2005; Sagoe *et al*. 2001; Hansen, 1990) that reports the results of RCA concrete made of PSC. 95% of the total cement used worldwide is PSC (Indian Cement Review, 2015; Saunders, 2015). Therefore, the motivation of this study was to check the behaviour of RCA made of PSC which may be more useful for the present construction. The fine aggregate (sand) are collected from the local river, conforming to IS: 383-1970. Two different types of demolished parent concrete are identified which are reported to have a characteristic compressive strength of 20MPa.The parent concrete is crushed using mini jaw crusher. The opening of the jaw crusher was maintained at 20mm for producing maximum size of 20mm coarse aggregate as per requirement of IS: 456-2000. As the two parent concrete samples are having different ages, they are divided into two groups named as RC-1 (0-1 year) and RC-2 (3 years). The concrete produced from RC-1 aggregate again demolished and crushed to examine the effect of number of recycling. Aggregate obtained from RC-1 concrete named as N2-RC-1 which indicates that the aggregate is recycled for two times. The age difference between the two recycles (RC-1 and N2-RC-1) was less than three months. Use of the first structure is explained earlier. After the first recycling, these aggregates (RC-1) were used in the laboratory specimens which has undergone failure load before the 2nd recycling (N2-RC-1). The properties of cement, sand and RCA are summarized in the Tables 3.1 - 3.4 respectively. These properties are found to be conforming to IS: 2386 (Part III)-1963. The values of specific gravity, water absorption, impact and crushing strength of RC-1 and RC-2 show that RC-1 is better than RC-2. This is perhaps due to more amount of mortar present on the surface of the RC-2 which can be seen in Fig. 3.1. Also another reason for this can be attributed to the lower specific gravity of the parent coarse aggregate. Same behaviour is obtained in case of N2-RC-1 which shows inferior physical properties in comparison to RC-1. Fig 3.2 shows the particle size distribution of all RCA used in the present study.

Table 3.1: Properties of RCA of different age

Type of aggregate	Age / No. recycling	Specific gravity	Bulk density (kg/l)	Loose bulk density (kg/l)	Water absorption (%)	Impact value (%)	Crushing value (%)	Fineness modulus
RC-1	0-1 year	2.48	1.409	1.24	4.469	26.910	26.514	3.38
RC-2	3 years	2.26	1.312	1.19	5.360	28.194	26.817	2.45
N2-RC-1	2	2.38	1.174	1.03	5.403	31.703	28.449	3.12
NCA	0	2.83	1.97	1.73	1.1	23.84	23.16	2.84

Table 3.2: Properties of cement and fine aggregate

Type of material	Specific gravity	Water absorption (%)
Portland slag cement (Konark)	3.015	_
Sand	2.658	0.0651

Table 3.3: Chemical composition of Portland slag cement

Chemical Components	Percentage (%)
SiO_2	12
CaO	43
MgO	6.7
Fe_2O_3	12
Al_2O_3	26

Table 3.4: Physical properties of Portland slag cement

Properties	Value
Specific gravity	3.015
Fineness by sieve analysis	2%
Normal consistency	32%


(a) RC-1


(b) RC-2


(c) N2-RC-1

Figure 3.1: RCA used in the present study

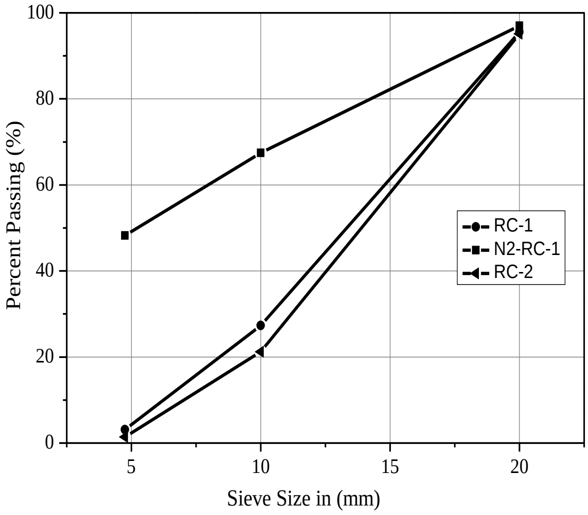


Figure 3.2: Particle size distribution of RCA

The concrete mixes for compressive strength tests were prepared with different *w/c* ratios according to IS: 10262-1982. Water content was kept constant as 186 kg/m^3 for all the mix, and as the *w/c* ratio varies, the cement content varies from 620 kg/m^3 to 286 kg/m^3 respectively. For air permeability, shrinkage and capillarity action tests, the respective samples were casted with constant cement content, *w/c* ratio, and water content as 372 kg/m^3, 0.5 and 186 kg/m^3 respectively. The quantities of mix proportions considered are presented in Tables 3.5 - 3.8.

Earlier version of Indian standard IS: 10262 (1982) recommends the relationship between the w/c ratio and compressive strength for Natural Aggregate Concrete (NAC). The present work is an attempt to study the relationship of w/c ratio with compressive strength for RCA concrete in the same line. Table 3.5 presents various mixture proportions for RCA concrete with varying w/c ratio (keeping the total amount of water constant) used for this purpose.

Table 3.5: Mixture proportion for RC-1 concrete

Mixture name	Mix 1	Mix 2	Mix 3	Mix 4	Mix 5	Mix 6	Mix 7	Mix 8
Cement (kg/m^3)	620	531	465	413	372	338	310	286
Sand (kg/m^3)	429	469	500	531	560	587	614	639
RCA (kg/m^3)	1007	1072	1120	1132	1140	1155	1171	1185
w/c	0.30	0.35	0.40	0.45	0.50	0.55	0.60	0.65
Water (kg/m^3)	186	186	186	186	186	186	186	186

Table 3.6: Mixture proportion for RC-2 concrete

Mixture name	Mix 1	Mix 2	Mix 3	Mix 4	Mix 5	Mix 6	Mix 7	Mix 8
Cement (kg/m^3)	620	531	465	413	372	338	310	286
Sand (kg/m^3)	428	469	500	531	560	587	614	639
RCA (kg/m^3)	964	999	1020	1033	1039	1041	1039	1035
w/c	0.30	0.35	0.40	0.45	0.50	0.55	0.60	0.65
Water (kg/m^3)	186	186	186	186	186	186	186	186

Table 3.7: Mixture proportion for N2-RC-1 concrete

Mixture name	Mix 1	Mix 2	Mix 3	Mix 4	Mix 5	Mix 6	Mix 7	Mix 8
Cement (kg/m^3)	620	531	465	413	372	338	310	286
Sand (kg/m^3)	429	469	500	531	560	587	614	639
RCA (kg/m^3)	1016	1052	1075	1087	1094	1096	1112	1120
w/c	0.30	0.35	0.40	0.45	0.50	0.55	0.60	0.65
Water (kg/m^3)	186	186	186	186	186	186	186	186

Table 3.8: Mixture proportion for shrinkage, capillary absorption, air permeability, splitting tensile strength and flexural strength Test

Mixture Name	Cement (kg/m^3)	NCA (kg/m^3)	RCA (kg/m^3)	FA (kg/m^3)	*w/c*	Water (kg/m^3)
RC-1	338	-	1142	587	0.55	186
RC-2	338	-	1041	587	0.55	186
N2-RC-1	338	-	1096	587	0.55	186
NCA	338	1253	-	587	0.55	186

The rotary concrete mixture is used for mixing all concrete in the laboratory. Cube moulds of sizes, $150 \times 150 \times 150\ mm$, prism moulds of size $75 \times 75 \times 150\ mm$, cylinder moulds of 150 mm dia. and 300 mm length and prism moulds of size $100 \times 100 \times 500\ mm$ are used for compressive strength, drying shrinkage, tensile splitting strength and flexural strength tests respectively according to IS:516-1999 and IS:1199-1959. For measuring air content, the concrete is casted in a particular mould recommended by IS: 1199-1959. After casting, the specimens are de-moulded after one day and cured in the water tank maintaining at 27^0 C until 28 days.

3.2.2 Influence of Age of RCA on Compressive Strength of Concrete

The compressive strengths for all samples are tested after 28 days of curing in a compression testing machine. Fig. 3.3 presents the compressive strength values obtained for all the samples as function of their *w/c* ratios. Associated trend lines represent the correlation between compressive strength of RCA concrete and its *w/c* ratio. This relationship is compared with the relationship recommended by IS: 10262-1982 for NCA concrete. This figure shows that the behaviour of RCA concrete with regard to the above relation is different from that of NCA concrete. The strength of NCA concrete is higher than RCA concrete at lower *w/c* ratios. This trend reverses for relatively higher *w/c* ratios. For RC-1 concrete, this reversal takes place at *w/c* of 0.37, whereas it occurs at *w/c* of 0.42 for RC-2. The reason behind this can be attributed to the higher water demand of RCA. The RCA requires a threshold quantity of water depending upon the parent adhered mortar to contribute to the strength. This minimum quantity is observed experimentally as 0.37 and 0.42 for RC-1 and RC-2 respectively. If the water is less than this minimum quantity, the RCA concrete may fail to yield the desired compressive strength. These results suggest that, in order to obtain higher compressive strength for RCA, *w/c* ratio should be higher than the above mentioned minimum limits.

RC-1 concrete is always found to be stronger than RC-2 (refer Fig. 3.3) and the difference between their strengths are almost same at each *w/c* ratios. The difference in strength may be due to the higher specific gravity of RC-1 than RC-2 (refer Table 3.1). Specific gravity of RCA depends on many parameters such as age, quality and quantity of adhered mortar and properties of parent aggregate. Also, it is to be noted that the decrease in compressive strength of RCA concrete is due to the increase in age of about 1-2 years is approximately 6%.

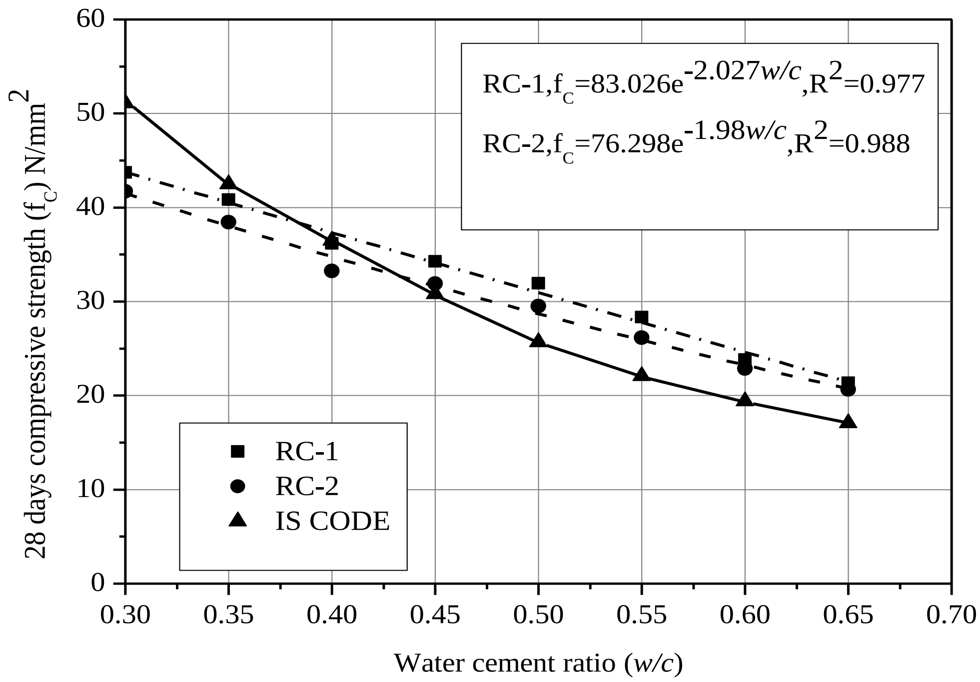


Figure 3.3: Correlations between *w/c* ratio and compressive strength

3.2.3 Number of Recycling of RCA on Compressive Strength

Compressive strength of N2-RC-1 concrete is recorded at various *w/c* ratios and the variation of the same is presented in Fig. 3.4. The compressive strength of N2-RC-1 concrete is less than that of RC-1 and the difference between their strengths are almost same at each *w/c* ratios. The decrease in strength of N2-RC-1 is about 2% compared to that of RC-1. The specific gravity of N2-RC-1 is observed to be lower than that of RC-1 as reported in Table 3.1 which may be the probable reason for strength decrement. The decrease in the specific gravity is perhaps due to the recycling process. Similar to the previous results, the N2-RC-1 concrete display more compressive strength than NCA concrete for the *w/c* ratios more than 0.42. This *w/c* ratio can be regarded as the minimum water required for the N2-RC-1 concrete to achieve the desired strength.

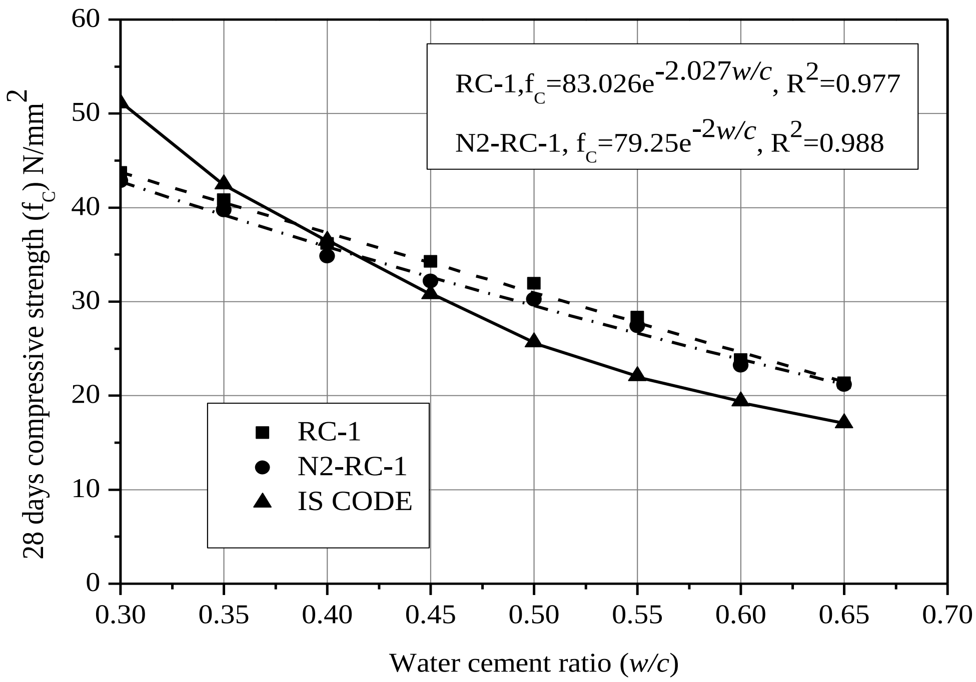


Figure 3.4: Correlation between *w/c* ratio and compressive strength of N2-RC-1 concrete

3.2.4 Capillary Water Absorption

The capillary water absorption, which is the cumulative amount of water per unit area (g/mm^2), is plotted in terms of square root of time in hours. Fig. 3.5 shows the capillary water absorption curves for RC-1, RC-2 and NCA concrete. The capillary water absorption for RC-2 is substantially higher (about 76%) than that of NCA concrete. The reason for this can be attributed to the lesser specific gravity (refer Table 3.1) resulting from the more adhered mortar on the surface of RC-2 than RC-1. More aged RC-2 sample may be more porous than RC-1 and thereby absorbs more water than RC-1. RC-1 concrete display about 11% more water absorption than NCA concrete, this makes it inferior in quality due to the obvious factor of recycling.

Fig. 3.6 shows the capillary water absorption curves for N2-RC-1, RC-1 and NCA concrete. It can be seen that the capillary water absorption of N2-RC-1 is significantly larger than both RC-1 and NCA by about 9 times. This abrupt increase of water absorption behaviour shows that successive recycling may produce very inferior quality of aggregates for use of concrete.

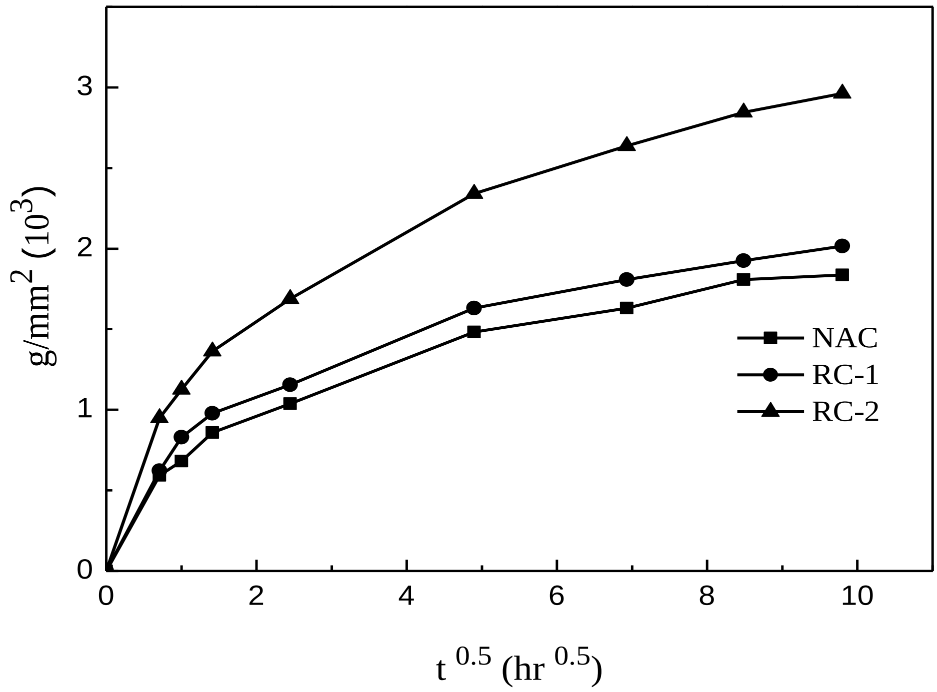


Figure 3.5: Variation of capillary water absorption for NCA, RC-1 and RC-2

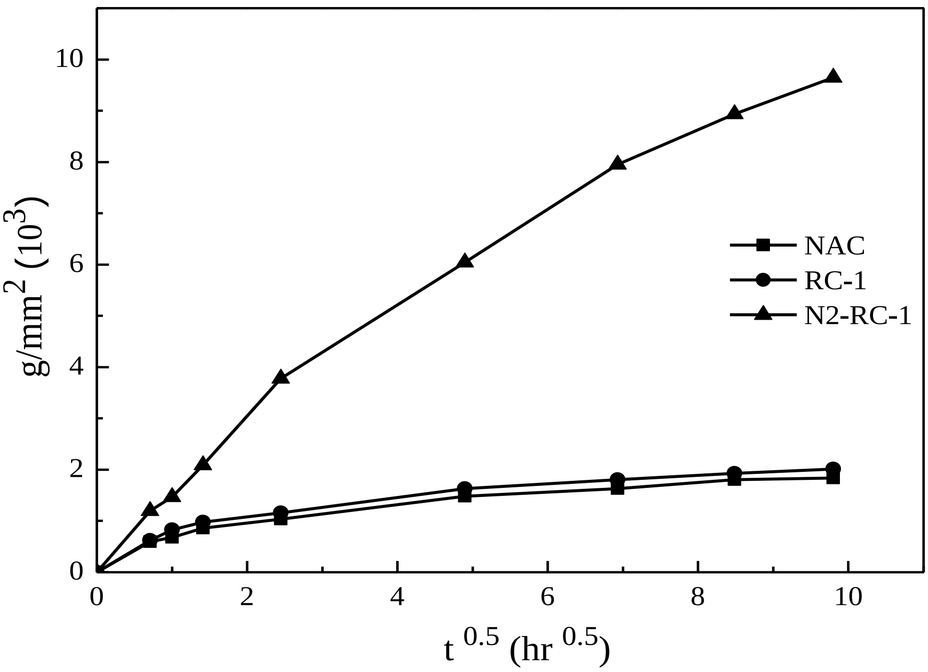


Figure 3.6: Variation of capillary water absorption for NCA, RC-1 and N2-RC-1

3.2.5 Drying Shrinkage

Drying shrinkage is defined as the contracting of a hardened concrete mixture due to the loss of capillary water. This shrinkage causes an increase in tensile stress, which may lead to cracking, internal warping, and external deflection, before the concrete is subjected to any kind of loading. Drying shrinkage of all the selected RCA concrete samples as well as NCA concrete sample has been tested as per the procedure outlined in Chapter 2 and the results are presented in Table 3.9. The drying shrinkage strain of RC-1 and RC-2 are found to be about 1.9 and 2.6 times more than that of NCA concrete respectively. The probable reason for this may be the more amounts of old mortar attached on the surface of RC-2 than RC-1. The shrinkage strain of N2-RC-1 is about 1.2 times more than RC-1 indicating fact that successive recycling increases the drying shrinkage strain.

Table 3.9: Drying shrinkage

Type of concrete	Drying length (mm)	Drying shrinkage (%)
RC-1	0.261	0.17
RC-2	0.341	0.23
N2-RC-1	0.312	0.21
NCA	0.135	0.09

Table 3.10: Air content of RC-1, RC-2, N2-RC1 and NCA samples

Type of concrete	Air content (%)
RC-1	12
RC-2	13
N2-RC-1	13
NAC	12

3.2.6 Air Content

Air entrainment is a necessary component of concrete mixtures exposed to freezing and thawing environments. The measurement of air content in fresh concrete of normal density is typically performed using the pressure method as discussed in Chapter 2. Table 3.10 shows the air content of all the selected concrete samples. It can be seen that the air content of RC-2 concrete at the fresh condition is slightly higher than that of RC-1 concrete. The more air content in the RC-2 concrete may be due to more amount of adhered old mortar on the surface of RC-2 aggregates. The N2-RC-1 concrete also shows more air content than RC-1 due to the same reasons.

3.2.7 Splitting Tensile and Flexural Strength

The splitting and flexural tensile strength of RC-1, RC-2 and N2-RC-1 at both 7 and 28 days are tabulated in Table 3.11. Splitting tensile strengths of RC-2 concrete at 7 and 28 days are found to be about 28% and 14% lower than RC-1 concrete respectively. The flexural strength of RC-2 concrete at 7 and 28 days are about 6% and 21% lower than RC-1 concrete.

The N2-RC-1 concrete shows lesser strength than that of RC-1 due to the re-recycling. While the splitting tensile strength of N2-RC-1 concrete is about 6% lower, the flexural strength is about 12% lower than the corresponding strengths of RC-1 at 28 days. The reason for the lower strength values of N2-RC-1 concrete may be due to its lower values of specific gravity compared to RC-1 aggregates.

Table 3.11: Splitting tensile and flexural strength of RCA concrete

Specimen name	Splitting tensile strength (MPa)		Flexural strength (MPa)	
	7 days	28 days	7 days	28 days
RC-1	2.636	2.961	4.937	6.605
RC-2	1.896	2.544	4.643	5.173
N2-RC-1	2.451	2.775	5.591	5.754
NCA	2.853	3.064	6.870	8.171

3.2.8 Cost Benefit Study for RCA Concrete

The cost benefit analysis has been carried out using typical scheduled rate (at NIT Rourkela) as presented in Table 3.12. The cost of materials considered in the study are follows: INR 7.30 for Cement, INR 0.15 for sand and INR 0.95 for NCA per kg. INR 800 is considered for the labour cost per cubic meter of concrete.

Table 3.12: Comparative cost estimate for 1 m^3 of concrete

Concrete type	Cement (kg)	Sand (kg)	NCA (kg)	RCA (kg)	Cost (INR)	Compressive Strength (MPa)
RCA concrete (RC-1)	338	587	0	1142	3350	18.4
Control concrete	338	587	1253	0	4530	22.6

3.3 RCA Concrete using Ureolytic Bacteria

Previous literature have reported that urease producing bacteria can improve the quality of concrete by mineral precipitation. In this study two urease producing bacteria namely, *Bacillus subtilis* and *Bacillus sphaericus* are used for enhancing the different engineering properties of RCA concrete. First part of this section presents the culture of bacteria, materials and mixture proportion, experimental procedure and results associated with RCA concrete incorporating bacteria.

In order to understand the effect of bacteria on the concrete more precisely, further studies were carried out on cement mortar. Second part of this section presents the results of setting time, compressive strength, sorptivity, drying shrinkage and micro-structural studies of cement mortar in the presence of bacteria.

3.3.1 Culture of Bacteria

B. subtilis (MTCC.736) and *B. sphaericus* (MTCC.7542) which facilitate the precipitation of calcium carbonate are collected from Institute of Microbial Technology (IMTECH), Chandigarh, India, and constantly maintained on nutrient agar slant.

In order to confirm that these bacteria are capable of producing calcium carbonate, a Calcite Precipitation Agar (CPA) test has been undertaken. CPA is a solid medium for screening of bacterial precipitation of calcium carbonate. 0.6g of nutrient broth, 5.7g of $CaCl_2$, 0.424g of $NaHCO_3$, 2.0g of NH_4Cl, 3.0g of Agar, 190ml of distilled water are taken in a 200ml conical flask. All media components are autoclaved. After autoclaving, urea is added to the medium. 20μl of broth culture is inoculated in the centre of a plate, and then incubated at 30°C for 6 days. After 6 days calcium carbonate is found in the plate as white patches and thus it is inferred that both *B. subtilis* and *B. sphaericus* are capable to produce calcium carbonate in the conditions explained above.

Having selected bacteria and confirmed their capability to precipitate the Calcium carbonate, the following procedure is used for cell culture for further studies.

i. 500*ml* of nutrient broth for *B. subtilis* and Luria Bertani for *B. sphaericus*, broth is prepared in two fresh clean conical flask of 1*l*.
ii. After autoclaving the nutrient medium, bacteria is inoculated into it and incubated for 24hours at 37^0C with constant shaking at 150 rpm.

The above bacteria culture is tested for cell concentrations using optical density test, which is based on light scattering principle. The cell concentration at 600 nm is found out as per McFarland's standards. The cell culture are diluted to the required concentrations and added to the water used for making cement concrete. The medium composition of nutrient broth used for routine culture is shown in Table 3.12.

Table 3.13: Medium composition for nutrient broth

Composition	Amount (g/l)
Peptone	5
NaCl	5
Yeast extract	3
Urea	20
Calcium chloride	15
NH_4Cl	20

Growth kinetic study of bacteria is done using UV-V spectrophotometer (Lambda 35, Singapore). One loop of bacterial culture from preserved slant is inoculated into the nutrient broth. The turbidity of the culture medium is measured by observing the optical density (O. D.) every one hour, and a graph is plotted to obtain the growth curve of the test organism. Fig. 3.7 shows a typical growth curve of the *B. subtilis* marked by different phases such as lag phase, log phase and stationary phase. The bacteria concentrations required for addition in the concrete are selected considering the growth kinetics of the bacteria. The lag phase of the growth kinetics corresponds to low concentration of bacterial cells (~10^1 cells/ml), whereas as, the mid-log phase of growth kinetics corresponds to the concentration of ~10^3 cells/ml. Generally, the highest number of live active cells is found in the late log phase or early stationary phase of growth kinetics which corresponds to about 10^6 cells/ml. At a concentration of about 10^7 cells/ml the number of live cells is found to be less, and the activity decreases. The present study uses the four cell concentrations, 10^1, 10^3, 10^6, and 10^7 cells/ml considering the growth kinetics of bacteria. The concentration of the bacteria was obtained by growing the culture for different time followed by centrifugation at 10,000 rpm for 10 min at 40^0C.

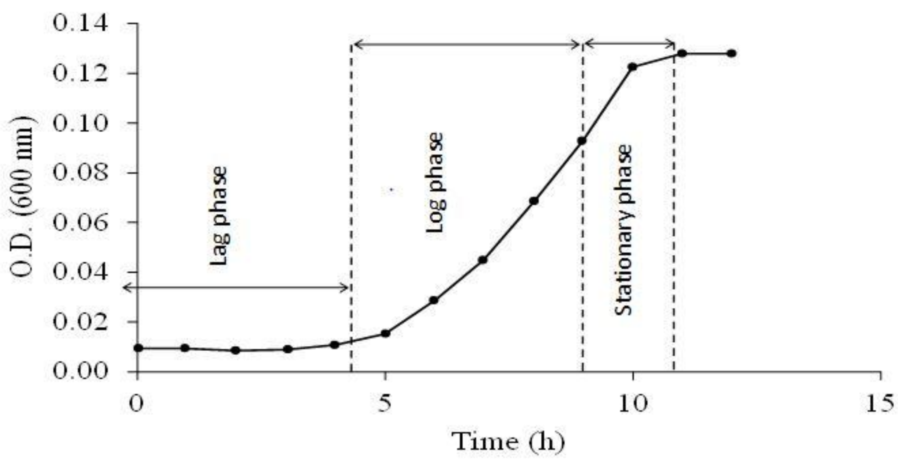


Figure 3.7: Typical growth curve of bacteria (*B. subtilis*)

The behaviour of bacterium is important as it must survive in warm and alkaline type environment of concrete. Fresh concrete has a pH of 11 to 13 and have a rise in temperature up to 55^0C due to heat of hydration. Therefore, the bacterium selected should survive in the alkaline and warm concrete like environment and capable of producing calcium carbonate. pH and temperature tolerance tests were conducted to confirm the suitability of *B. sphaericus* and *B. subtilis*. It was found that both *B. sphaericus* and *B.*

subtilis could survive in the pH range of 8 to 13 at both 37ºC and 55ºC from tolerance tests.

3.3.2 Materials and Mixture Proportion

Portland slag cement (PSC) made from waste blast furnace slag is popular in the regions adjacent to steel industries. The present study uses PSC cement conforming to IS: 455-1989 for making concrete. The chemical composition and the physical properties of the cement used are given in Tables 3.2-3.4 respectively.

Locally available sand (fine aggregate) conforming to IS: 383-1970 is collected from a nearby river and used in the present study. This study considers only one type of RCA (RC-1 as mentioned in the previous section) for the preparation of concrete mix. The particle size distribution curve of RCA is presented in Fig. 3.8. The physical properties of both fine aggregate, RCA and NCA are evaluated as per IS: 2386 (Part III)-1963 and reported in Table 3.1.

In order to study the improvement of compressive strength of RCA concrete, various concrete mixes are considered with and without bacteria. The mix design is carried as per the normal concrete design procedure available in IS: 10262-1982 following a weight batching. The RCA concrete mixes are prepared by full (100%) replacement of NCA with RCA. The cement content is kept constant at 372 kg/m^3 with a constant total *w*/*c* ratio of 0.5. Concrete with RCA and NCA prepared without incorporating bacteria is also considered in this study as reference. Four different bacterial concrete with cell concentrations of 10^1, 10^3, 10^6, 10^7 cells/ml were prepared and these mixes are represented as B-1, B-2, B-3 and B-4 respectively. When *B. subtilis* is used these mixes are termed as B-1a, B-2a, B-3a and B-4a whereas, for *B. sphaericus*, these mixes are named as B-1b, B-2b, B-3b and B-4b. All together ten number of mixes of concrete are considered: four bacterial concrete for each of the two bacteria considered and two control concrete mixes with RCA and NCA. The details of all the mixtures are presented in Table 3.13. It is to be noted that a few previous literature (Chahal *et al. 2012*; Tittelboom *et al.* 2010 and De-Muynck *et al.* 2008) have reported the mix-proportion with *w*/*c* ratio as 0.5 for bacterial concrete. Also, the study presented in Section 3.3.1 of this thesis shows that *w*/*c* ratio higher than 0.37 results in relatively better compressive strength for RC-1 concrete. Accordingly, a *w*/*c* ratio of 0.5 was chosen for this study.

RCA was soaked in water for one hour and then kept out of the water for 24 hours before casting to avoid the extra water absorption by RCA and to have the same saturated surface dry condition of RCA as that of NCA. Under laboratory conditions, both coarse and fine aggregates were dry blended with cement for 2 minutes before adding the water. Around 10% of total water was taken out and used for mixing the bacteria solution. The remaining water (90%) was added and mixed with dry aggregates and cement for one minute. The diluted bacterial solution was finally mixed with concrete for another 3 minutes.

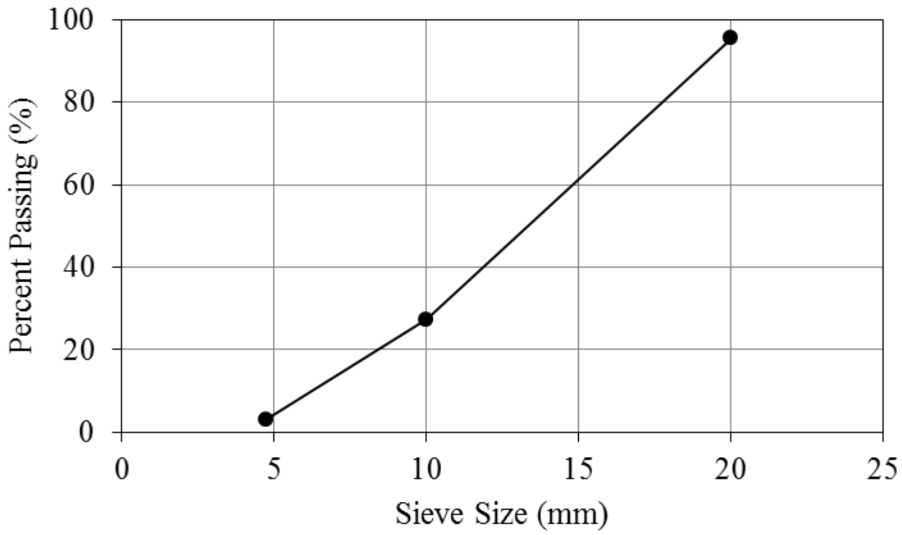


Figure 3.8: Particle size distribution of RC-1

Table 3.14: Concrete mix proportion

Mixture Name	NCA Concrete	Bacterial RCA Concrete				
		Control	B-1	B-2	B-3	B-4
Bacterial Concentrations (cells/ml)	0	0	10^1	10^3	10^6	10^7
Cement (kg/m^3)	372	372	372	372	372	372
Natural Sand (kg/m^3)	578	563	563	563	563	563
RCA (kg/m^3)	1233	1137	1137	1137	1137	1137
w/c ratio	0.5	0.5	0.5	0.5	0.5	0.5
Water (kg/m^3)	186	186	186	186	186	186

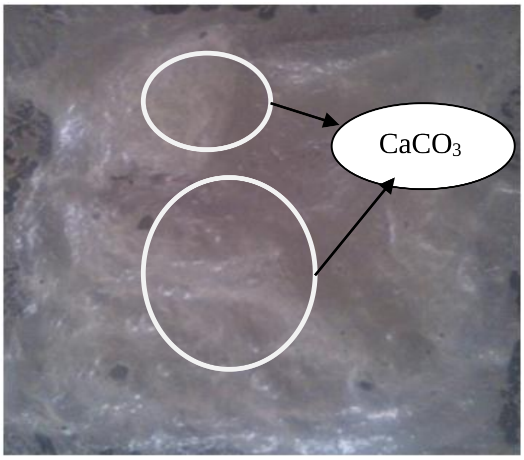
3.3.3 Experimental Results

Compressive strength, drying shrinkage and capillary water absorption test of hardened concrete and air content test of fresh concrete were carried out for all the concrete samples listed in previous section. Three samples were tested for each category and the average of the three was considered as the final result. Following sections discuss the results obtained from these test.

3.3.3.1 Compressive Strength

The addition of bacteria to the fresh concrete results in the formation of $CaCO_3$ precipitation that can be observed through the naked eye as shown in Fig. 3.9. Fig. 3.9 presents photographs of typical RCA concrete with *B. Subtilis* and *B. sphaericus* (10^6 cells/ml), RCA control and NCA concrete without bacteria. White foam like material can be visualised in the outer surface of the bacterial concrete sample (Figs. 3.9a and 3.9b) which are absent in other two (Figs. 3.9c and 3.9d).

The mean compressive strength for specimens with different concentration of bacteria at 7 days and 28 days are presented in Table 3.14. It is observed that the compressive strength of bacterial concrete increases with the increase of cell concentration for both 7 days and 28 days strength. However, after cell concentration of 10^6 cells/ml the trend reverses. The same results are plotted in Figs. 3.10 and 3.11 for *B. subtilis* and *B. sphaericus*. The maximum increment of 28 days compressive strength of RCA concrete is found to be 20.93% for *B. Subtilis* (B-3a) and 35.87% for *B. sphaericus* (B-3b) with respect to RCA control mix with an optimum cell concentration of 10^6 cells/ml. The same trend is also reported for bacterial NCA concrete in the literature [Chahal *et al.* 2012a, 2012b]. This increase of compressive strength may be due to the precipitation of $CaCO_3$ by bacteria on the micro-organism cell surfaces and within the inner side of the concrete which is confirmed in the microstructure analysis (refer section 3.4.3.5 and 3.4.3.6). The compressive strength is improved by the microbiological precipitation of $CaCO_3$ in the micro pores of concrete. Since the cell concentration of 10^6 cells/ml yields maximum compressive strength of RCA concrete the further investigation on bacterial concrete are conducted only considering this cell concentration for both the two selected bacteria (B-3a and B-3b).



(a) RCA concrete with *B. subtilis* (B-3a)

(b) RCA concrete with *B. sphaericus* (B-3b)

(c) RCA concrete without bacteria (Control)

(d) NCA concrete without bacteria

Figure 3.9: Photographs of fresh concrete specimens

Table 3.15: Effect of bacteria on compressive strength (MPa) at 7 & 28 days

Mixture Name	7 days		28 days	
	B. subtilis	*B. sphaericus*	*B. subtilis*	*B. sphaericus*
NCA (0 cells/ml)	33.15	33.15	44.08	44.08
Control (0 cells/ml)	29.06	29.06	38.22	38.22
B-1 (10^1cells/ml)	31.27	30.13	41.02	43.10
B-2 (10^3cells/ml)	32.70	33.59	43.13	46.71
B-3 (10^6cells/ml)	34.15	38.86	46.22	51.93
B-4 (10^7 cells/ml)	32.80	34.64	44.60	48.55

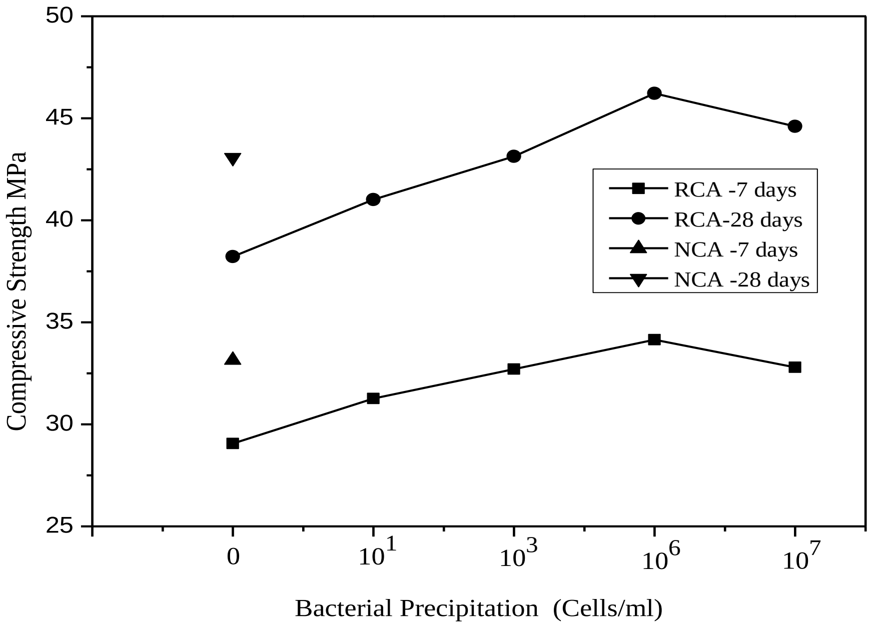


Figure 3.10: Effect of *B. subtilis* on compressive strength

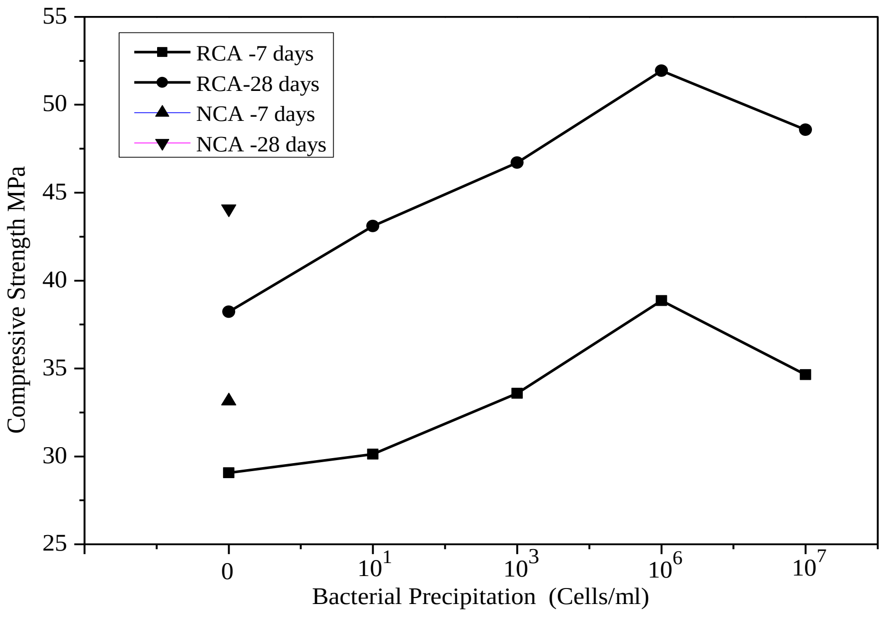


Figure 3.11: Effect of *B. sphaericus* on compressive strength

3.3.3.2 Drying Shrinkage

Drying shrinkage of the concrete specimen is measured after 28 days curing including 7 days moist air curing. The drying length and drying shrinkage obtained for four specimens, NCA, RCA Control, B-3a and B-3b are tabulated in Table 3.15. It is noticed that the addition of bacteria to RCA concrete decreases the drying shrinkage. This can be attributed to denser RCA concrete formed by bacterial activity. It is to be noted that the drying shrinkage of RCA control concrete is more than that of NCA concrete. Similar observations are also reported in previous literature [Kou and Poon 2009; Manzi et al. 2013; Jose and Soberon 2002; and Eguchi *et al*. 2007]. The increase of drying shrinkage of RCA control concrete without bacteria is perhaps due to the shrinking of old mortar adhered to the surface of RCA. However, previous studies [Kou and Poon 2012; Kou and Poon 2009] show that it can be controlled by reducing the *w*/*c* ratio suitably. Another cause for higher drying shrinkage of RCA control concrete maybe its low elastic modulus, as compared to NCA concrete, which offer less restraint to the potential shrinkage.

Table 3.16: Drying shrinkage of the concrete specimens

Type of Concrete	Drying Length (mm)	Drying Shrinkage (%)
NCA	0.135	0.09
RCA Control	0.260	0.17
B-3a (*B. subtilis*)	0.040	0.03
B-3b (*B. sphaericus*)	0.090	0.06

3.3.3.3 Air Content

Table 3.16 shows the results of air content in the four samples, NCA, RCA control, B-3a and B-3b. Air content of both the bacterial concrete samples are found to be slightly more than the RCA control concrete, probably due to the bacterial activity (such as photosynthesis, etc.). The air content of bacterial concrete may be reduced by increasing the mixing time to allow the extra air to flow out of the concrete mix.

Table 3.17: Air content of freshly mixed concrete

Type of Concrete	Air Content (%)
NCA	12
RCA Control	12
B-3a (*B. Subtilis*)	13
B-3b (*B. Sphaericus*)	13

3.3.3.4 Capillary Water Absorption

The capillary water absorption of selected specimens are tested and presented in Fig. 3.12. The slope of these curves is decreasing in nature; similar behaviour was observed in previous studies [Bai and Sabir 2002; and Hanžič and Ilić 2003]. The water absorption for RCA samples (control, B-3a and B-3b) is higher than NCA samples, which is due to the additional water absorbed by the old mortar adhered to RCA. Capillary water absorption of bacterial concrete sample (B-3a and B-3b) is less than that of RCA control specimen. This is attributed to the denser concrete formed by bacterial precipitation of $CaCO_3$.

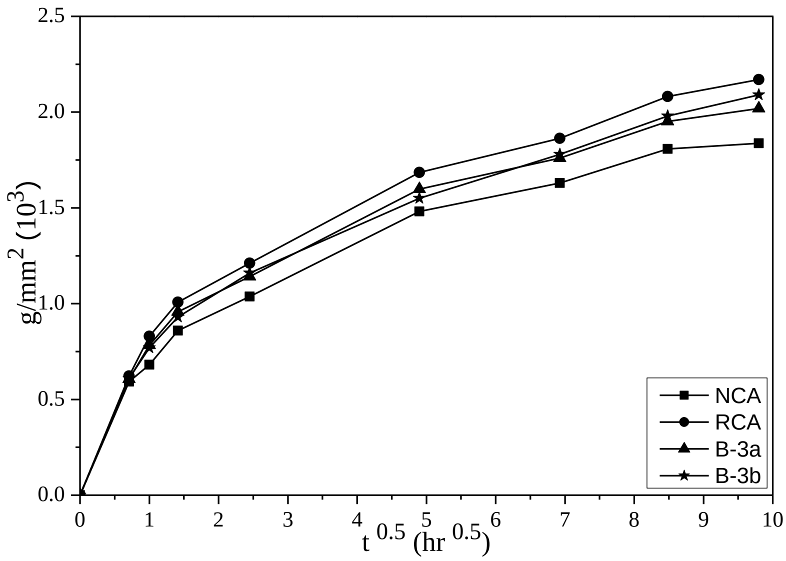


Figure 3.12: Variation of capillary water absorption

3.3.3.5 SEM and EDX

Both the bacterial concrete samples (B-3a and B-3b) and RCA control specimen are examined through SEM, and the results are presented in Figs. 3.13-3.15. Deposition of calcium carbonate as calcite in both the bacterial concrete samples (B-3a and B-3b) is observed through SEM. More crystalline calcium carbonate is observed in the pores of bacterial concrete than RCA control concrete. Fig. 3.16 shows the intensities of various compounds from EDX at a point in the region marked in the Fig. 3.13. The intensities of various compounds indicate the presence of calcite precipitated in the form of calcium carbonate.

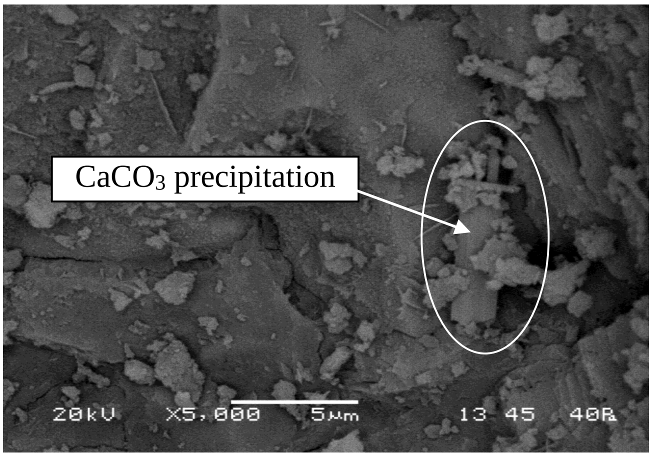


Figure.3.13: SEM of B-3a concrete sample

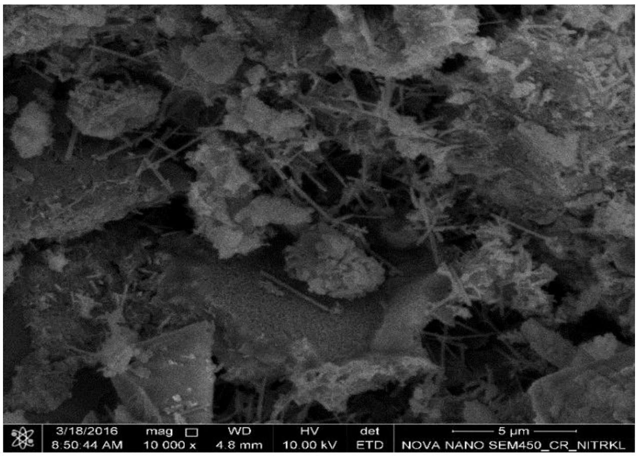


Figure 3.14: SEM of B-3b concrete sample

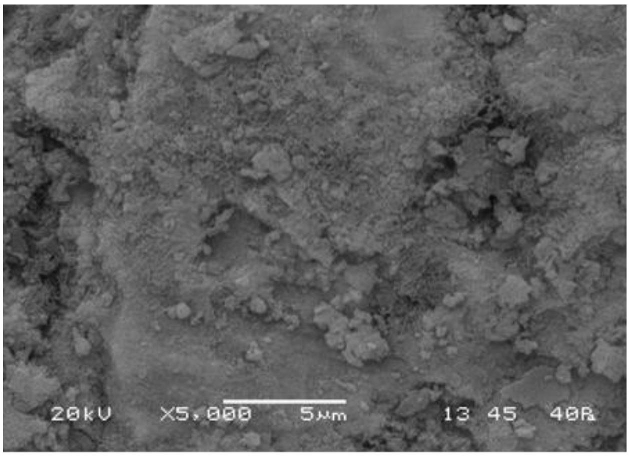


Figure 3.15: SEM of RCA control mix

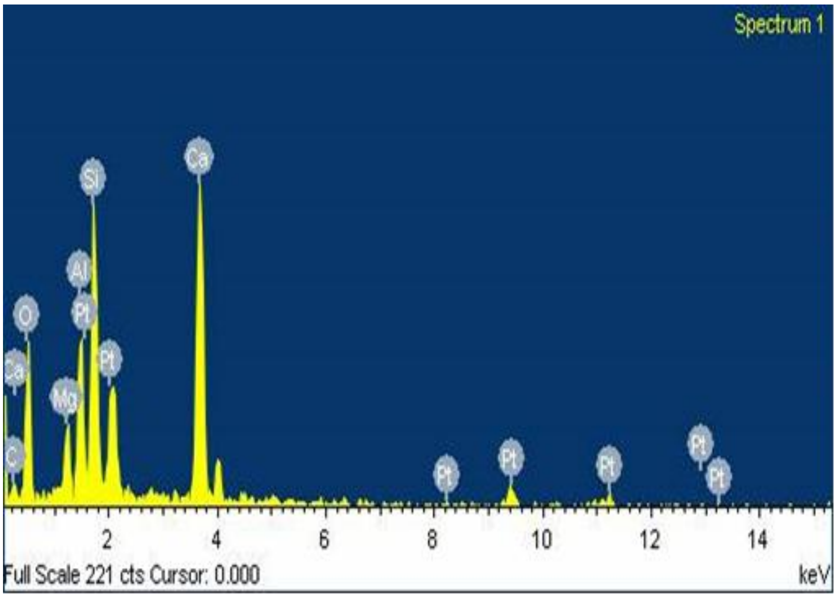


Figure 3.16: EDX of B-3a sample concrete at marked outlines

3.3.3.6 XRD Spectroscopy

The results of the XRD of both the bacterial concrete samples (B-3a and B-3b) and RCA control specimen are presented in Figs. 3.17-3.19. It can be seen that more calcite is precipitated with higher intensity in bacterial concrete. Also, it has been seen that in the case of bacterial concrete, calcite is precipitated in more crystalline form.

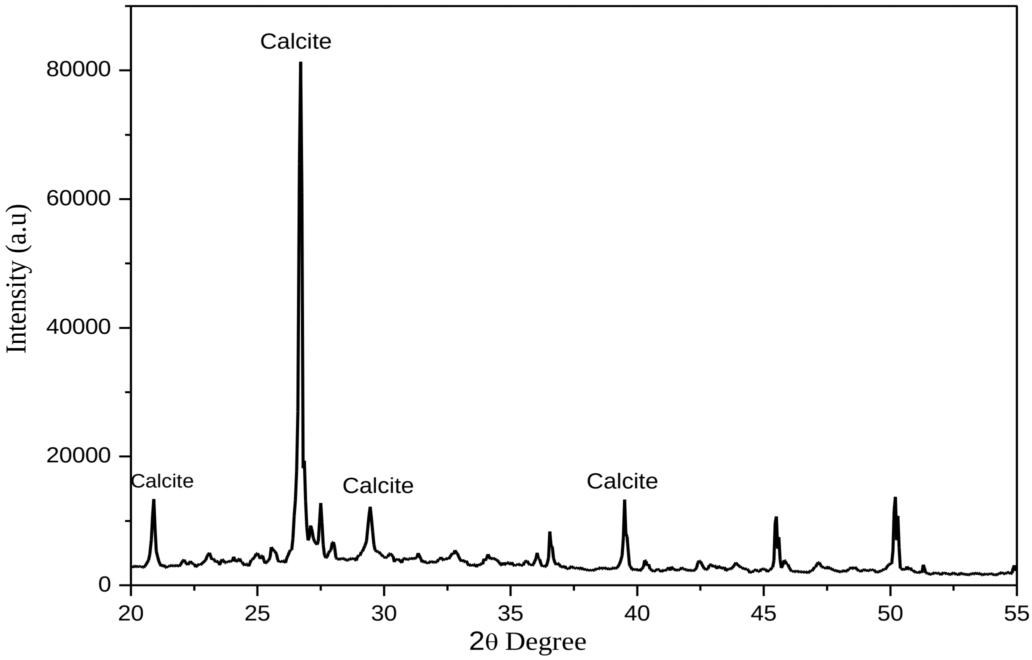


Figure 3.17: XRD analysis of B-3a concrete

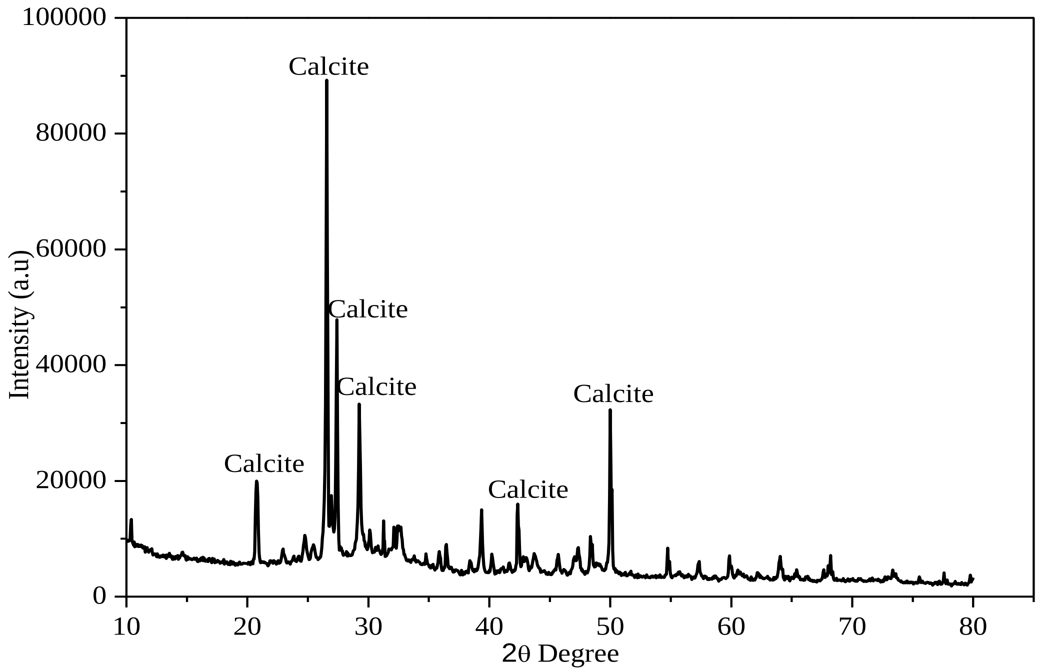


Figure 3.18: XRD analysis of B-3b concrete

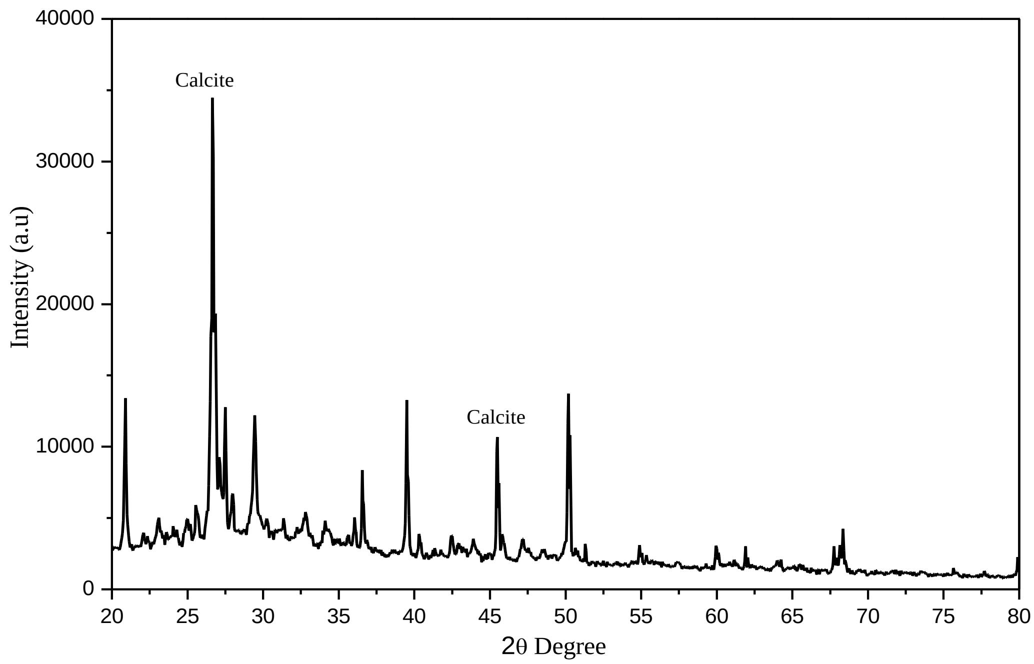


Figure. 3.19: XRD analysis of RCA control

3.4 Cement Mortar using Ureolytic Bacteria

Aggregate is the main source of dispersion for strength of concrete. Also, it does not take part in the mineral precipitation by the bacteria. Accordingly, the purpose of this section is to study the effect of bio-mineralization on the cement-based product avoiding the problems associated with the variation of aggregate properties. Accordingly, the effect of bacteria on some specific properties of cement, such as standard consistency and setting time, and properties of cement mortar such as compressive strength, sorptivity need to be investigated for more clarity. Published literature on the effect of bacteria on the above mentioned aspects are limited. This section discussed the study of cement and cement mortar incorporating *B. sphaericus*. Culture and growth of *B. sphaericus* for this part of work are carried out similar to the methods discussed in the section 3.3.1. The specifications of materials, Portland slag cement and fine aggregate, used for this study are explained in section 3.2.1

3.4.1 Effect of Bacteria on the Properties of Fresh Cement Mortar

Standard tests on cement is conducted for the consistency of cement as per Indian standard IS 4031 Part 4 (1988). The consistency of the cement paste is found to be 32%. In order to check the effect of bacteria in the initial and final setting time of cement, standard tests for setting time are conducted on cement mortar as per Indian standard IS 455 (1989).

Previous literature on the influence of bacteria in the setting time are very limited. No study has been reported on the initial and final setting time of cement by using *B. sphaericus*. Initial and final setting time of cement paste with and without bacteria are found out and presented in Table 3.17. Initial setting time of mortar with the inclusion of bacteria is 50 minutes which is marginally lower than the normal mortar (52 minutes). Similarly, final setting time of mortar with bacteria is 6 Hrs 07 Minutes while for normal mortar, it is 6 Hrs 00 Minutes. The effect of bacteria on the setting time is found to be very negligible.

Table 3.18: Comparison of setting time of cement

Specimen	Initial Setting time (minutes)	Final Setting time
Cement paste (Normal)	52	6 Hrs 00 Minutes
Cement paste (with bacteria)	50	6 Hrs 07 Minutes

3.4.2 Effect of Bacteria on the Properties of Harden Cement Mortar

The variation of compressive strength of mortar cubes with different concentrations of *B. sphaericus* are studied. Cement to sand ratio of 1:6 and water cement ratio of 0.55 are considered to prepare the mortar cubes. Accordingly, the amount of cement, sand and water are calculated as shown in the Table 3.18. Mortar cubes are prepared by mixing the water containing cell culture in selected concentrations, with the cement and sand. This mortar cubes are referred in this study as bacterial mortar cubes. The bacterial mortar cubes are represented by prefix ‘B’ as shown in Table 3.18. Bacterial cubes are cured in 2% urea (in water) and 25mM $CaCl_2$ per ml of curing water. Mortar cubes without bacteria is referred as control cube specimens. Control cubes, denoted as CT (refer Table 3.17) are cured with normal tap water. In order to study the effect of curing medium on control specimen a set control cubes are cured using 2% urea and 25mM $CaCl_2$ per ml of curing water. This set is referred as CUC (mortar cured with urea). Cubes are casted in triplicates and compacted in a vibration machine. After demoulding, all specimens are cured until the testing.

3.4.2.1 Variation of Compressive Strength with Cell Concentration

The cubes are tested in a load controlled universal testing machine to obtain the compressive strength at 7 days and 28 days. The compressive strengths of all the specimens are presented in Table 3.19. It can be observed from the table that as the cell concentration increases the compressive strength at both 7 days and 28 days increases initially and then decreases. It can be seen that the maximum percentage increase in compressive strength of about 58% is observed (at 7 days) in the mortar cube having a cell concentration of 10^7 *cells/ml*. The variation of the compressive strength at 7-day and 28-day is also expressed graphically in Fig. 3.20. The maximum strength at 28 days is about 23% over the control specimen and this occurs at a cell concentration of about 10^7 *cells/ml* and hence this cell concentration can be treated as optimum dosage to obtain maximum compressive strength.

Table 3.19: Casting details of mortar cubes for compressive strength and sorptivity test

Mortar cube ID	Bacteria concentration (cells/ml)	Number of specimens for			Mix proportion			Curing medium
		7 day strength	28 day strength	Sorptivity test	Cement (kg)	Sand (kg)	Water (*ml*)	
CT	0	3	3	3	0.13	0.77	72	†
CUC	0	-	3	-	0.13	0.77	72	Ø
B1	10^5	3	3	3	0.13	0.77	72*	Ø
B2	10^6	3	3	3	0.13	0.77	72 *	Ø
B3	10^7	3	3	3	0.13	0.77	72 *	Ø
B4	10^8	3	3	3	0.13	0.77	72 *	Ø
B5	10^9	3	3	3	0.13	0.77	72 *	Ø

* indicates that the volume of water including bacteria and culture medium
† indicates tap water as curing solution
Ø indicates a mix of tap water, urea and calcium chloride as curing solution

Maximum percentage increase in compressive strength in a previous study shows similar results. Ghosh *et al.* (2005) reported that maximum improvement in compressive strength of mortar at 28 days is about 25% with 10^5*cells/ml* by using *Bacillus pasteurii*. Achal *et al.* (2011) had reported increase in compressive strength of mortar cube incorporating *B. sphaericus* to be about 23% and 36% at 7days and 28 days respectively. An increase of about 18% in strength of mortar with *B. pasteurii* at 28 days is reported by Ramachandran *et al.* (2001). Ramachandran *et al.* (2001) found that in initial days of curing, bacteria shows good nourishment due to filling of pores mortar cubes by microbiologically induced mineral precipitation.

The compressive strength at 28 days of the specimen CUC (urea cured) is found to be about 5.67MPa. The compressive strength of CT specimen, cured with normal tap water at 28 days is about 5.9MPa. The change in compressive strength due to curing solution is found to be negligible. Negligible difference in compressive strength implies that curing solution alone has no significant effect on the specimen. The increase in compressive strength in other bacterial specimens (B1, B2, B3, B4 and B5) are due to the bacterial activity in the presence of (2% urea and 25mM $CaCl_2$ per ml of curing water) curing solution.

Table 3.20: Compressive strength of mortar cubes with different bacteria concentrations

Mortar cube ID	Cell concentration (cells/*ml*)	Mean compressive strength (% increase) at 7 days (MPa)	Mean compressive strength (% increase) at 28 days (MPa)	Remark
CT	0 (Control)	3.44	5.90	
CUC	0 (Control)	3.01 (-14%)	5.67 (-4%)	
B1	10^5	4.46 (29.70%)	6.98 (18.30%)	
B2	10^6	5.34 (55.23%)	7.02 (18.98%)	
B3	10^7	5.44 (58.23%)	7.28 (23.38%)	Maximum
B4	10^8	4.91 (42.73%)	6.19 (4.90%)	
B5	10^9	4.71 (36.90%)	6.10 (3.38%)	

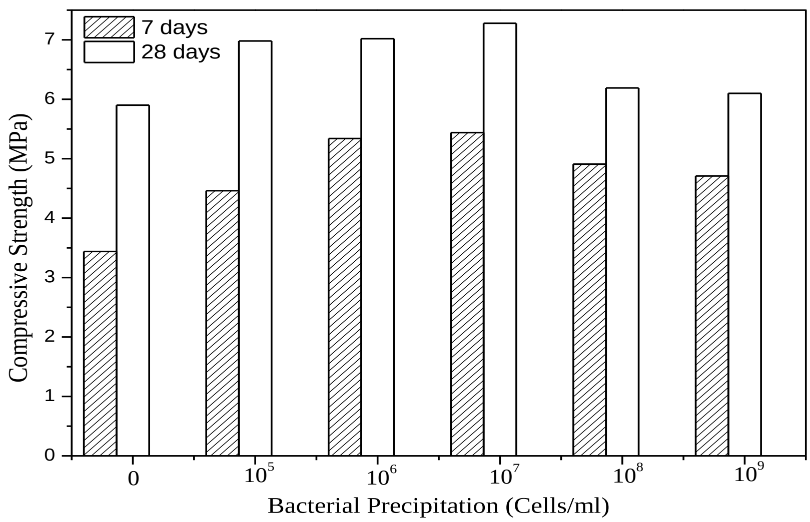


Figure 3.20: Variation of compressive strength with cell concentration – at 7 and 28 day

3.4.2.2 Sorptivity

Sorptivity test is conducted for both control and all bacterial specimens (B1, B2, B3, B4 and B5). The casting details of the specimens are provided in Table 3.20. The cumulative water absorption versus square root of time in hours (√t) for all bacterial specimens are computed as shown in Fig. 3.21. For durability point of view, sorptivity coefficient should be minimum. Sorptivity coefficient, S (mm/h$^{0.5}$), slope (Gonen and Yazicioglu, 2007) of the trend lines, for all the specimens are found out from Fig. 3.21 and they are listed in Table 3.20. It can be seen that sorptivity coefficient is minimum (0.79) for B3, where bacteria concentration is about 10^7 cells/ml. The lesser values of sorptivity coefficient imply that concrete is denser. This may be due to sealing of the pores by carbonation which in turn increase durability. Similar observations are also reported elsewhere (Gonen and Yazicioglu, 2007).

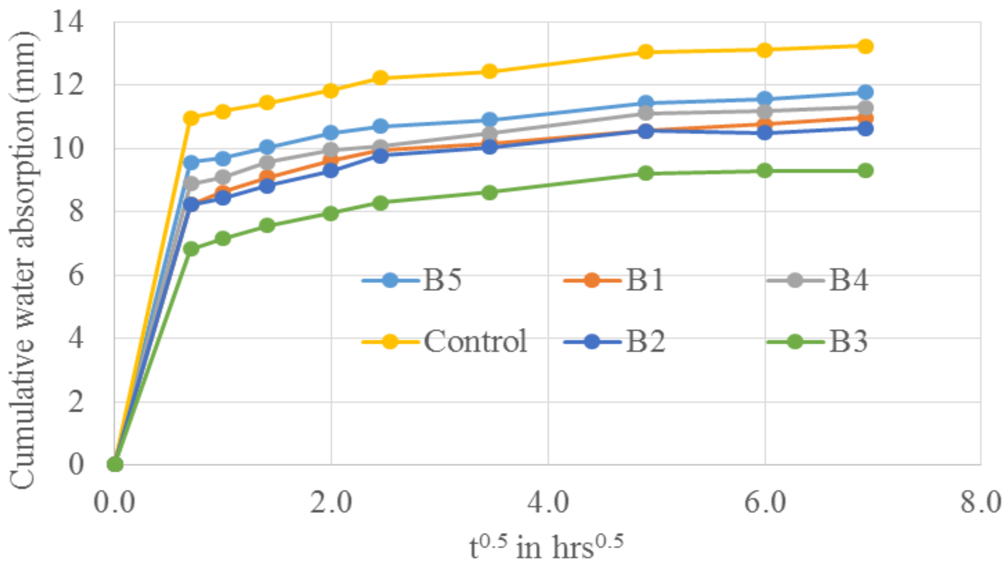


Figure 3.21: Cumulative water absorption for various cell concentrations

Table 3.21: Sorptivity coefficients of all specimens

Specimen ID	Bacterial conc. (cell/ml)	Sorptivity coefficient, S	Compressive strength (MPa)
Control	0	1.00	5.67
B1	10^5	0.89	6.98
B2	10^6	0.86	7.02
B3	10^7	0.79	7.28
B4	10^8	0.90	6.19
B5	10^9	0.90	6.10

3.4.2.3 XRD Spectrometry

The addition of bacteria into the mortar cubes resulted in the formation of calcite crystals. XRD is used to quantify the intensity of various compounds present in both bacterial and control specimens. Fig. 3.22 shows the results of the XRD and it can be seen that the number of calcite peaks (9 numbers of ‘C’) are more in bacterial mortar cube samples and less in control sample (4 numbers of ‘C’). The increase in number of peaks signifies that the presence of calcite is more in bacterial cubes than in control cubes. The sharp peaks of the XRD analysis indicate the crystallinity of the calcium carbonate. Similar conclusions are also reported elsewhere (Chahal *et al.* 2012). This increase in calcite content is perhaps responsible for the increase in compressive strength of bacterial mortar cubes.

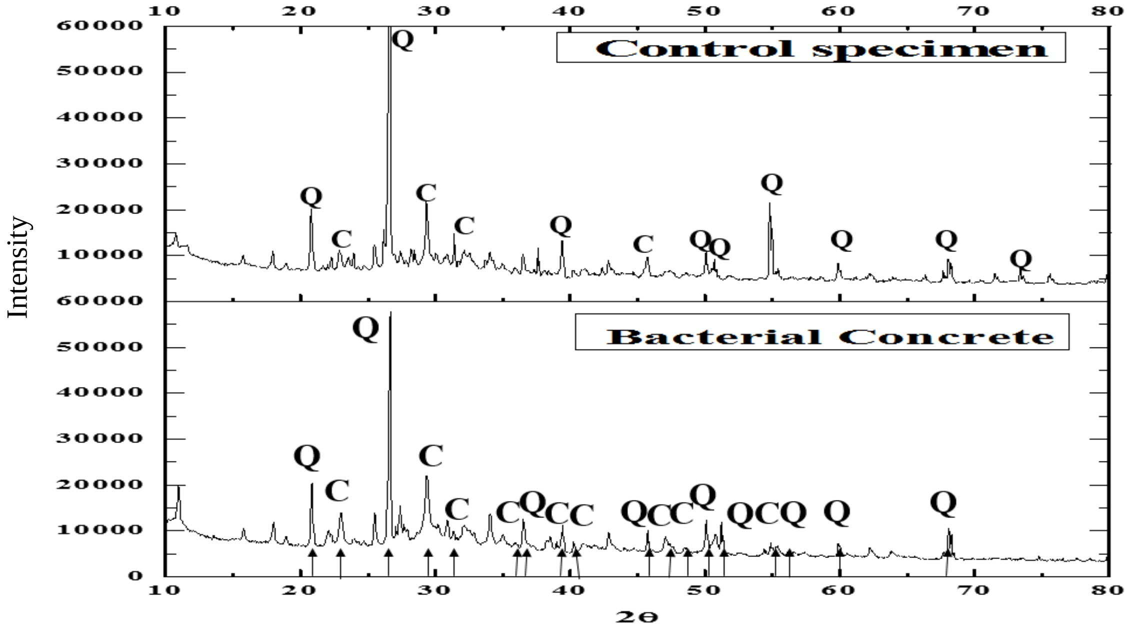


Figure 3.22: XRD of bacteria and control mortar cubes ('Q' represents quartz or silica and 'C' represents calcite).

3.4.2.4 FESEM

Fig. 3.23 shows rod shaped impression which is consistent with the shape of *B. sphaericus* (Siddique and Chahal. 2011). While, Fig. 3.24 shows the FESEM images of samples from control and bacterial specimen (B4) for 7 days of curing. Fig. 3.25 shows the images of the same samples for 28 days curing. The white patches of calcite crystals can be observed in these images and it is found that the amount of calcite crystals is more in the bacterial samples than in the control samples. The presence of the crystalline calcite may be the reason for the improvement of compressive strength.

In order to check the continuous variation in the microstructure of the cement mortar due to the presence of bacterial calcite precipitation, analysis of FESEM at 7, 14 and 28 days are carried out. Figs. 3.26(a) - 3.26(c) show the FESEM images of bacterial mortar cubes (B4) after 7, 14 and 28 days of curing respectively. It is observed that needle shaped structures are seen more in concentration and their number increases with number of days of curing and are less when it reaches a curing period of 28 days. Concentration of Lamellar rhombohedra structures increases as the curing period increases.

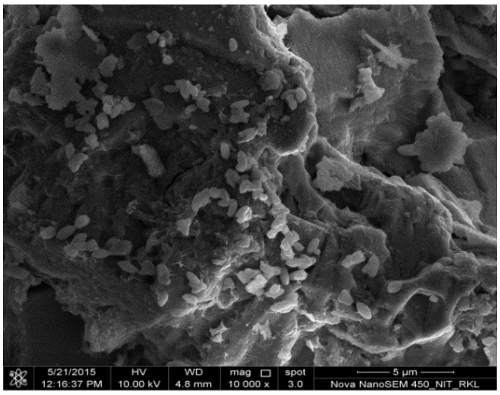
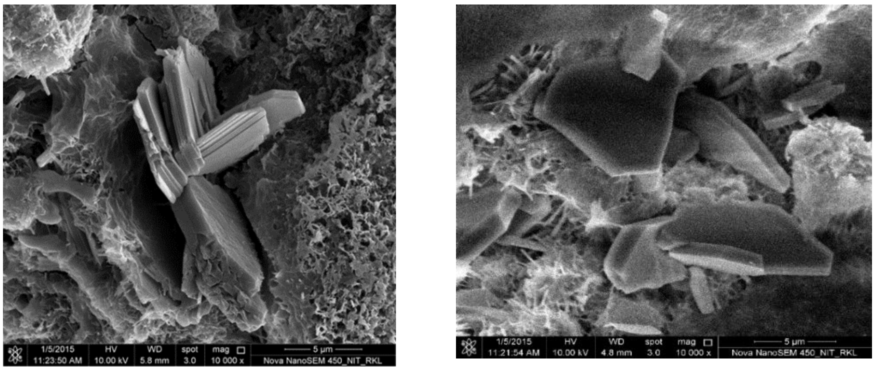
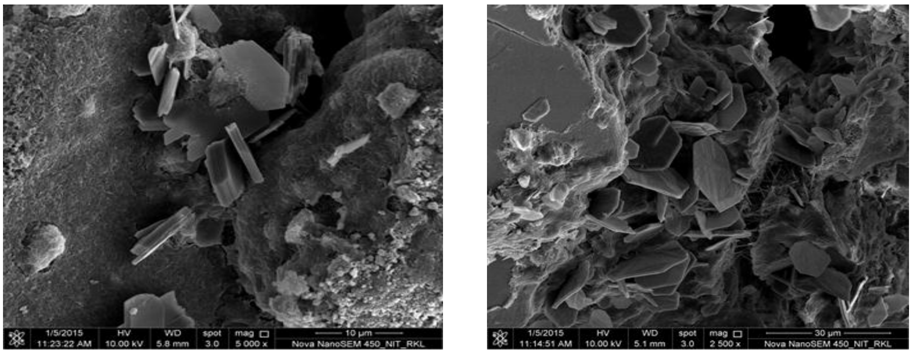


Figure 3.23: FESEM image showing bacteria spreading over calcite crystals



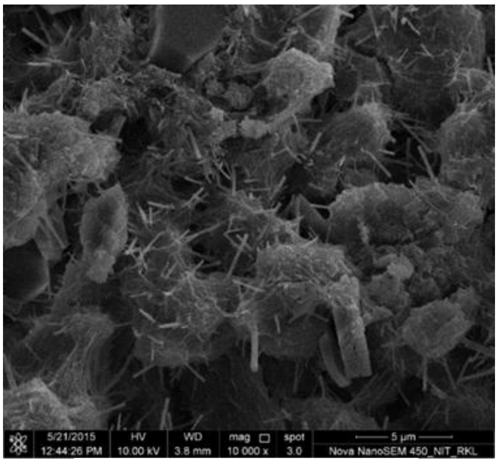
Control specimen Bacterial specimen B4

Figure 3.24: FESEM image of cubes after 7 day curing

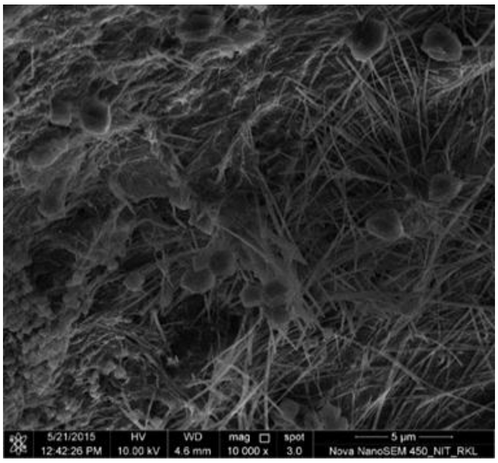


Control specimen Bacterial specimen B4

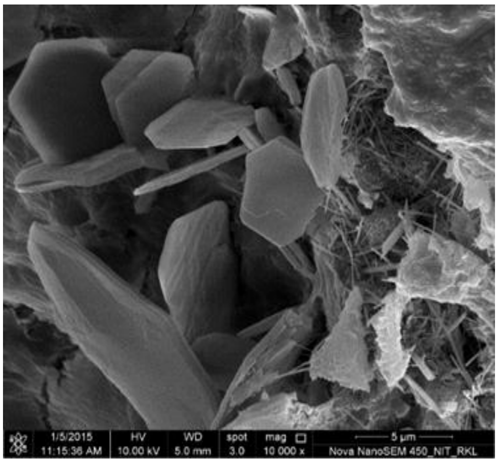
Figure 3.25: FESEM images of cubes after 28 day curing



(a) After 7 days curing



(b) After 14 days curing



(c) After 28 days curing

Figure 3.26: FESEM images of cubes

3.5 Conclusions

The objectives of the present work was to investigate the relationship of w/c ratio with compressive strength of RCA concrete considering age and number of recycling and to study the behaviour of RCA concrete with regard to the capillary water absorption, drying shrinkage, air content, flexural strength and tensile splitting strength. Experiments are conducted to study the above mentioned aspects and following are the major conclusions from the present study.

It is well known that the properties of concrete made with RCA are inferior to the normal concrete. The first part of this chapter discussed the aspects such as number of recycling and the age of the RCA, and its effects on the mechanical properties of RCA concrete. Second part of this chapter presented the experimental results of enhancement of mechanical properties of RCA concrete using two types of ureolytic bacteria. The last part of this chapter investigated properties of cement and cement mortar incorporating bacteria. The salient conclusions from each part of the study are summarised below.

<u>Behaviour of RCA concrete</u>

- The compressive strength of concrete prepared from older (2 years, RC-2) aggregate is found to be lower in comparison to RC1 (1-year-old). The reduction of compressive strength was about 6%. The reduction in compressive strength was probably higher amount of adhered porous mortar which reduces the strength of aggregate significantly.
- The strength of NCA concrete is higher than RCA concrete at lower w/c ratios. However, a reversal of this trend, i.e. RCA concrete shows higher compressive strength than NCA concrete after a particular threshold *w/c* ratio. The present study found that the RCA concrete requires a threshold minimum quantity of water depending upon the parent adhered mortar to contribute to the strength. This minimum quantity of water in terms of *w/c* ratio for RC-1 and RC-2 was about 0.37 and 0.42 respectively. In order to obtain higher compressive strength for RCA (than NCA), *w/c* ratio should be higher than the above mentioned minimum limits.
- The compressive strength of concrete after successive (two times) recycling, N2-RC-1 is less than that of RC-1 (one time) and the decrease in strength of N2-RC-1 is about 2% compared to that of RC-1. N2-RC-1 shows higher compressive strength than NCA for w/c ratios higher than 0.42. The successive recycling reduces the quality of the adhered mortar and this may be reason for the decrease in strength after further recycling.
- Capillary water absorptions of RC-1 and RC-2 concrete are about 11% and 76% more compared to that of NCA. It is found that the capillary water absorption of N2-RC-1 is about 9 times larger than both RC-1 and NCA concrete. This abrupt increase of water absorption behaviour of N2-RC-1 leads to conclude that successive recycling may yield poor quality of aggregates that may not suit for concrete.
- The drying shrinkage strain of RC-1 and RC-2 are about 1.9 and 2.6 times more than that of NCA concrete respectively whereas that of successive recycled concrete, N2-RC-1 is about 1.2 times more than RC-1, which shows that successive recycling increases the drying shrinkage strain of concrete.
- Air contents of RC-2 and N2-RC-1 are found to be higher than that of NCA and RC-1.

- While the decrease in splitting tensile strength of RC-2 concrete compared to RC-1 is in the range of 14-28%, the same in flexural strength is in the range of 6% to 21%. The successive recycling reduces the splitting tensile strength and flexural strength by 6% and 12% respectively.

Enhancement of RCA concrete using bacteria

- Properties of RCA concrete such as compressive strength, capillary water absorption and drying shrinkage are improved by the addition of *B. subtilis* bacteria.
- The compressive strength of RCA concrete at 28 days is found to be increased by 20.93% for *B. subtilis* (B-3a) and 35.87% for *B. sphaericus* (B-3b) with respect to RCA control mix at an optimum cell concentration of 10^6 cells/ml.
- Both bacillus bacteria play vital roles for increment in compressive strength of RCA concrete due to the calcium carbonate precipitation in the pores. Calcium carbonate precipitation by both bacteria in the form of calcite is confirmed through microstructure analysis using SEM, EDX and XRD.
- Both bacteria decrease the drying shrinkage strain and capillary water absorption of RCA concrete and thereby enhances the durability. This can be attributed to denser RCA concrete formed by bacterial activity.
- Air content in bacterial RCA concretes are found to be slightly more than control mix RCA concrete during the initial stage of mixing. This can be reduced perhaps by increasing the mixing time to allow the extra air to go out of the concrete mix. Generalised conclusions on this aspect require further research.

Effect of bacteria on cement and cement mortar

- *B. sphaericus* bacteria is found to be not altering the setting times of the cement paste.
- It is observed that *B. sphaericus* can survive in alkaline concrete like environment and can produce Calcium Carbonate. Addition of bacteria alone cannot improve the properties of concrete/cement mortar. Ureolytic bacteria require urea and a source of calcium to produce $CaCO_3$.
- Compressive strength (at 7-day and at 28-day) of mortar cube is found to be increasing with the increase of bacteria cell concentration up to 10^7 cells/ml.

However, it is found that further increase of bacteria concentration reduces the compressive strength of cement mortar.

- The optimum doses of bacteria is found to be 10^7 cells/ml and the corresponding increments of average compressive strength are observed to be 58% (at 7-day) and 23% (at 28-day).
- Curing solution alone does not have any impact on the compressive strength of concrete.
- Sorptivity coefficient decreases as the concentration of bacteria increases which increases the durability of specimen. The minimum sorptivity coefficient is obtained for a cell concentration of 10^7 cells/ml.
- The XRD study shows that the presence of more calcite peaks in bacterial mortar sample at 28 days than the control specimen.
- The morphology of the bacterial calcite is studied by FESEM. The direct involvement of bacteria in calcite production can be inferred by rod shaped impressions which is consistent with the dimensions of the bacteria on the calcite crystals.

Chapter 4

Concrete using Silica Fume and Fly Ash

4.1 Introduction

Usage of substitute minerals in concrete helps the conservation of raw materials, reduces CO_2 emissions and ultimately helps to a cleaner environment. Increased use of supplementary cementing materials in place of cement in concrete structures worldwide contributes to sustainability in construction. SF, a by-product of silicon metal, and FA, a by-product of coal-fired thermal power stations are the two globally available supplementary cementitious materials which possess pozzolanic properties. SF like all other pozzolanic materials is capable of reacting with the calcium hydroxide, $Ca(OH)_2$ liberated during cement hydration to produce hydrated calcium silicate (C–S–H), which is accountable for the strength of hardened concrete. The high content of very fine amorphous spherical (100 nm average diameter) silicon dioxide particles (present more than 80%) is the main reason for high pozzolanic activity of SF. The SF can improve both chemical and physical properties of concrete. Similarly, FA can react with $Ca(OH)_2$ and yield calcium silicate hydrate (C-S-H) gel and calcium aluminate hydrate (CAH) which are active in creating denser matrix leading to higher strength and better durability.

An extensive literature review revealed that majority of the published literatures, if not all, present studies on ordinary Portland cement. However, production of Portland cement in the recent times is almost stopped and the present construction industry depends mostly on slag cement. This fact calls for a detailed study on the concrete made with PSC partially replaced by SF/FA.

Most of the previous studies on SF concrete consider the partial replacement of cement keeping the total weight of cementitious material, fine and coarse aggregate constant. The main purpose of these studies was to evaluate the effect of SF on the behaviour of concrete. However, in practical constructions, SF concrete is prepared as per relevant codes and standards. Many international design codes (IS: 10262-1982, ACI234R-96) advices an extra cement of 10% while mineral admixture is used as partial replacement of cement. Also, due to the differential specific gravity of cement and SF, the mix design results varying weights of aggregates as the percentage replacement of SF varies. Therefore, the concrete behaviour changes due to the combined effect of additional amount of cement, SF and reduced amount of aggregates.

Also, all the previous studies on SF concrete are concentrated in high-strength concrete. No attempt has been carried out using SF as a replacement of cement for low/medium grade concretes (viz. M20, M25).

This Chapter presents experimental investigations on the behaviour of medium grade concrete designed as per industry practice using PSC partially replaced by SF/FA.

4.2 Materials and Test Specimens

This chapter presents the experimental results of concrete made of PSC partially replaced with SF and FA. This section presents a brief description of ingredient materials used in the present study and the details of different test specimens.

SF of grade 920-D having specific surface area of about 19.5 m^2/g obtained from Elkem Private Ltd. is used to replace the cement partially in the present study. Chemical composition of SF is analysed according to ASTM C 1240 and presented in Table 4.1. The cement and SF used in the experiment are tested to check the conformity with the relevant Indian Standards.

FA used in the experiment is tested to check the conformity with the relevant Indian Standards and presented in Table 4.2 PSC conforming to IS: 455-1989; having 28-day compressive strength of 48 MPa is used in this study. The chemical components and physical properties of cement are presented in Chapter 3.

Natural river sand (zone-II) conforming to IS: 383-1970 is used as fine aggregate. The specific gravity and water absorption values are obtained as 2.65 and 0.8%, respectively. Crushed, angular graded coarse aggregate obtained from local quarry having nominal maximum size of 20 mm is used. The specific gravity and the water absorption of the coarse aggregates are 2.75 and 0.6% respectively. Water reducing superplasticizer (Sikaplast 301 I, Polycarboxylic based) having a specific gravity of 1.08 is used in the present study.

Table 4.1: Chemical and Physical Properties of SF

Parameter	Specification	Analysis
Chemical Requirements		
SiO_2 (%)	> 85.00	88.42
Moisture Content (%)	< 3.00	0.15
Loss of Ignition (%)	< 6.00	1.50
Physical Requirements		
>45 Micron (%)	< 10.00	0.72
Pozz. Activity Index (7d)	> 105	137
Sp.Surface (m^2/g)	> 15.0	19.5
Bulk Density (kg/m^3)	500-700	615

Table 4.2: Chemical and Physical properties of FA

Parameter	Specification	Analysis
Chemical Requirements		
SiO_2 (%)	> 35.00	39.88
MgO	< 5.00	1.15
Physical Requirements		
Fineness-Specific surface in m^2/kg	< 320.00	329
> 45 Micron (%)	>34	2.72

Test specimens are prepared for compressive strength, tensile splitting strength and flexural strength test. 15 samples are considered for each mix proportions in each test category. Concrete mixing is done using laboratory rotary mixture machine. The workability of the concrete mixtures is measured using the slump cone test according to IS: 1199-1959. Cubes of size $100 \times 100 \times 100mm$ (Fig. 4.1a), cylinders of size $100 \times 200mm$ (Fig. 4.1b) and prisms of size $100 \times 100 \times 500mm$ (Fig. 4.1c) are casted for the determination of compressive strength, split tensile strength, and flexural strength respectively. All the specimens are cured in normal tap water at natural weather condition up to 28 days.

(a) Concrete cube

(b) Concrete cylindrical samples

(c) Concrete prism

Figure 4.1: Typical specimens prepared in the present study

4.3 Concrete using SF

The first part of this chapter is to study the behaviour of concrete made of PSC cement partially replaced by SF. Seven sets of concrete mixes are prepared by partial replacement of cement with equal weight of SF. Mix designs are prepared as per Indian Standard IS:10262-2009 and weight proportions of cement, SF, natural sand, coarse aggregates, water and admixture are shown in Table 4.3. The SF dosage of 0% (control mix), 5%, 10%, 15%, 20%, 25%, and 30% of the total cementitious material are considered in the present study. The cement content in the control mix is obtained as about 308kg/m^3, while the total cementitious content, in the mixtures where SF is used, is increased to be 338 kg/m^3 (10% more than the cement content of the control mix) as per IS: 10262: 2009. As the percentage of SF increases the cement contents in SF concrete mixtures reduces from 322 kg/m^3 (5% SF) to 237 kg/m^3 (30% SF). It can be observed from the table that the total weight of the aggregate reduces with increased percentage of SF. This is due to the increased volume of cementitious materials (specific gravity of SF is lower than that cement). As SF reduces the workability of the fresh concrete, water reducing superplasticizer is added proportional to the dosage of SF. Water content is kept constant (148 kg/m^3) for all the mixtures considered in this study.

Table 4.3: Mix proportions considered in the present study

Mixture name	Control	5% SF	10% SF	15% SF	20% SF	25% SF	30% SF
Cement (kg/m^3)	308	322	305	288	272	254	237
SF (kg/m^3)	-	16.8	33.8	50.8	67.8	84.8	101.8
Natural sand (kg/m^3)	715	702	700	698	695	694	692
Coarse aggregate (kg/m^3)	1304	1281	1278	1274	1269	1266	1262
w/c	0.48	0.43	0.43	0.43	0.43	0.43	0.43
Water (kg/m^3)	148	148	148	148	148	148	148
Admixture (kg/m^3)	1.23	2.71	3.05	3.39	3.73	4.07	4.41

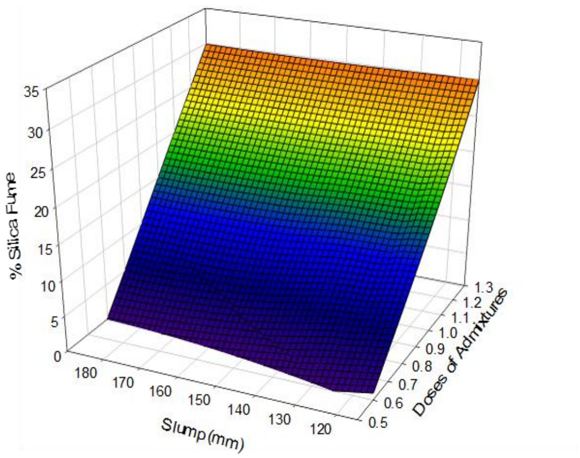


Figure 4.2: Surface plot for a water content of 148 kg/m^3

4.3.1 Dosage of SF and Workability of Concrete

Workability of SF concrete reduces vigorously due to very high specific surface area of SF. Addition of water reducing admixture (Super plasticizer) is necessary for SF concrete to obtain desired workability. Accordingly, dosage of superplasticizer is increased (keeping the water content of 148kg/m^3 constant) with the increase of SF dosage to maintain the slump value within a range of 150 ± 30mm. Slump test is conducted on fresh concrete for all the concrete mixes considered in this study. Fig. 4.2 shows a surface plot between the dosage of SF, dosage of admixture and the slump value obtained.

4.3.2 Mechanical Properties of SF Concrete

Test specimens are prepared from all the seven mixes for compressive strength, split tensile strength, and flexural strength as discussed in the previous section. Average value of the results of fifteen specimens in each of the selected category is considered as the representative result. This section presents the results obtained from the experimental studies.

4.3.2.1 Compressive Strength

Mean compressive strength of concrete obtained for each SF dosage is plotted in Fig. 4.3. It can be seen that the compressive strength, in general, increases with increase of SF dosage. For SF dosage of 5% there is hardly any change in compressive strength. However, as the SF dosage increases beyond 5%, the strength increases gradually to reach a maximum value at 20%. The increase in compressive strengths of about 29%, 40%, 59% and 44%, are observed for the corresponding SF dosages of 10%, 15%, 20% and 25% respectively. There is no further increment in compressive strength observed beyond the SF replacement of 20%.

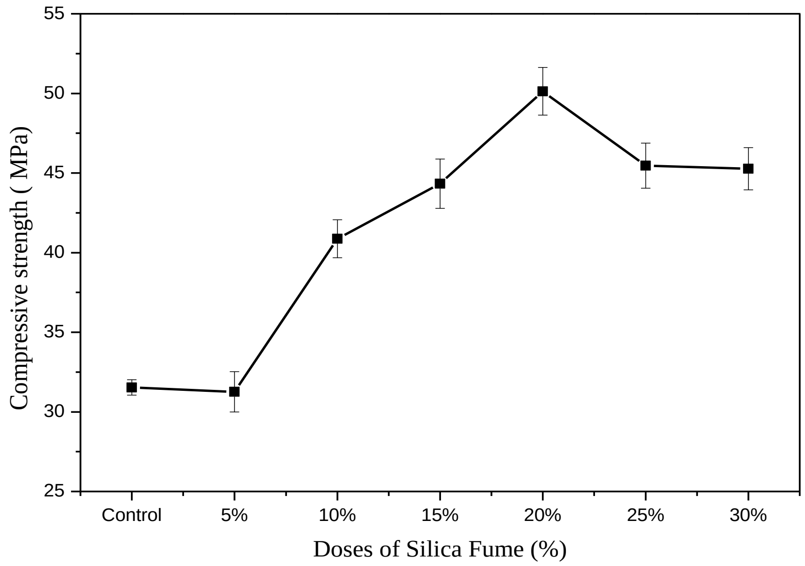


Figure 4.3: Effect of SF on compressive strength development of concretes

A number of published literature [Khedr and Abou-Zeid 1994; Mazloom *et al.* 2004; Bhanjaa and Sengupta 2005; Poon *et al.* 2006; Bingöl and Ilhan 2013; and Sarıdemir 2013] report similar observations. This study is different and unique in terms of two aspects: (a) use of PSC concrete and (b) use of additional 10% cementitious materials for SF concrete. Although the designed strength of the control mix for all these studies are not same, the relative increase of compressive strength (due to SF) obtained in these studies are compared with the results of present study and presented in Fig. 4.4.

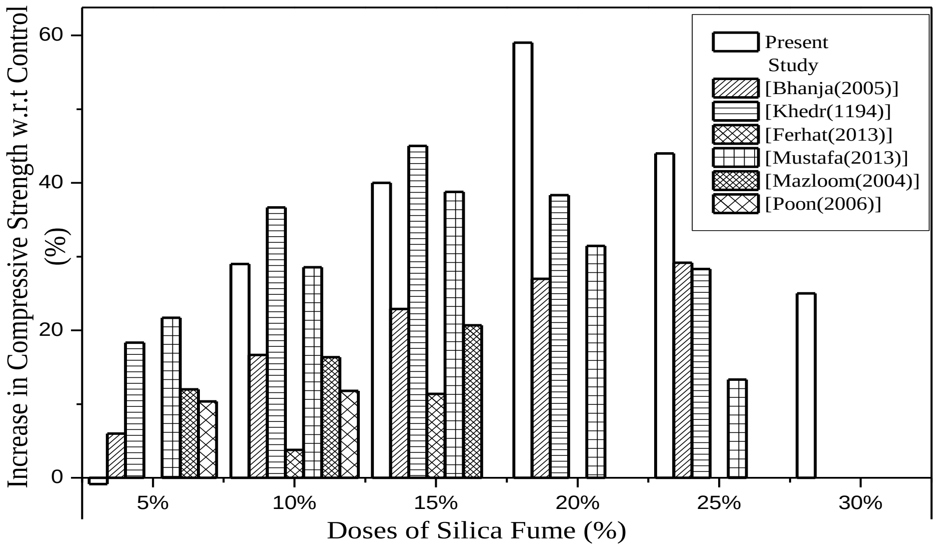


Figure 4.4: Comparison of compressive strength with previous studies

It can be seen that at 5% replacement, the study carried out by [Sarıdemir 2013] is found to be the maximum at about 22% while [Bhanjaa and Sengupta 2005] and present studies yield hardly any improvement in compressive strength. However, at 20% replacement the present study shows an increase in compressive strength of about 59% compared to 38% by [Khedr and Abou-Zeid 1994] which is maximum among the previous studies considered. At 15% replacement, the present study shows that the increase in compressive strength is comparable with that of previous studies. It can be concluded that if the guidelines suggested by IS code is followed in the replacement of SF, a cap of about 21% (59% - 38%) in compressive strength can be achieved at 20% replacement.

4.3.2.2 Tensile Splitting Strength

Tensile splitting strength of SF concrete for selected SF dosages is presented in Fig. 4.5. This figure shows that as the SF dosage increases the tensile splitting strength increases gradually to reach a maximum value of 3.8 MPa at a dosage of 20%. It can be seen that the strength reduces for further increase of SF content. The increase in tensile splitting strength are about 12%, 13%, 33%, 49%, 43% and 42% at 5%, 10%, 15%, 20%, 25% and 30% of SF dosages respectively.

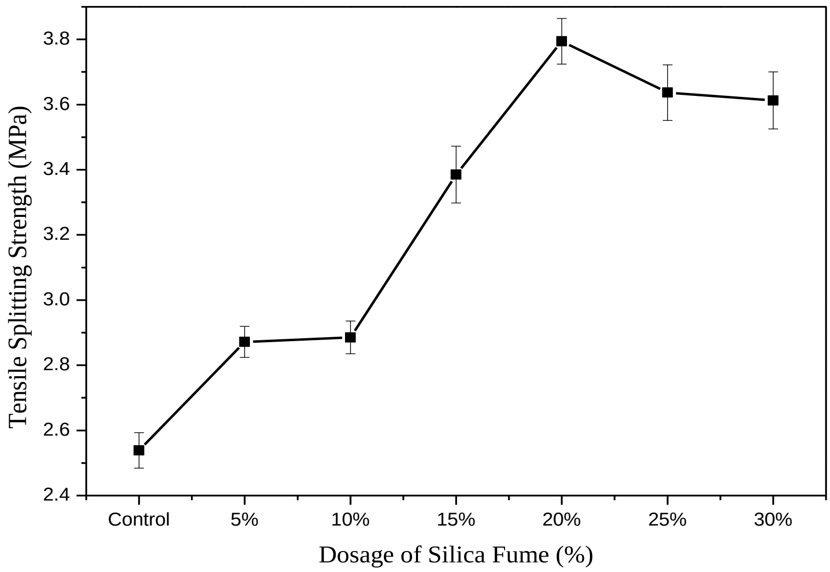


Figure 4.5: Effect of SF on tensile splitting strength development of concretes

It is to be noted that the behaviour of strength increments and the optimum dosage of SF is almost similar for both compressive and tensile splitting strength. The maximum strength is achieved in both cases for SF dosage between 15% and 25%. The relative increase of split tensile strength (due to SF) obtained in two previous studies [Bhanjaa and Sengupta 2005; and Sarıdemir 2013] are compared with the results of present study and presented in Fig. 4.6. It can be seen that for all replacement, the increase in strength is lower in the present study in comparison with previous studies considered. At lower replacement the difference in strength is higher, however at higher replacement, the difference is comparable.

4.3.2.3 Flexural Strength

Flexural strength of SF concrete for various percentage of SF is presented in Fig. 4.7. It can be observed that 5% SF makes no difference in flexural strength. As the replacement of SF increases the flexural strength increases gradually to reach a maximum value of 8.5 MPa at 25% SF. Further increase of SF dosage found to reduce the flexural strength of concrete. The increase in tensile splitting strength is found to be about 4.45%, 5%, 12%, 33% and 18% at the SF replacement of 10%, 15%, 20%, 25% and 30% respectively. It is also observed that the flexural strengths follow slightly different trend compared to that of compressive strength and tensile splitting strength.

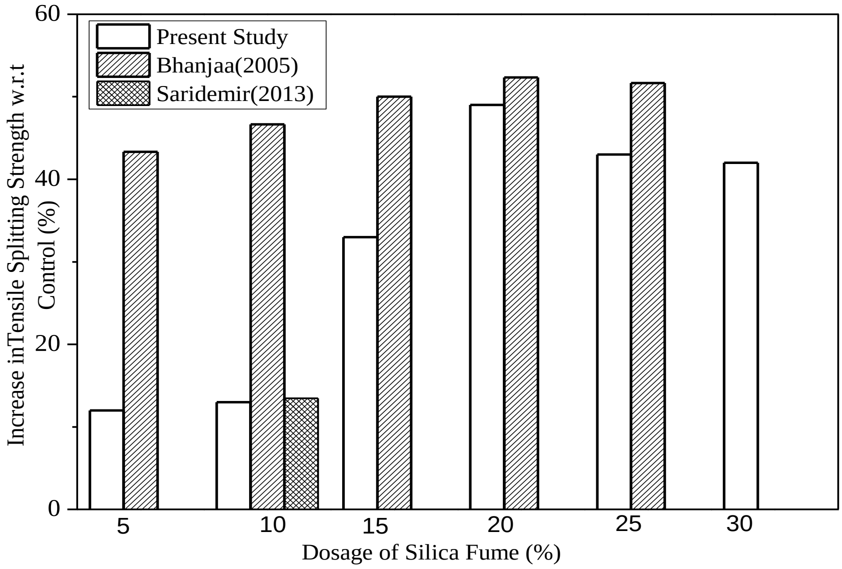


Figure 4.6: Comparison of tensile splitting strength with previous studies

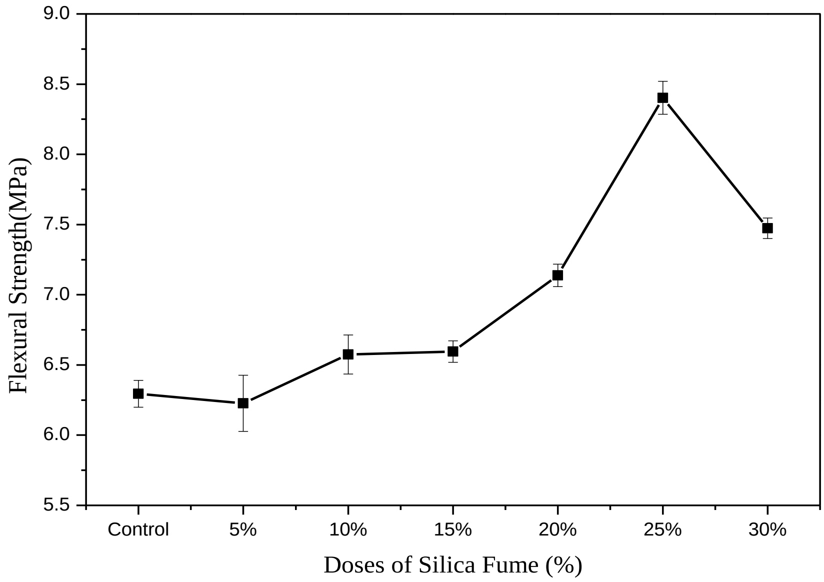


Figure 4.7: Effect of SF on flexural strength development of concretes

Flexural strength of SF concrete obtained in the present study is compared with that of a previous study [Bhanjaa and Sengupta 2005] as shown in Fig. 4.8. It can be seen that for all replacement except at 5%, the increase in flexural strength is higher in the present study in comparison with the earlier study considered. The maximum increase in flexural strength from the present study is about 33% compared to 23% from the previous study at 25% SF replacement. Therefore, it can be concluded that flexural strength of concrete increases about 10% when IS code procedures are followed for mix design of SF concrete.

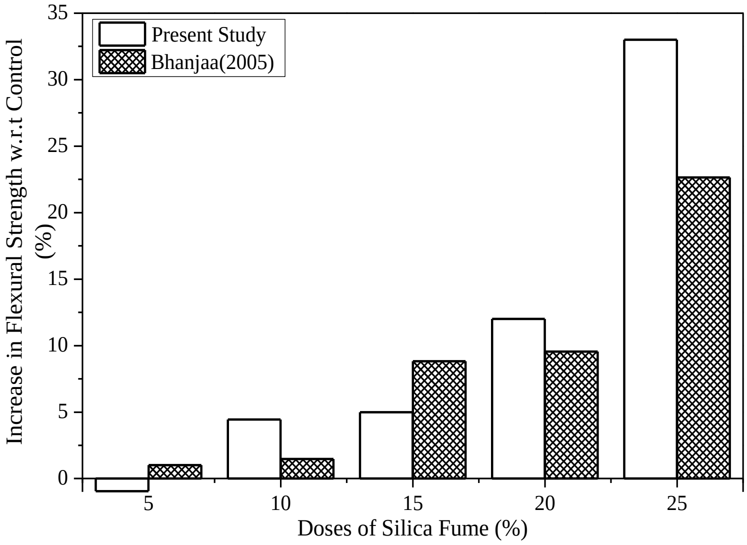


Figure. 4.8: Comparison of flexural strength with previous studies

4.3.3 Correlation between Different Properties of SF Concrete

The compressive strength of concrete is generally treated as the significant parameter for the quality governor. Design codes express the other parameters in terms of compressive strength. The pair of values obtained for the tensile splitting strength and compressive strength from 105 data-set (seven concrete mixes with 15 numbers of samples each) is shown in Fig. 4.9. Similar plots for flexural strength and compressive strength are presented in Fig. 4.10. The data are analysed statistically and best fitted empirical relationships are developed between these quantities by performing a regression analysis and presented in Eqs. 4.1-4.2:

$$f_{sp} = a\left(f_{cc}\right)^{b} \tag{4.1}$$

$$f_{fl} = c\left(f_{cc}\right)^{d} \tag{4.2}$$

Where f_{sp}, f_{fl} and f_{cc} denote tensile splitting, flexural and compressive strengths of concrete, expressed in MPa, respectively. The values obtained for the constants (*a*, *b*, *c* and *d*) along with R^2 value from the regression analysis in the present study are compared with [Bhanjaa and Sengupta 2005] as shown in Table 4.4.

Table 4.4: Values of constants

Investigation	*A*	*b*	R^2 (Eq. 1)	*c*	*d*	R^2 (Eq. 2)
Present Study	0.204	0.745	0.95	0.571	0.669	0.93
Bhanjaa and Sengupta(2005)	0.248	0.717	0.94	0.275	0.810	0.93

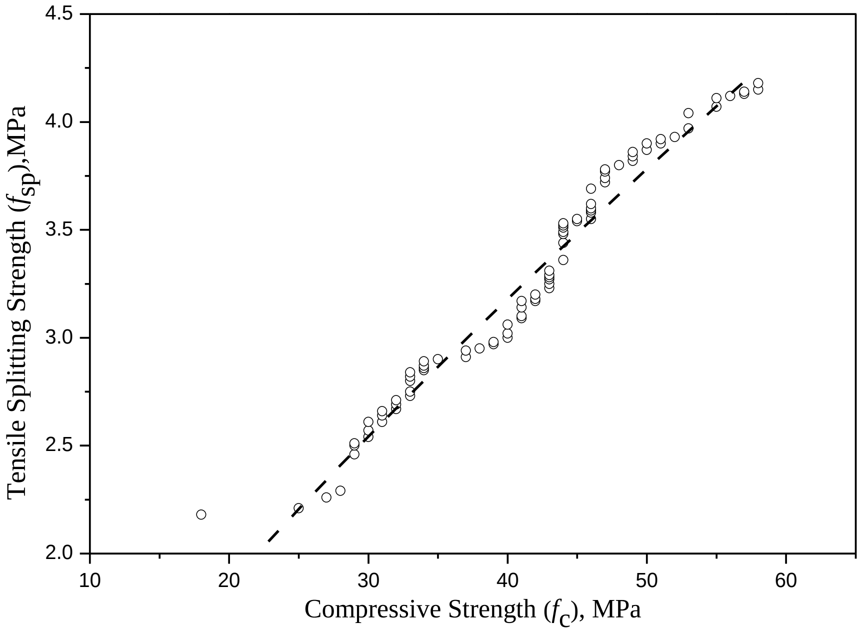


Figure 4.9: Relationship between tensile splitting strength and compressive strength

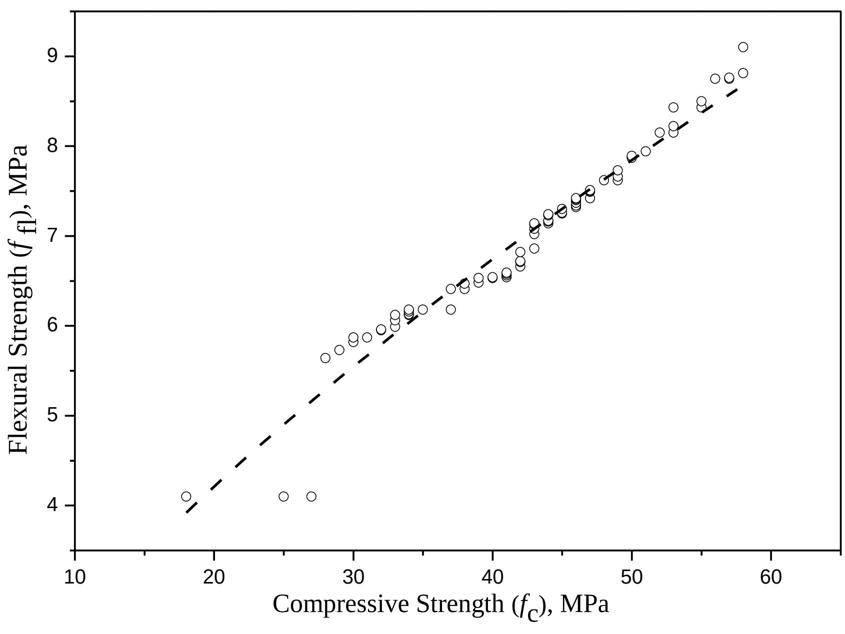


Figure 4.10: Relationship between flexural strength and compressive strength

The UPV tests are carried out on all concrete cubes at 28 days as per IS: 13311(Part1)-1992; before conducting the compressive strength test. UPV measurements are conducted using PUNDIT Lab+ device manufactured by PROCEQ, Switzerland (Fig. 4.11). UPVs were evaluated using direct transmission (cross probing) with path length equal to the concrete specimen width (100 mm).

Figure 4.11: UPV test set up

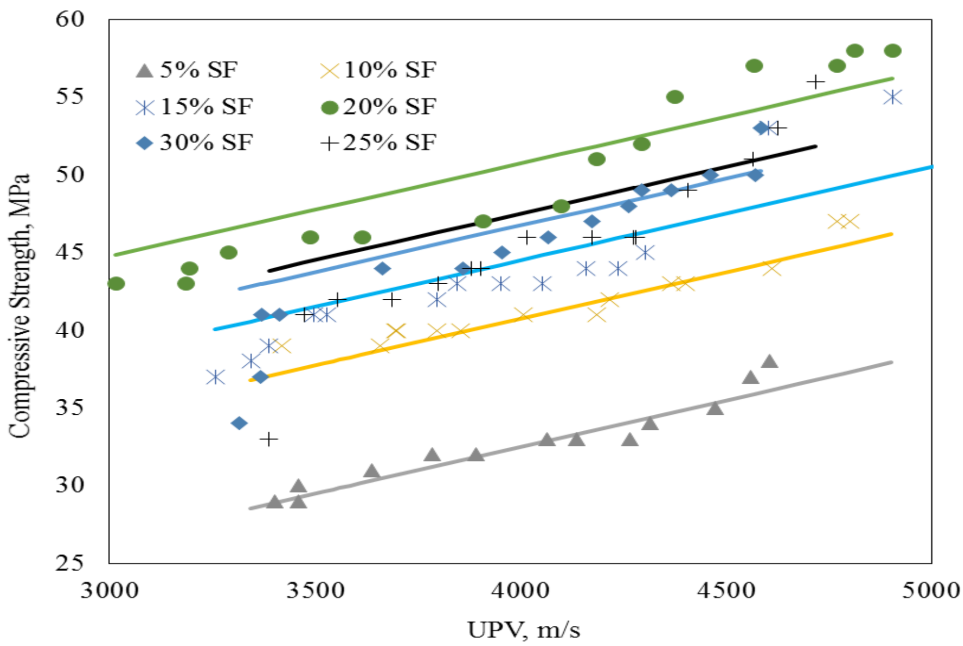


Figure 4.12: Relationship between compressive strength and UPV values

A relationship between UPV values and compressive strengths from 105 data-set is developed using regression analysis as follows:

$$f_{cc} = 0.006\ v + c \tag{4.3a}$$

$$\text{Where, } c = 4.75 + 0.03s(50 - s) \tag{4.3b}$$

Where f_{cc} denotes compressive strengths of concrete (in MPa), v denotes ultrasonic pulse velocity, and s denotes amount of SF replaced (in %).

The application of these equations, however, is limited to moderate grade of control mix (M20-M30). Fig. 4.12 presents the experimental data set of compressive strengths of hardened concrete and corresponding ultrasonic pulse velocities along with the Eq. 4.3 for six different mixes involving SF. This figure shows Eq. 4.3 is fairly matching the experimental values for different concrete mixes.

4.4 Concrete using FA

The second part of this chapter deals with the concrete made of PSC partially replaced by FA. Three sets of concrete mixes are prepared by partial replacement of cement with equal weight of FA. Mix designs are prepared as per IS: 10262-2009 and weight proportions of cement, FA, natural sand, coarse aggregates, water and admixture shown in Table 4.5. The dosages of FA are 20%, 40% and 60% of the total cementitious material. The cement content in the control mix is about 394 kg/m^3. Like SF, FA also reduces the workability of fresh concrete. Therefore, water reducing super-plasticizer is used in the FA concrete to maintain uniform workability. Water content is kept constant (177.3 kg/m^3) for all the mixtures.

Table 4.5: Mix proportions considered for the FA concrete

Mixture name	Control	20% FA	40% FA	60% FA
Cement (kg/m^3)	394	315.2	236.4	157.6
FA (kg/m^3)	-	78.8	157.6	236.4
Natural sand (kg/m^3)	641	641	641	641
Coarse aggregate (kg/m^3)	1114	1114	1114	1114
w/c	0.45	0.45	0.45	0.45
Water (kg/m^3)	177.3	177.3	177.3	177.3
Admixture (kg/m^3)	1.97	2.76	3.15	3.55

4.4.1 Mechanical Properties of FA Concrete

All the specimens mentioned in the previous section are tested according to relevant Indian Standards respectively. Average value of the results of fifteen specimens in each of the selected category is considered. This section presents the results obtained from the experimental studies.

4.4.1.1 Compressive Strength

Mean compressive strength of concrete obtained for each FA dosage is plotted in Fig. 4.13. It can be seen that the compressive strength, in general, increases with increase of FA. A dosage of 20% FA increases the compressive strength for about 14% compared to control specimen. The strength decreases gradually for further increase of FA dosage beyond 20%. The decrease in compressive strength at higher dose may be due to the improper matrix formation in presence of PSC.

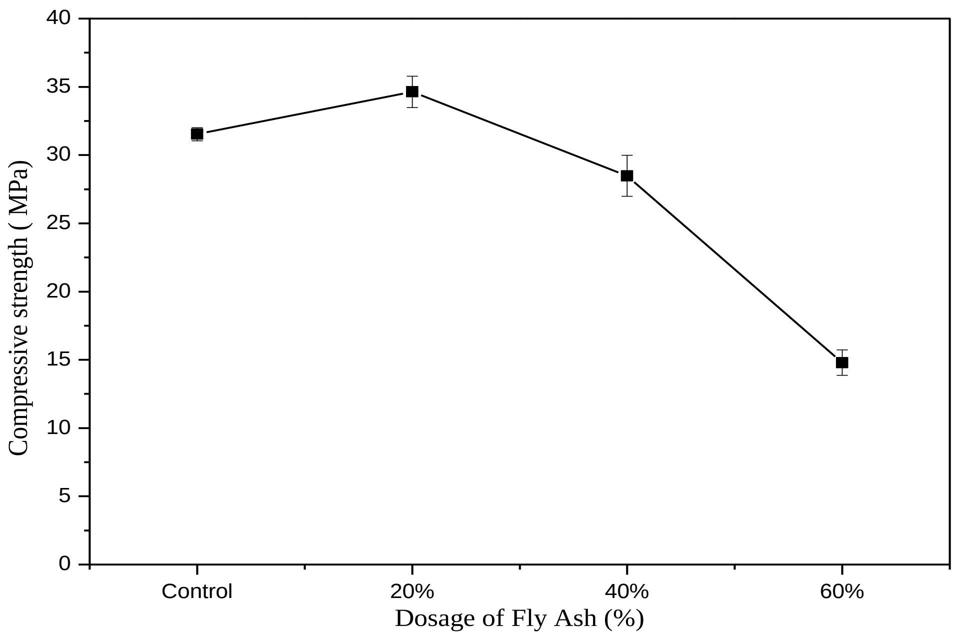


Figure 4.13: Effect of FA on compressive strength development of concretes

4.4.1.2 Tensile Splitting Strength

Tensile splitting strength of FA concrete is presented in Fig. 4.14. This figure shows that FA reduces the tensile splitting strength of concrete gradually. However, the reduction of tensile splitting strength is found to be negligible up to 20% of FA replacement.

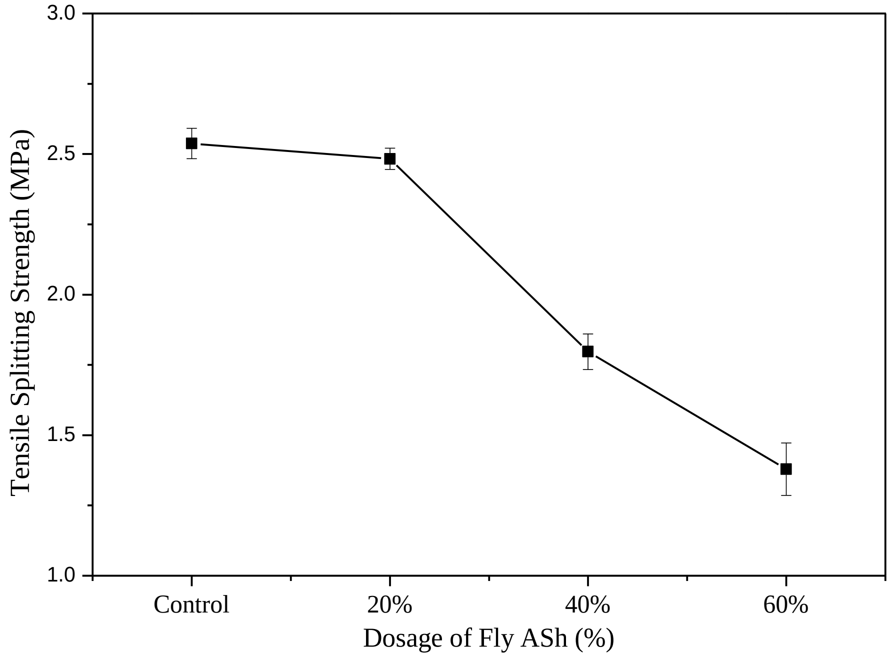


Figure 4.14: Effect of FA on tensile splitting strength development of concretes

4.4.1.3. Flexural Strength

Flexural strength of FA concrete for various dosage of FA is presented in Fig. 4.15. It can be observed from this figure that up to 20% of FA replacement makes no difference in flexural strength compared to the control specimen. Further increase of FA found to decrease the flexural strength of concrete gradually.

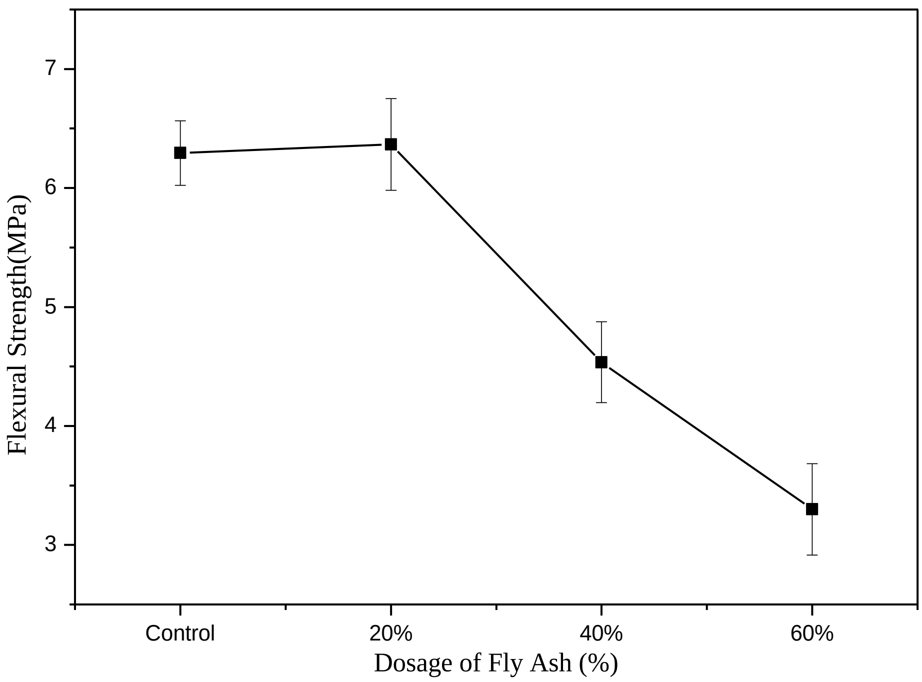


Figure 4.15: Effect of FA on flexural strength development of concretes

4.4.2 Correlation between Different Properties of FA Concrete

An attempt is made to correlate the tensile splitting strength and flexural strength of FA concrete with their compressive strength. The pair of values obtained for the tensile splitting strength and compressive strength from 45 data-set (three concrete mixes with 15 numbers of samples each) is shown in Fig. 4.16. A similar plot for flexural strength and compressive strength is presented in Fig. 4.17. The data are analysed statistically, and best fitted empirical relationships are developed between these quantities by performing a regression analysis as expressed in Eqs. 4.4-4.5:

$$f_{sp} = a + b\left(f_{cc}\right)^{c} \tag{4.4}$$

$$f_{fl} = d\left(f_{cc}\right)^{e} \tag{4.5}$$

Where f_{sp}, f_{fl} and f_{cc} denote tensile splitting, flexural and compressive strengths of FA concrete in MPa, respectively. The values obtained for the constants (*a*, *b*, *c* and *d*) along with R^2 value from the regression analysis are presented in Table 4.5.

Table 4.6: Values of constants

Investigation	*a*	B	*c*	R^2(Eq. 1)	*d*	*e*	R^2(Eq. 2)
Present Study	1.38	3.47	5.52	0.94	0.33	0.82	0.89

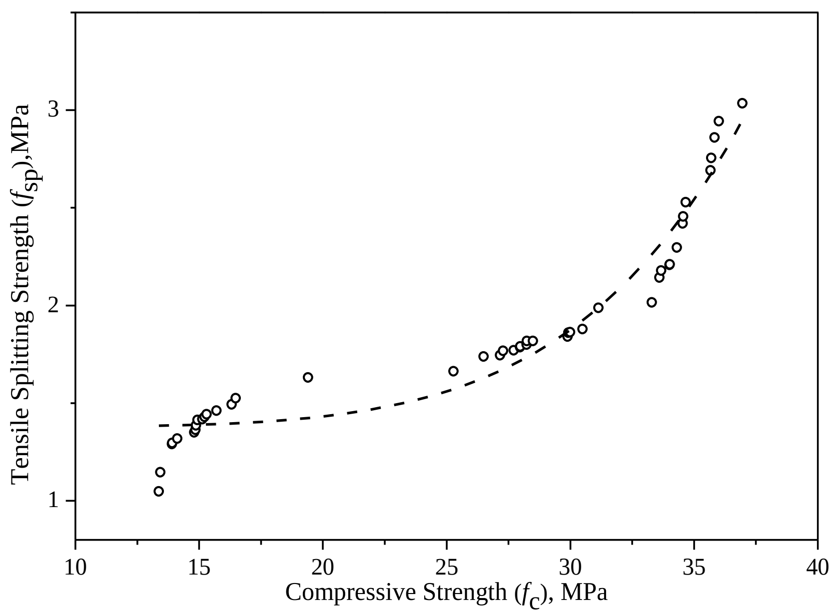


Figure 4.16: Relationship between tensile splitting strength and compressive strength

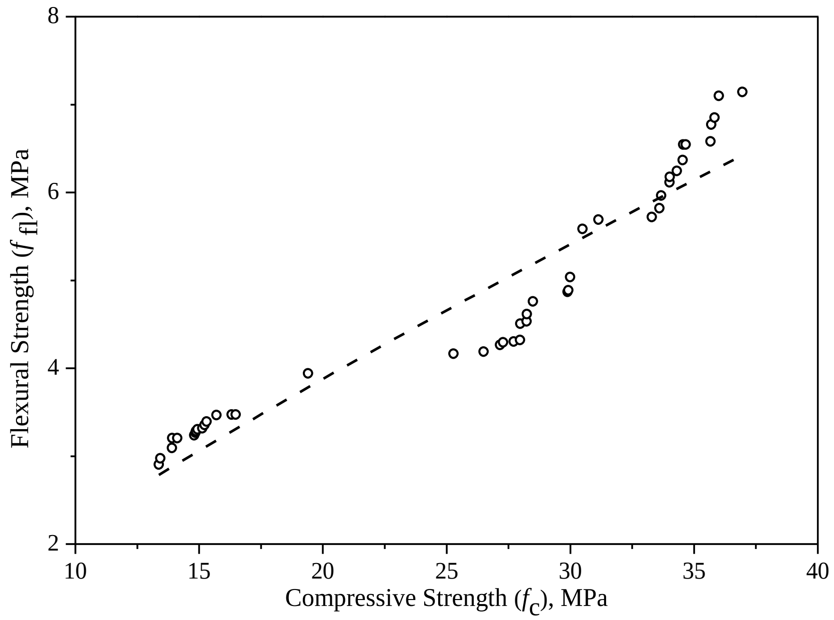


Figure 4.17: Relationship between flexural strength and compressive strength

4.5 Cost Benefit Study for SF and FA Concrete

The cost benefit analysis has been carried out using typical scheduled rate (at NIT Rourkela) as presented in Table 4.7. The cost of materials considered in the study are follows: INR 7.30 for Cement, INR 0.15 for sand, INR 0.95 for NCA and INR 28.00 for SF per kg. INR 800 is considered for the labour cost per cubic meter of concrete.

Table 4.7: Comparative cost estimate for 1 m^3 of concrete

Concrete type	Cement (kg)	Sand (kg)	NCA (kg)	SF (kg)	FA (kg)	Cost (INR)	Comp. Strength (MPa)
SF concrete (20%)	315	641	1114	78.8	-	6446	48.8
FA concrete (20%)	315	641	1114	-	78.8	4239	32.5
Control concrete	394	641	1114	-	-	4816	30.6

4.6 Conclusions

The experimental investigation is carried out to study the enhancement of properties such as compressive strength, tensile splitting strength and flexural strength of concrete made of PSC using SF and FA. The salient conclusions of the present study are as follows:

1. General behaviour of medium grade PSC concrete partially replaced with SF is found to be similar to that of Portland cement concrete as reported in

previous literature. Incorporation of SF is found to be increasing the compressive, tensile splitting and flexural strength of concrete made of PSC.

2. The compressive strength of concrete increases gradually from a SF dosage of 5%, to reach an optimum value at 20%. The percentage of increment in strength is observed to be about 29%, 40%, 59% and 44%, for the corresponding SF dosages of 10%, 15%, 20% and 25% respectively. A cap of about 21% in comparison with previous studies is found at 20% replacement. This cap can be attributed to the use of 10% extra cement.
3. The splitting strength of concrete cylinders increases gradually with the SF dosages to reach an optimum value at 20%. When compared with previous studies, it is found that the increment of strength is lower at lower dosages of SF but it is comparable at higher dosages of SF.
4. The flexural strength of concrete prism increases gradually with increase of SF dosage, to reach an optimum value at 25%. The increase in flexural strengths is about 4.45%, 5%, 12%, 33% and 18% at the SF replacements of 10%, 15%, 20%, 25 and 30% respectively. A cap of about 10% in comparison with previous studies is observed at 20% replacement.
5. Best fitted empirical relationships are developed for representation of the properties such as flexural strength and tensile splitting strength in terms of compressive strength for PSC SF concrete. The average ratio between the flexural and tensile splitting strengths of SF concrete is found to be slightly more than ACI recommended values.
6. A single equation is proposed to express compressive strengths in terms of UPV and percentage replacement of SF for PSC SF medium grade concrete. This equation is found to be fairly matching the experimental values for different concrete mixes.
7. Up to 20% of FA replacement in concrete made of PSC shows increase in compressive strength of concrete with no degradation of its tensile splitting and flexural strength. But at 40% and 60% replacement yields inferior quality of concrete.

Chapter 5

Probabilistic Models for Silica Fume and Fly Ash Concrete

5.1 Introduction

Uncertainties are inherent in structural design. The loading on the structure and the capacities of structural members are not deterministic quantities [Nowak and Collins 2000]. Accordingly, probability-based or reliability-based designs for different types of constructions have been developed. Application of these methods requires the probabilistic models of loads and resistances [Nowak and Collins 2000]. Although strength and load parameters are non-deterministic, they exhibit interesting statistical regularity which is the basis of the probabilistic models of load and resistance [Ellingwood *et al.* 1980]. Establishment of such models usually requires experimental study of load and resistance regularities in common structural types such as concrete, steel, and masonry. In structural engineering practice where construction technologies are occasionally mixed, it is usually accepted that structural load requirements are independent of construction technology to facilitate design with different construction materials [Ellingwood *et al.* 1982]. On the contrary, the resistance variation for each material, structural member, or failure mode should be individually and carefully studied. Although much work has been done on resistance models of normal concrete and steel elements, such studies in the concrete made with partial replacement of cement by industrial waste materials such as SF and FA are not available. The fluctuating nature of the mechanical properties (compressive strength, flexural strength and tensile splitting strength) can have a significant impact on the performance of structures. Extensive experimental work is carried out to find the fluctuating nature of SF and FA concrete. The collected statistical data were then used to derive the variable features of the selected mechanical properties. This included the mean value, variance, skewness, suitable distribution functions, and probabilistic models. In addition to the uncertainties in the properties of constituents of normal concrete, quality and variations in the properties of

SF and FA may affect the mechanical properties of the concrete made using these materials. This chapter discusses the variability of mechanical properties of SF and FA concrete and its implications in the performance of typical buildings.

5.2 Variability in SF Concrete

In order to determine the variability in the selected mechanical properties of SF concrete, results of compressive strengths, tensile splitting strengths and flexural strengths from the experimental programme conducted in Chapter 4 are considered. It consists of seven sets of concrete mixes with partial replacement of cement with SF as discussed earlier. Details of the mixture proportions used in this study are the same as the tests explained in Chapter 4.

This section further deals with on the representation of variability of above properties of SF concrete using different probability distribution models. Values of all the mechanical properties obtained experimentally are converted to probability distributions. Selected standard probability distribution models are assumed and the models are verified using standard goodness-of-fit tests. Details of selected two parameter probability distribution models and selected goodness-of-fit tests are explained in Appendix B briefly.

The selected two parameter standard probability distribution models in this study are; truncated normal, lognormal, gamma and Weibull distributions. The best-fit probability distribution model is captured by performing certain statistical goodness-of-fit tests such as modified Kolmogorov-Smirnov (KS), Log-likelihood (LK) and minimum Chi-square criterion (CS) at 5% significance level. The probability distribution which has minimum values of KS distance and CS value, and maximum value of LK is considered as the best fit. These methods have been successfully used in many past literatures [Chen *et al.* 2014, and Stone *et al.* 1986]. A total of 490 experimental values of compressive strength (210), tensile splitting strength (140) and flexural strength (140) with different SF content are used for the representation of probability distribution.

The distribution is rejected if the goodness-of-fit test values are below the critical value specified at 5% significance level. The values obtained for the rejected distributions are omitted in the presentation of results. The selection criteria for a best fit distribution are the minimum values of KS distance and CS along with maximum value of LK. The CS value may not be always reliable [Chen *et al.* 2014] because it depends on binning of data into intervals and it is best suitable when large random variables are used. Therefore in this study the best fitted distribution is decided mostly from KS distance and LK value even if CS value is not minimum.

5.2.1 Compressive Strength

Compressive strengths of concrete cubes (7 mixes × 30 samples each = 210 total samples) with various SF contents are presented in Table 5.1. It can be seen that the compressive strength of control specimen is found to be varying from 24.18 MPa to 34.60 MPa with a mean and standard deviation (SD) of 30.27 MPa and 2.17 MPa. Similarly, the minimum, maximum, mean and SD for other concrete specimens having different percentage of SF is shown in this table. The mean compressive strength of concrete increases with SF content and it reaches maximum value (53.97 MPa) at 20% SF content. The table also shows that the SD of compressive strength increases with the increase in SF content. This may be attributed to the high inherent variability in the properties of SF. Mean compressive strengths of concrete obtained for each SF dosage are plotted in Fig. 5.1.

Table 5.2 shows the estimated shape and scale parameters of distributions, KS distances, LK and CS values for compressive strength. This table shows that all three criteria (KS, CS and LK) are not in agreement with a single distribution for variability description of compressive strength. However, there are very little deviations among the goodness-of-fit test values for all cases of mix proportions. A single distribution meets all the selection criteria of minimum KS distance, minimum CS and maximum LK value simultaneously for mix with 10%, 15% and 20% SF replacement. Hence, lognormal and Weibull distribution are found to be the best fit models for mix with 10%, 15% and 20% SF respectively. However, for mix with 0%, 5%, 25% and 30% SF, no single distribution meets all the selecting criteria but the values of all distributions are close to each other. Therefore, based on KS distance and LK value, either Weibull or lognormal is found to be the closest fit model for these concrete mixes. The probability distributions obtained from experiments and the assumed cumulative probability distribution models for compressive strength of SF concrete for different mix proportions of SF are shown in Fig. 5.2 for graphical representation.

Table 5.1: Compressive strength of SF concrete (in MPa)

Sl. No.	Control	5% SF	10% SF	15% SF	20% SF	25% SF	30% SF
1	24.18	18.73	37.46	37.75	43.71	41.26	29.94
2	26.23	25.42	39.56	38.77	43.74	41.41	30.54
3	26.66	25.78	39.68	39.39	44.37	41.52	34.07
4	26.76	26.29	39.98	39.48	44.78	42.34	34.15
5	27.26	26.55	40.21	41.08	45.23	42.57	34.22
6	27.41	26.57	40.43	41.29	46.81	43.18	37.84
7	27.81	26.68	40.50	42.70	46.84	44.21	39.38
8	28.54	27.34	40.51	43.44	48.23	44.26	40.50
9	28.64	29.11	40.72	43.52	48.28	44.47	40.71
10	28.84	29.18	41.29	43.89	48.83	46.01	41.19
11	29.21	29.23	41.71	44.24	51.78	46.03	41.19
12	29.72	29.55	41.74	44.32	52.51	46.13	44.27
13	29.83	29.64	41.86	44.38	54.47	46.18	44.50
14	30.22	30.02	42.15	44.49	55.88	46.24	44.61
15	30.51	30.46	42.81	44.56	56.77	46.77	45.81
16	30.98	31.59	43.65	45.27	57.51	48.16	46.65
17	31.17	32.14	43.88	45.82	57.59	49.68	46.71
18	31.34	32.53	44.55	47.30	58.17	50.12	47.05
19	31.52	32.91	45.16	50.15	58.25	50.34	48.10
20	31.56	33.15	45.40	50.35	58.28	50.66	48.11
21	31.82	33.30	46.80	51.76	58.34	51.59	48.26
22	32.57	33.44	47.17	52.64	58.38	53.12	49.18
23	32.81	33.45	47.36	53.61	58.41	53.16	49.78
24	32.81	33.64	47.53	53.64	58.69	53.28	50.74
25	32.85	34.55	47.93	54.40	58.86	54.35	50.75
26	33.28	34.63	48.98	54.83	58.97	56.06	52.33
27	33.57	35.25	49.17	55.27	59.89	56.13	53.65
28	34.11	35.46	49.72	56.29	60.54	58.96	57.86
29	34.29	37.43	50.14	57.18	62.12	61.17	58.92
30	34.60	38.01	50.98	60.82	62.86	62.52	62.30
Mean	30.37	30.73	43.97	47.42	53.97	49.06	45.11
SD	2.71	4.17	3.79	6.29	6.18	5.96	8.03

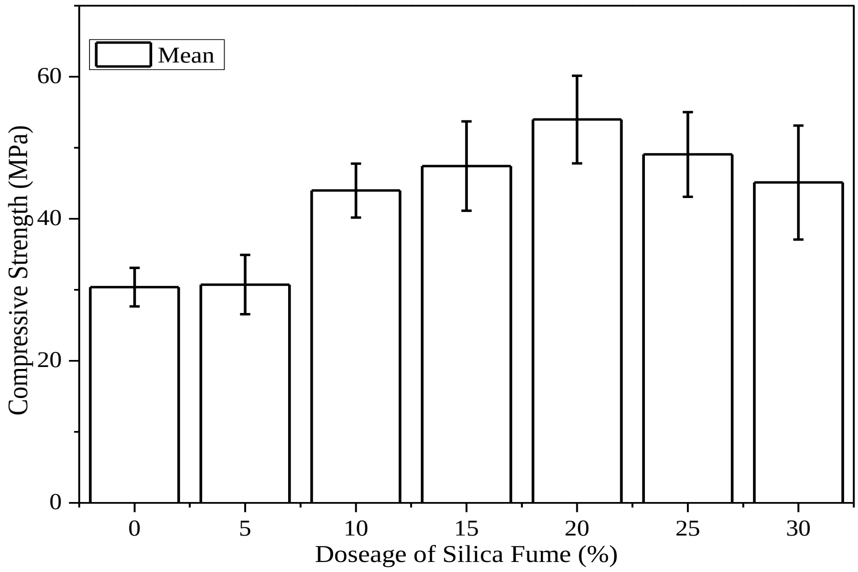
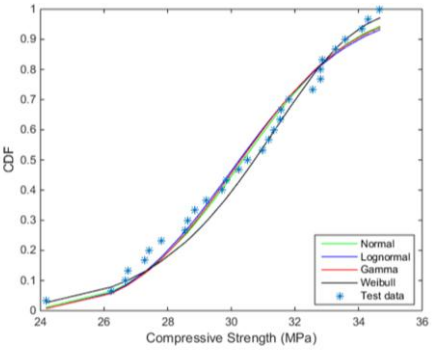


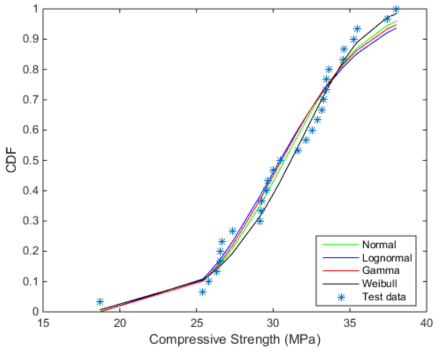
Figure 5.1: Variation of Mean, SD of compressive strength of SF concrete

Table 5.2: Estimated parameters, KS distances, LK, and CS for different distribution functions describing compressive strength of SF concrete

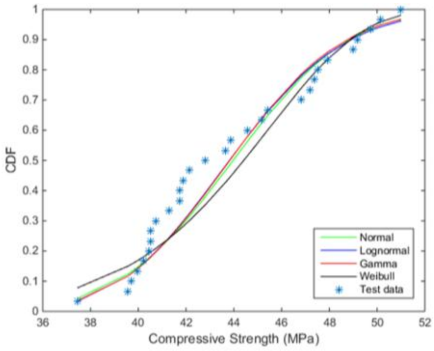
Mix	Distribution	Shape	Scale	KS	CS	LK
Control	**Weibull**	13.49	31.57	**0.077**	0.648	**-71.49**
	Gamma	125.85	0.2414	0.073	0.88	-72.37
	Normal	30.376	2.717	0.062	0.713	-72.05
	Lognormal	0.091	30.23	0.072	0.76	-72.58
5% SF	**Weibull**	8.96	32.47	**0.076**	1.83	**-84.04**
	Gamma	51.31	0.59	0.077	1.70	-86.06
	Normal	30.738	4.179	0.067	1.77	-84.97
	Lognormal	0.146	30.416	0.082	1.36	-86.81
10% SF	Weibull	12.56	45.73	0.164	2.29	-83.58
	Gamma	140.35	0.313	0.147	0.90	-81.84
	Normal	43.97	3.797	0.151	0.97	-82.09
	Lognormal	0.085	43.816	**0.141**	**0.70**	**-81.75**
15% SF	Weibull	8.136	50.211	0.188	3.22	-98.61
	Gamma	59.706	0.794	0.167	0.99	-96.83
	Normal	47.425	6.298	0.175	1.49	-97.27
	Lognormal	0.131	46.993	**0.157**	**0.79**	**-96.70**
20% SF	**Weibull**	11.181	66.387	**0.179**	**2.27**	**-95.01**
	Gamma	77.657	0.692	0.204	4.06	-96.69
	Normal	53.773	6.091	0.183	3.07	-96.27
	Lognormal	0.116	53.410	0.192	3.75	-96.97
25% SF	Weibull	8.418	51.76	0.386	2.72	-97.92
	Gamma	72.55	0.676	0.412	0.41	-94.96
	Normal	49.067	5.963	0.406	0.79	-95.63
	Lognormal	0.118	48.715	0.409	**0.33**	**-94.70**
30% SF	**Weibull**	6.29	48.425	0.448	**0.84**	**-104.99**
	Gamma	31.425	1.435	0.446	1.88	-104.80
	Normal	45.114	8.039	0.444	1.44	-104.59
	Lognormal	0.1843	44.389	0.433	1.92	-105.13



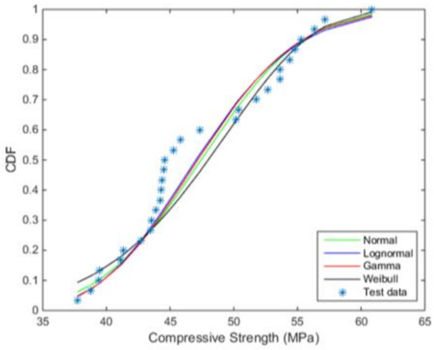
(a) Control



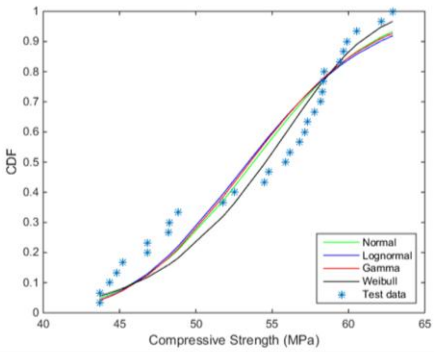
(b) 5% SF



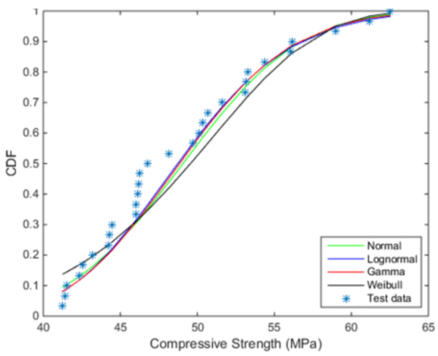
(c) 10% SF



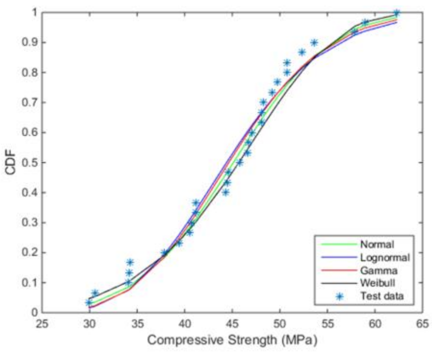
(d) 15% SF



(e) 20% SF



(f) 25% SF



(g) 30% SF

Figure 5.2: Experimental and assumed cumulative probability distributions for compressive strength of SF concrete

5.2.2 Flexural Strength

Flexural strength of concrete prisms (7 mixes × 20 samples each = 140 total samples) with various SF content is presented in Table 5.3. The flexural strength of control specimen is varying from 5.94 MPa to 6.41 MPa with a mean and standard deviation (SD) of 6.32 MPa and 0.33 MPa. Similarly, the minimum, maximum, mean and SD for other concrete specimens having different percentage of SF is presented in Table 5.3. The mean flexural strength of concrete increases with SF content and it reaches maximum value (8.35 MPa) at 25% SF content. It can be seen from this table that the SD values of concrete follows non uniform trend with the increase in SF content. Mean flexural strength of concrete obtained for each SF dosage is plotted in Fig. 5.3.

Table 5.3: Flexural strength (MPa) of SF concrete

Sl No.	Control	5% SF	10% SF	15% SF	20% SF	25% SF	30% SF
1	5.94	7.01	6.86	6.32	7.12	7.87	7.15
2	6.12	5.94	7.01	6.51	7.25	8.81	7.62
3	6.47	5.98	6.54	6.48	7.23	8.75	7.13
4	6.81	4.10	6.31	6.92	6.71	7.93	7.73
5	6.06	5.73	5.88	6.56	6.58	8.43	7.94
6	6.65	5.87	6.23	6.14	6.56	8.15	7.50
7	6.16	7.42	6.27	6.78	7.16	8.21	7.66
8	5.96	6.58	6.65	6.24	7.23	8.75	7.40
9	6.82	6.41	5.75	6.05	7.49	8.50	7.31
10	6.52	6.54	7.09	6.87	6.85	9.10	7.89
11	5.63	6.53	7.24	6.82	7.41	7.36	7.23
12	6.47	6.11	5.66	6.80	7.33	8.81	7.08
13	6.15	7.14	7.41	6.59	7.25	8.75	7.15
14	6.70	5.82	6.90	6.81	7.51	8.43	7.62
15	5.87	6.17	6.72	6.99	7.30	8.15	7.66
16	6.65	7.03	6.25	6.78	7.11	7.36	7.40
17	6.19	6.75	6.94	6.27	6.46	8.81	7.31
18	6.50	6.69	7.05	7.02	7.15	8.75	7.50
19	6.34	7.44	6.28	6.88	7.41	7.93	7.73
20	6.41	6.63	6.99	6.51	6.96	8.15	7.13
Mean	6.32	6.39	6.60	6.62	7.10	8.35	7.46
SD	0.33	0.74	0.50	0.29	0.31	0.48	0.26

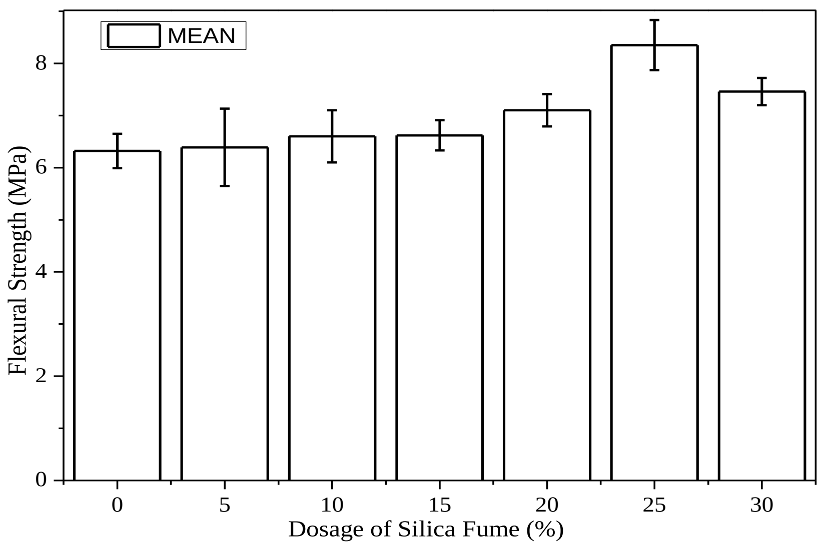
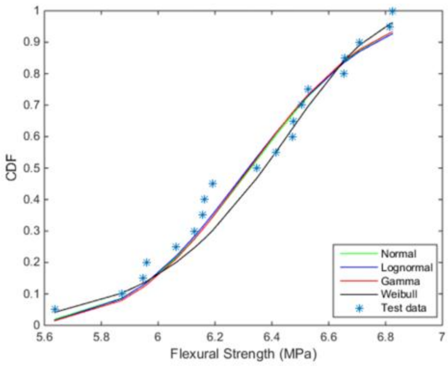


Figure 5.3: Variation of Mean, SD of flexural strength of SF concrete

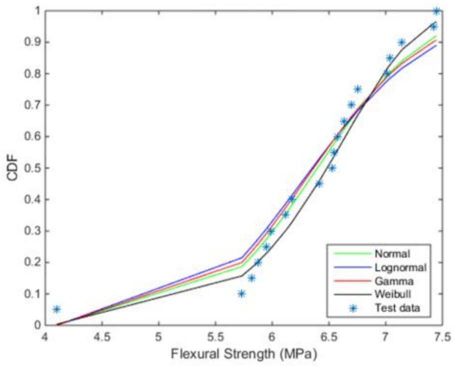
The estimated shape and scale parameters of distributions, KS distances, LK and CS values for flexural strength of different SF concrete mix are presented in Table 5.4. The goodness-of-fit test values for all cases possess very little deviation among the selected distributions. For mix with 5%, 15% and 20% SF, Weibull distribution meets all the selection criteria. Similarly, for mix with 30% SF, lognormal distribution meets all the selection criteria. However, no single distribution meets all the selecting criteria for mix with 0%, 10% and 25% SF. Hence on the basis of KS and LK value, either a lognormal or Weibull distribution can be considered as the best fit distribution for these mixes. The variation of the cumulative probability distribution of flexural strength with assumed models for different mix proportions is shown in Fig. 5.4.

Table 5.4: Estimated parameters, KS distances, LK, and CS for different distribution functions describing flexural strength of SF concrete

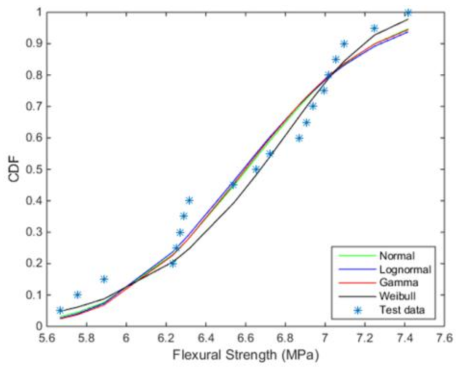
Mix	Distribution	Shape	Scale	KS	CS	LK
Control	Weibull	22.707	6.477	0.142	-	-5.761
	Gamma	374.401	0.016	0.105	-	-6.001
	Normal	6.326	0.333	0.106	-	-5.919
	Lognormal	0.0532	6.296	**0.097**	-	-6.071
5% SF	**Weibull**	11.376	6.695	**0.080**	**-**	**-20.344**
	Gamma	67.395	0.094	0.121	-	-23.293
	Normal	6.398	0.074	0.086	-	-22.093
	Lognormal	0.129	6.347	0.114	-	-24.023
10% SF	Weibull	16.135	6.827	0.152	-	-13.641
	Gamma	178.158	0.037	0.117	-	-14.274
	Normal	6.606	0.502	0.118	-	-14.109
	Lognormal	0.077	6.586	**0.106**	-	-14.394
15% SF	**Weibull**	28.821	6.757	**0.118**	**-**	**-2.436**
	Gamma	529.181	0.012	0.137	-	-3.463
	Normal	6.621	0.293	0.155	-	-3.343
	Lognormal	0.044	6.612	0.159	-	-3.540
20% SF	**Weibull**	30.797	7.239	**0.107**	**-**	**-2.949**
	Gamma	520.572	0.013	0.526	-	-5.100
	Normal	7.107	0.315	0.155	-	-4.824
	Lognormal	0.045	7.113	0.163	-	-5.171
25% SF	**Weibull**	21.944	8.567	**0.147**	**-**	**-12.472**
	Gamma	300.758	0.027	0.529	-	-13.749
	Normal	8.353	0.488	0.143	-	-13.530
	Lognormal	0.059	8.39	0.143	-	-13.886
30% SF	Weibull	30.725	7.587	0.123	-	-2.405
	Gamma	829.575	0.009	0.135	-	-1.358
	Normal	7.46	0.266	0.123	-	-1.402
	Lognormal	0.0356	7.455	0.125	-	**-1.351**



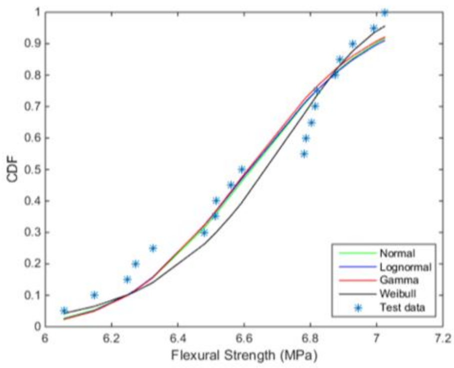
(a) Control



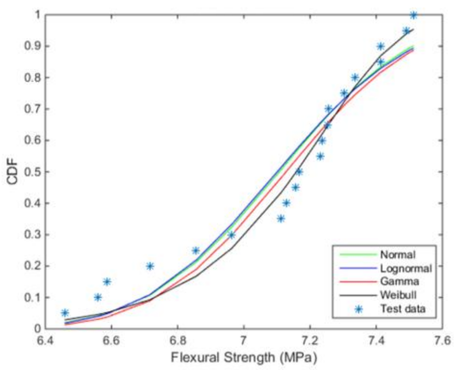
(b) 5% SF



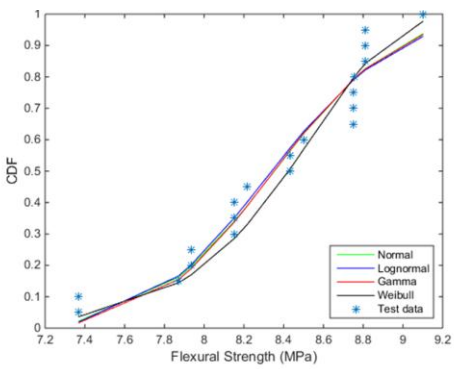
(c) 10% SF



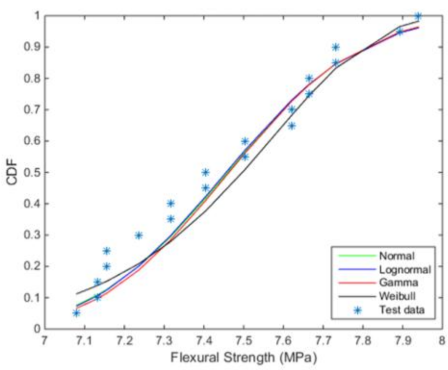
(d) 15% SF



(e) 20% SF



(f) 25% SF



(f) 30% SF

Figure 5.4: Experimental and assumed cumulative probability distributions for flexural strength of SF concrete

5.2.3 Tensile Splitting Strength

Tensile splitting strength of concrete cylinder (7 mixes × 20 samples each = 140 total samples) with various SF content is presented in Table 5.5. The tensile splitting strength of control specimen is varying from 2.21 MPa to 2.96 MPa with a mean and SD of 2.60 MPa and 0.23 MPa respectively. Similarly, the minimum, maximum, mean and SD for other concrete specimens having different percentage of SF is shown in the Table 5.5. The mean tensile splitting strength of concrete increases with SF content and it reaches maximum value of 3.91 MPa at 20% SF replacement. This table shows that he SD of tensile splitting strength increases with the increase in SF content. Mean tensile splitting strength of concrete obtained for each SF dosage is plotted in Fig. 5.5.

The estimated shape and scale parameters of distributions, KS distances, LK and CS values for tensile strength of different mixes of SF concrete are presented in Table 5.6. For mix with 0%, 15% and 30% SF replacement, a single distribution meets all the selection criteria. Hence, lognormal, Gamma and Gamma distributions are found to be the best fit models for mix with 0%, 15% and 30% SF respectively. No single distribution meets all the selection criteria for mix with 5%, 10%, 20% and 25% SF replacement. The probability distributions obtained from experiments and the assumed cumulative probability distribution models for tensile strength of SF concrete for different mix proportions of SF are shown in Fig. 5.6 graphically.

Table 5.5: Tensile splitting strength (MPa) of SF concrete

Sl. No.	Control	5% SF	10% SF	15% SF	20% SF	25% SF	30% SF
1	2.21	2.51	2.50	2.85	3.44	3.14	3.17
2	2.69	2.57	2.60	2.95	3.48	3.17	3.20
3	2.64	2.66	2.67	2.99	3.52	3.24	3.24
4	2.71	2.75	2.73	3.17	3.54	3.28	3.27
5	2.67	2.81	2.79	3.20	3.57	3.36	3.31
6	2.57	2.85	2.83	3.22	3.61	3.53	3.48
7	2.46	2.88	2.86	3.27	3.71	3.57	3.52
8	2.54	2.91	2.90	3.28	3.74	3.69	3.55
9	2.67	2.93	2.94	3.48	3.76	3.78	3.58
10	2.26	2.95	2.97	3.51	3.79	3.81	3.83
11	2.61	2.96	3.02	3.53	4.06	3.86	3.86
12	2.29	2.97	3.05	3.59	4.10	3.92	3.89
13	2.73	3.02	3.08	3.83	4.14	3.96	3.96
14	2.85	3.08	3.10	3.90	4.17	4.03	4.11
15	2.18	3.17	3.17	3.92	4.18	4.12	4.14
16	2.81	3.27	3.20	4.10	4.21	4.15	4.26
17	2.35	3.39	3.32	4.15	4.24	4.19	4.33
18	2.88	3.51	3.48	4.22	4.27	4.20	4.39
19	2.90	3.55	3.53	4.24	4.30	4.21	4.44
20	2.96	3.60	3.62	4.33	4.33	4.32	4.51
Mean	2.60	3.02	3.02	3.59	3.91	3.78	3.80
SD	0.23	0.31	0.30	0.46	0.31	0.38	0.45

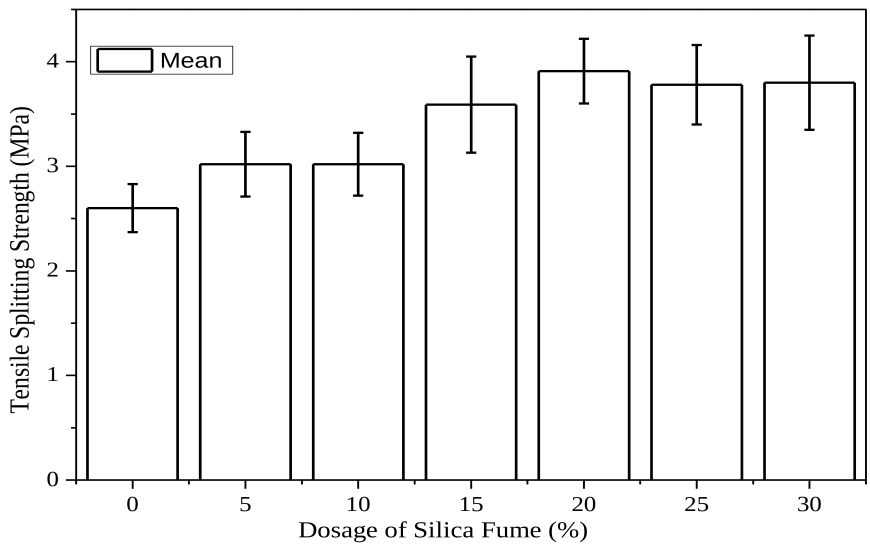
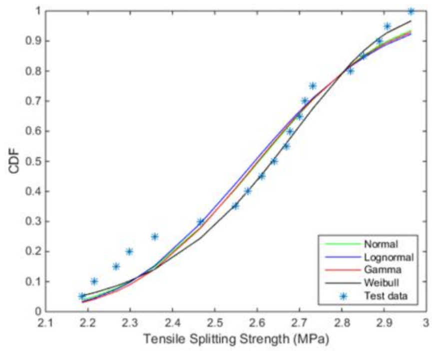


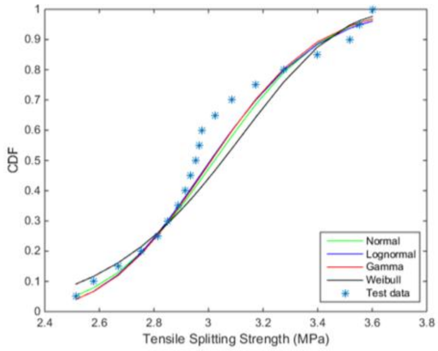
Figure 5.5: Variation of Mean, SD of tensile splitting strength of SF concrete

Table 5.6: Estimated parameters, KS distances, LK, and CS for different distribution functions describing tensile splitting strength of SF concrete

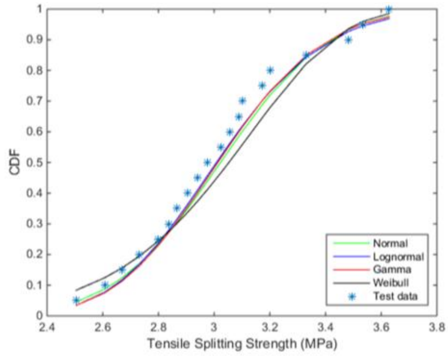
Mix	Distribution	Shape	Scale	KS	CS	LK
Control	Weibull	13.597	2.707	0.107	-	-1.457
	Gamma	123.064	0.212	0.110	-	-0.653
	Normal	2.604	0.237	0.102	-	-0.873
	Lognormal	0.0933	2.585	**0.102**	**-**	**-0.503**
5% SF	Weibull	10.213	10.213	0.186	-	-5.969
	Gamma	99.679	0.03	**0.106**	-	-4.525
	Normal	3.302	0.313	0.159	-	-4.675
	Lognormal	0.1025	3.007	0.141	-	-4.336
10% SF	Weibull	10.467	3.161	0.14	-	-5.465
	Gamma	103.881	0.029	0.07	-	-4.006
	Normal	3.022	0.306	0.09	-	-4.217
	Lognormal	0.1005	3.007	0.08	-	**-3.957**
15% SF	Weibull	8.773	3.795	0.154	-	-13.118
	Gamma	61.461	0.058	**0.122**	**-**	**-12.656**
	Normal	3.591	0469	0.143	-	-49.661
	Lognormal	0.1312	3.560	0.131	-	-12.668
20% SF	**Weibull**	15.074	4.055	**1.588**	**-**	**-4.683**
	Gamma	158.572	0.0247	1.626	-	-4.968
	Normal	3.913	0.317	1.56	-	-4.929
	Lognormal	0.081	3.900	1.557	-	-5.020
25% SF	**Weibull**	12.305	3.949	**0.12**	**-**	**-8.182**
	Gamma	98.141	0.038	0.12	-	-9.053
	Normal	3.782	0.386	0.11	-	-8.842
	Lognormal	0.104	3.762	0.11	-	-9.206
30% SF	Weibull	9.739	4.005	0.159	-	-12.301
	Gamma	74.27	0.057	**0.106**	**-**	**-11.951**
	Normal	3.806	0.452	0.135	-	-12.023
	Lognormal	0.119	3.781	0.119	-	-11.971



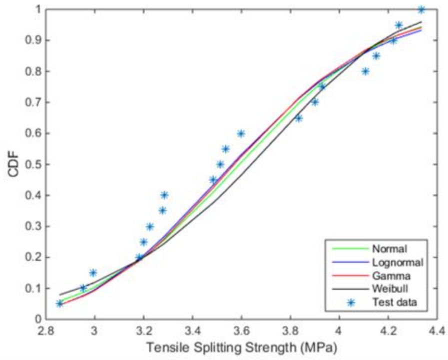
(a) Control



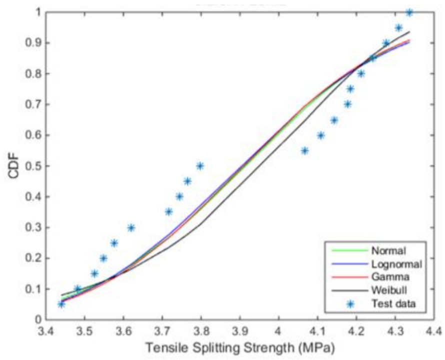
(b) 5% SF



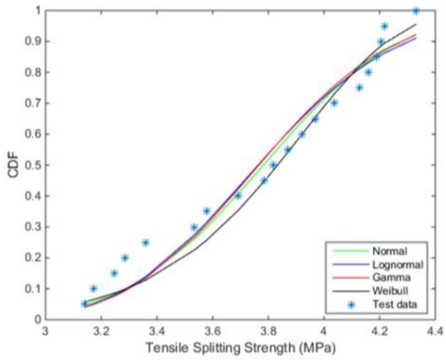
(c) 10% SF



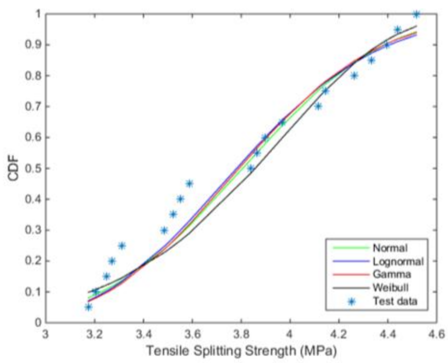
(d) 15% SF



(e) 20% SF



(f) 25% SF



(g) 30% SF

Figure 5.6: Experimental and assumed cumulative probability distributions for tensile splitting strength of SF concrete

5.3 Variability of FA Concrete

This section discusses the test results of compressive strength, flexural strength, and tensile splitting strength of FA concrete and the similar goodness of tests for the description of the variability in their values. A total of 270 experimental values of compressive strength (90 nos.), tensile splitting strength (90 nos.) and flexural strength (90 nos.) with different FA content are used for the representation of probability distribution. The same statistical goodness of fit test have been used for the development of probabilistic distribution for FA concrete as discussed in previous section is considered.

5.3.1 Compressive Strength

Compressive strength of concrete cubes (3 mixes × 30 samples each = 90 total samples) with various FA content is presented in Table 5.7. The compressive strength of control specimen is found to be varying from 24.18 MPa to 34.60 MPa with a mean and SD of 30.27 MPa and 2.17 MPa. Similarly, the minimum, maximum, mean and SD for other concrete specimens having different percentage of FA is shown in this table. The mean compressive strength of concrete increases with FA content and it reaches maximum value (34.63 MPa) at 20% FA content. The table also shows that the SD of compressive strength decreases with the increase in FA content. This may be attributed to the high inherent variability in the properties of FA. Mean compressive strength of concrete obtained for each FA dosage is plotted in Fig. 5.7.

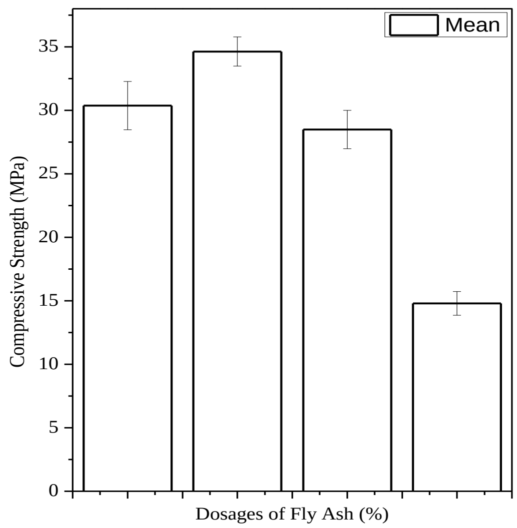


Figure 5.7: Variation of Mean, SD of compressive strength of FA concrete

Table 5.7: Compressive strength of FA concrete (in MPa)

Sl. No.	Control	20% FA	40% FA	60% FA
1	24.18	32.28	25.4	13.37
2	26.23	32.88	26.4	13.37
3	26.66	33.29	26.5	13.43
4	26.76	33.29	26.5	13.43
5	27.26	33.58	27.16	13.75
6	27.41	33.6	27.16	13.9
7	27.81	33.67	27.28	13.91
8	28.54	33.67	27.28	13.97
9	28.64	34.01	27.71	14.12
10	28.84	34.01	27.71	14.12
11	29.21	34.02	27.81	14.25
12	29.72	34.3	27.97	14.79
13	29.83	34.31	27.98	14.8
14	30.22	34.43	27.98	14.8
15	30.51	34.45	28.21	14.86
16	30.98	34.54	28.24	14.86
17	31.17	34.56	28.25	14.88
18	31.34	34.65	28.49	14.94
19	31.52	34.66	28.74	15.13
20	31.56	34.66	28.85	15.13
21	31.82	35.67	29.89	15.22
22	32.57	35.67	29.89	15.22
23	32.81	35.7	29.92	15.31
24	32.81	35.7	29.97	15.31
25	32.85	35.83	29.99	15.7
26	33.28	35.83	29.99	15.74
27	33.57	36	30.45	16.25
28	34.11	36.25	30.49	16.32
29	34.29	36.51	31.14	16.48
30	34.60	36.95	31.14	16.48
Mean	30.37	34.63	28.48	14.79
SD	2.71	1.150	1.505	0.934

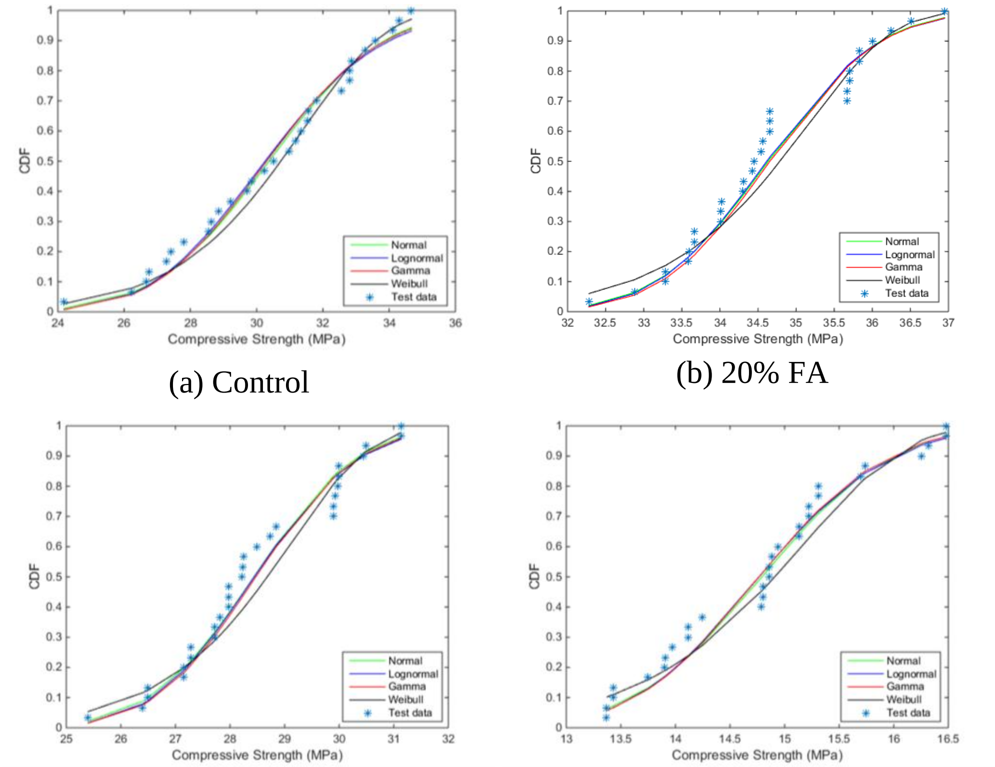
Table 5.8 shows the estimated shape and scale parameters of distributions, KS distances, LK and CS values for compressive strength of FA concrete. This table shows that all three criteria (KS, CS and LK) are not in agreement with a single distribution for variability description of compressive strength except in case of 40% FA concrete. However, there are very little deviations among the goodness-of-fit test values for all the cases of mix proportions. A single distribution meets all the selecting criteria of minimum KS distance, minimum CS and maximum LK value for mix with 40%, FA replacement only. However, lognormal is found to be the best fit models for other mix with 20% and 60% FA respectively. The probability distributions obtained from experiments and the assumed cumulative probability distribution models for compressive strength of FA concrete for different mix proportions of FA are shown in Fig. 5.8.

Table 5.8: Estimated parameters, KS distances, LK, and CS for different distribution functions describing compressive strength of FA concrete

Mix	Distribution	Shape	Scale	KS	CS	LK
20% FA	Weibull	32.254	35.186	0.207	5.367	-47.992
	Gamma	939.47	0.0369	0.164	4.597	-46.235
	Normal	34.632	1.150	0.157	4.741	-46.269
	Lognormal	0.033	3.544	**0.150**	4.675	**-46.216**
40% FA	Weibull	20.827	29.192	0.170	3.882	-55.556
	Gamma	370.59	0.076	0.126	4.161	-54.295
	Normal	28.483	1.505	0.128	4.004	-54.349
	Lognormal	0.052	3.347	**0.125**	**3.880**	**-54.30**
60% FA	Weibull	15.232	17.111	0.135	0.719	-41.527
	Gamma	259.88	0.056	0.109	1.102	-39.955
	Normal	14.794	0.934	0.098	0.977	-40.043
	Lognormal	0.063	2.692	**0.110**	1.011	**-39.942**

5.3.2 Flexural Strength

Flexural strength of concrete prisms (3 mixes × 30 samples each = 90 total samples) with various FA content is presented in Table 5.9. The flexural strength of control specimen is varying from 5.94 MPa to 6.41 MPa with a mean and SD of 6.32 MPa and 0.33 MPa. Similarly, the minimum, maximum, mean and SD for other concrete specimens having different percentage of FA is presented in Table 5.8. The mean flexural strength of concrete increases with FA content and it reaches maximum value (8.35 MPa) at 20% FA content. It can be seen from this table that the SD values of concrete follows non uniform trend with the increase in FA content. Mean flexural strength of concrete obtained for each FA dosage is plotted in Fig. 5.9.



(a) Control (b) 20% FA

(c) 40% FA (d) 60% FA

Figure 5.8: Experimental and assumed cumulative probability distributions for compressive strength of FA concrete

The estimated shape and scale parameters of distributions, KS distances, LK and CS values for flexural strength of different FA concrete mix are presented in Table 5.10 The goodness-of-fit test values for all cases possess very little deviation among the selected distributions. For mix with 20% FA, Weibull distribution meets the KS and CS selecting criteria. Similarly, for mix with 40% FA, lognormal distribution meets the KS and LK selecting criteria. For 60% FA gamma distribution is suitable. However, no single distribution meets all the selecting criteria for all the mix of FA concrete. The variation of the cumulative probability distribution of flexural strength with assumed models for different mix proportions is shown in Fig. 5.10.

Table 5.9: Flexural strength (MPa) of FA concrete

Sl No.	Control	20% FA	40% FA	60% FA
1	5.94	5.51	3.94	2.90
2	6.12	5.58	3.94	2.97
3	6.47	5.69	4.16	3.03
4	6.81	5.69	4.16	3.03
5	6.06	5.82	4.18	3.09
6	6.65	5.82	4.18	3.09
7	6.16	5.84	4.26	3.20
8	5.96	5.91	4.26	3.20
9	6.82	5.96	4.29	3.20
10	6.52	6.11	4.29	3.23
11	5.63	6.11	4.30	3.23
12	6.47	6.17	4.30	3.26
13	6.15	6.24	4.32	3.26
14	6.70	6.36	4.50	3.28
15	5.87	6.39	4.53	3.30
16	6.65	6.54	4.53	3.30
17	6.19	6.54	4.61	3.31
18	6.50	6.54	4.61	3.35
19	6.34	6.54	4.62	3.35
20	6.41	6.57	4.72	3.39
21	5.63	6.58	4.76	3.39
22	6.47	6.58	4.78	3.42
23	6.15	6.74	4.86	3.46
24	6.70	6.77	4.86	3.47
25	5.87	6.77	4.88	3.47
26	6.65	6.85	4.88	3.47
27	6.19	7.10	5.03	3.47
28	6.70	7.10	5.03	3.55
29	5.87	7.14	5.07	3.55
30	6.65	7.15	5.07	3.68
Mean	6.32	6.36	4.53	3.33
SD	0.33	0.48	0.34	0.18

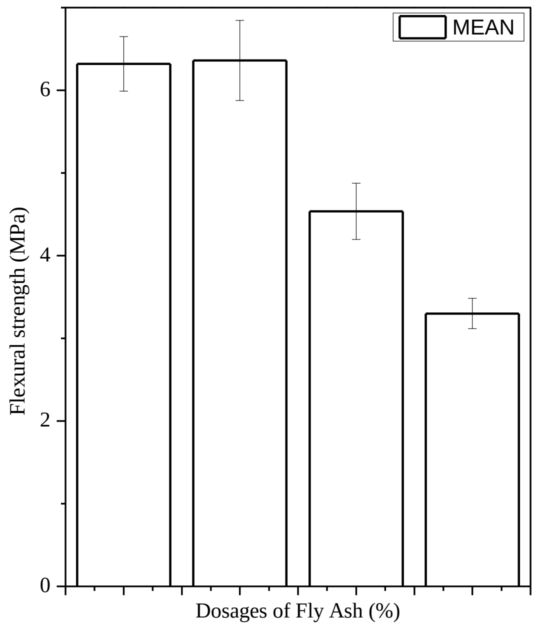


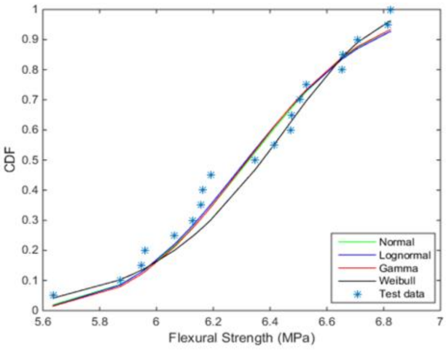
Figure 5.9: Variation of Mean, SD of flexural strength of FA concrete

Table 5.10: Estimated parameters, KS distances, LK, and CS for different distribution functions describing flexural strength of FA concrete

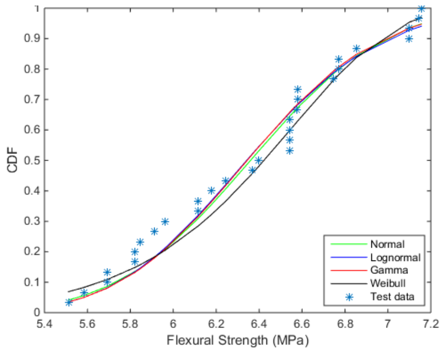
Mix	Distribution	Shape	Scale	KS	CS	LK
20% FA	**Weibull**	6.583	14.871	**0.103**	**2.818**	-21.075
	Gamma	173.66	0.0366	0.126	4.670	-20.656
	Normal	6.360	0.489	0.113	4.197	-20.617
	Lognormal	0.077	1.847	0.124	4.479	-20.714
40% FA	Weibull	4.696	15.097	0.183	0.379	-10.448
	Gamma	183.062	0.0248	0.172	1.319	-9.724
	Normal	4.535	0.340	0.169	1.030	-9.752
	Lognormal	0.075	1.509	**0.162**	1.226	**-9.743**
60% FA	Weibull	3.385	20.212	0.086	1.647	-8.163
	Gamma	326.64	0.010	**0.073**	**1.497**	-8.467
	Normal	3.300	0.184	0.071	1.444	-8.606
	Lognormal	0.056	1.192	0.075	1.632	-8.369

5.3.3 Tensile Splitting Strength

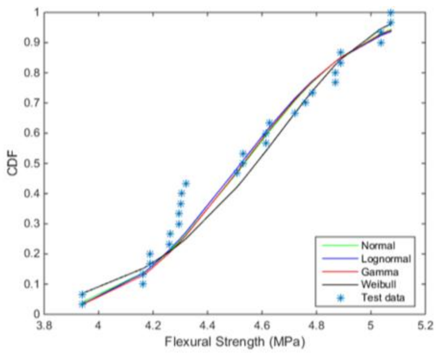
Tensile splitting strength of concrete cylinder (3 mixes × 30 samples each = 90 total samples) with various FA content is presented in Table 5.11. The tensile splitting strength of control specimen is varying from 2.21 MPa to 2.96 MPa with a mean and SD of 2.60 MPa and 0.23 MPa respectively. Similarly, the minimum, maximum, mean and SD for other concrete specimens having different percentage of FA is shown in the Table 5.9. The mean tensile splitting strength of concrete increases with FA content and it reaches maximum value of 3.91 MPa at 20% FA replacement. This table shows that the SD of tensile splitting strength increases with the increase in FA content. Mean tensile splitting strength of concrete obtained for each FA dosage is plotted in Fig. 5.11.



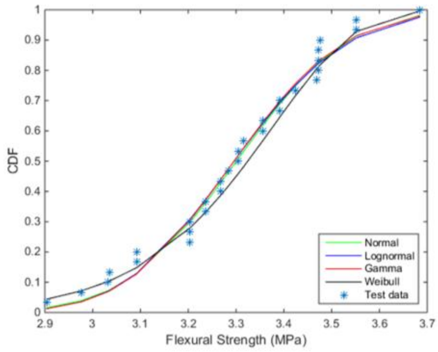
(a) Control



(b) 20% FA



(c) 40% FA



(d) 60% FA

Figure 5.10: Experimental and assumed cumulative probability distributions for flexural strength of FA concrete

The estimated shape and scale parameters of distributions, KS distances, LK and CS values for tensile strength of different mixes of FA concrete are presented in Table 5.12. Like in case of flexural strength of FA concrete, here also no single distribution meets all the selecting criteria. For mix with 20% FA, lognormal distribution meets the selecting criteria considering KS and LK. Normal and Weibull distributions are found to be the best fit models for 40% and 60% FA respectively. The probability distributions obtained from experiments and the assumed cumulative probability distribution models for tensile strength of FA concrete for different mix proportions of FA are shown in Fig. 5.12

Table 5.11: Tensile splitting strength (MPa) of FA concrete

Sl. No.	Control	20% SF	40% SF	60% SF
1	2.21	1.98	1.66	1.17
2	2.69	2.00	1.66	1.19
3	2.64	2.01	1.70	1.24
4	2.71	2.01	1.71	1.26
5	2.67	2.14	1.73	1.28
6	2.57	2.14	1.74	1.29
7	2.46	2.17	1.74	1.29
8	2.54	2.20	1.76	1.29
9	2.67	2.21	1.77	1.31
10	2.26	2.21	1.77	1.32
11	2.61	2.23	1.77	1.35
12	2.29	2.23	1.77	1.35
13	2.73	2.29	1.78	1.36
14	2.85	2.42	1.78	1.38
15	2.18	2.42	1.79	1.38
16	2.81	2.45	1.79	1.41
17	2.35	2.46	1.79	1.41
18	2.88	2.52	1.79	1.41
19	2.90	2.52	1.81	1.41
20	2.96	2.69	1.81	1.42
21	2.67	2.72	1.84	1.43
22	2.26	2.75	1.84	1.44
23	2.61	2.79	1.85	1.45
24	2.29	2.83	1.85	1.45
25	2.64	2.85	1.86	1.46
26	2.71	2.94	1.86	1.46
27	2.67	2.94	1.87	1.49
28	2.57	3.03	1.88	1.49
29	2.46	3.10	1.89	1.52
30	2.67	3.13	1.89	1.52
Mean	2.60	2.48	1.79	1.37
SD	0.23	0.35	0.063	0.093

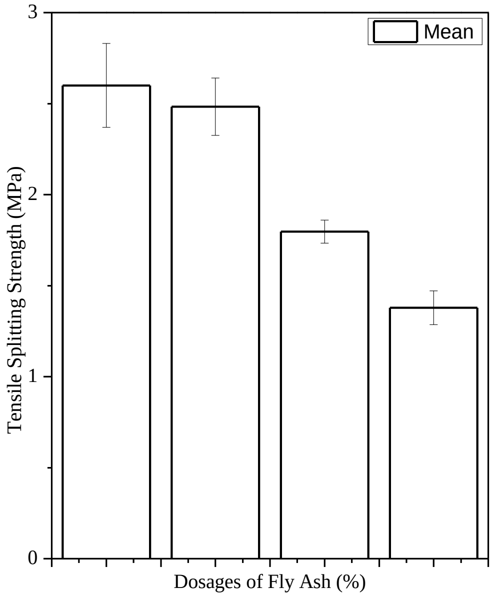
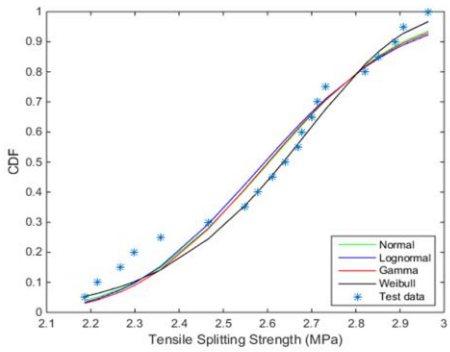


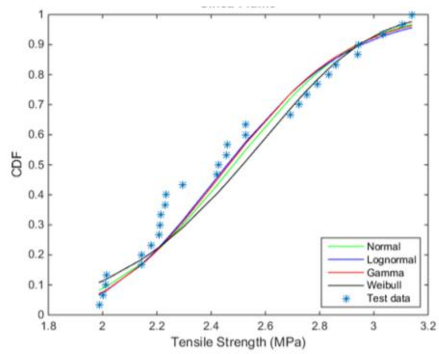
Figure 5.11: Variation of Mean, SD of tensile splitting strength of FA concrete

Table 5.12: Estimated parameters, KS distances, LK, and CS for different distribution functions describing tensile splitting strength of FA concrete

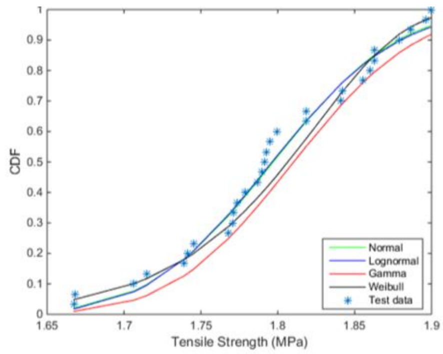
Mix	Distribution	Shape	Scale	KS	CS	LK
20% FA	Weibull	2.640	7.627	0.155	1.816	-12.278
	Gamma	50.54	0.0491	0.153	1.287	-10.822
	Normal	2.483	0.357	0.157	1.348	-11.247
	Lognormal	0.142	0.899	**0.149**	1.044	**-10.706**
40% FA	Weibull	1.826	32.861	0.144	0.611	-40.512
	Gamma	823.222	0.022	0.169	1.037	-39.839
	Normal	1.797	0.063	**0.087**	**0.928**	-40.636
	Lognormal	0.035	0.585	0.080	0.951	-40.491
60% FA	**Weibull**	1.421	18.037	**0.092**	**2.468**	-29.915
	Gamma	221.509	0.0062	0.143	3.730	-28.763
	Normal	1.379	0.093	0.110	3.183	-29.116
	Lognormal	0.068	0.319	0.118	3.277	-28.649



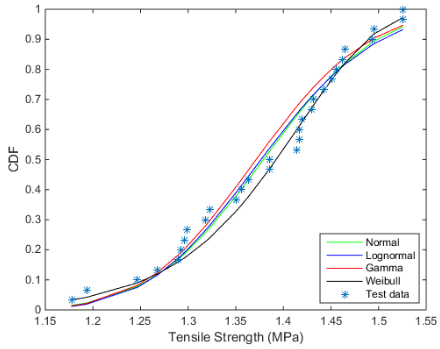
(a) Control



(b) 20% FA



(c) 40% FA



(d) 60% FA

Fig. 5.12: Experimental and assumed cumulative probability distributions for tensile splitting strength of FA concrete

5.4 Proposed Probability Distributions for SF and FA Concrete

Although mechanical properties considered are found to be fluctuating, based on the available statistical methods, the probabilistics regularity in the data is described using probability distribution models. Two probability distributions, namely, Weibull and lognormal probability distributions are found to be appropriate for the representation of variability for SF and FA concrete. However, considering the convenience, it is recommended to use lognormal distributions for the simulation of the variability of all mechanical parameters of SF and FA concrete in this study.

5.5 Probability-Based Seismic Risk Assessment for Typical Building using SF and FA Concrete

Having established the probabilistic representations of the variability in the mechanical properties of the SF and FA concrete, it is prudent to study the effect of the proposed probability distributions of compressive strength in the performance of buildings through probability-based seismic risk assessment. This section presents the prediction of seismic performance of buildings constructed using SF concrete or FA concrete.

5.5.1 Methodology

The methodology adopted in the present study is to evaluate the seismic risk of the typical buildings in terms of fragility curves and reliability indices which incorporate the variabilities in the materials and loading. An accepted simplified method reported by Ellingwood (2001) is adopted in the present study. The methodology for estimation of seismic risk involves three parts. First part is the identification of the seismic hazard, P [A = a], described by the annual probabilities of specific levels of earthquake motion. The seismic hazard at a site is usually represented through a seismic hazard curve, $G_A(x)$ which is a plot of P [A = a] versus the level of earthquake motion (a). The response analysis of the structure is carried out by conducting nonlinear time history analyses for different earthquakes, and the response is expressed in terms of maximum inter- storey drift at any storey. Third part is the calculation of limit state probabilities of attaining a series of (increasingly severe) limit states, LS_i, through the expression:

$$P[LS_i] = \sum_a P[LS_i \mid A = a] P[A = a] \tag{5.1}$$

The conditional probability in the above equation is denoted as the seismic fragility, $F_R(x)$. This conditional probability, explicitly stated, is the probability of meeting or exceeding a specified level of damage, LS, given a ground motion which has a certain level of intensity, a. This conditional probability is often assumed to follow a two parameter lognormal probability distribution [Song and Ellingwood 1999; Cornell *et al.* 2002; and Haran *et al.* 2015].

A point estimate of the limit state probability of state i can be obtained by convolving the fragility $F_R(x)$ with the derivative of the seismic hazard curve, $G_A(x)$, thus removing the conditioning on acceleration as per Eq. (5.2).

$$P[LS_i] = \int F_R(x) \frac{dG_A}{dx} dx \tag{5.2}$$

The parameters of the fragility-hazard interface must be dimensionally consistent for the probability estimate to be meaningful. The reliability index corresponding to the probability of failure can be found by the following standard equation as shown below.

$$\beta_{Pf} = -\phi^{-1}(P[LS_i]) \tag{5.3}$$

Where ϕ ( ) represents the standard normal distribution. The seismic demand is usually described through probabilistic seismic demand models (PSDMs) particularly for nonlinear time history analyses which are given in terms of an appropriate intensity measure (*IM*). It has been suggested by Cornell *et al* (2002) that the estimate of the median demand, *EDP* can be represented in a generalized form by a power model as given in Eq. 5.4.

$$EDP = a(IM)^b \tag{5.4}$$

Where, *a* and *b* are the regression coefficients of the Probabilistic Seismic Demand Model (PSDM).

Fragility curves are cumulative probability distributions that indicate the probability that a component/system will be damaged to a given damage state or a more severe one, as a function of a particular demand. The seismic fragility, $F_R(x)$ can be expressed in closed form using the following equation,

$$P(D \geq C \mid IM) = \phi\left(\frac{\ln \frac{S_D}{S_C}}{\sqrt{\beta_{D|IM}^2 + \beta_c^2}}\right) \tag{5.5}$$

where, *D* is the drift demand, *C* is the drift capacity at chosen limit state, S_D and S_C are the median of the demand and the chosen limit state (*LS*) respectively. $\beta_{d/IM}$ and β_c are dispersions in the intensity measure and capacities respectively. A fragility curve can be obtained for each limit state. The dispersion in capacity, β_c is dependent on the building type and construction quality. ATC 58 (2012) suggests values of β_c as 0.10, 0.25 and 0.40 depending on the quality of construction good, fair and poor respectively. In this study, dispersion in capacity has been assumed as 0.25. The methodology adopted in this study has been used by many researchers [Nielson 2005; Davis *et al.* 2010; Rajeev and Tesfamariam 2012; and Haran *et al.* 2015] in past to develop fragility curves of RC structures.

Limit states (LS) define the capacity of the structure to withstand different levels of damage. The median inter-storey drift limit states for RC moment resisting frame structures defining the capacity of the structure at various performance levels (S_C) are suggested by Ghobarah (2000) and ASCE/SEI 41-06 (2007). Drift limits for RC frames as per ASCE/SEI 41-06 (2007) are considered in the present study as 2% and 4% for significant damage (SD) and near collapse (CP) performance level respectively.

The seismic hazard (G_A) at a building site is displayed through a complimentary cumulative distribution function (*CCDF*). The hazard function is the annual frequency of motion intensity at or above a given level, *x*, to the intensity. Elementary seismic hazard analysis shows that at moderate to large values of ground acceleration, there is a logarithmic linear relation between annual maximum earthquake ground or spectral acceleration, and the probability, $G_A(a)$, that specific values of acceleration are exceeded. This relationship implies that A is described by following equation suggested by Ellingwood (2001).

$$G_A(x) = 1 - \exp[-(x/u)^{-k}] \tag{5.6}$$

u and *k* are parameters of the distribution. Parameter *k* defines the slope of the hazard curve which, in turn, is related to the coefficient of variation (COV) in annual maximum peak acceleration.

The seismic hazard curve of north-east region (Manipur) of India, which is one of most seismically active regions, developed by Pallav *et al.* (2012) as shown in Fig. 5.13 is selected for the present study. Annual frequency of being exceedance of PGA for a 2500 year return period (2% exceedance probability in 50 years) for Manipur Region is found to be 1.08g.

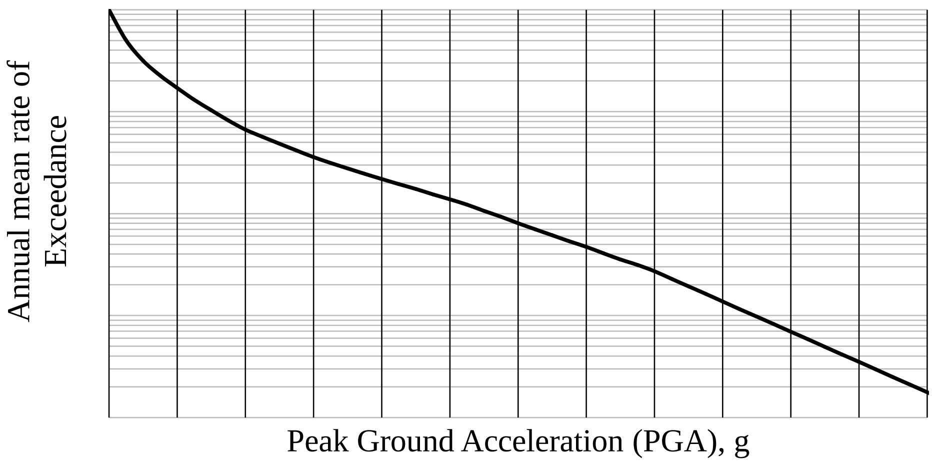


Fig.5.13: Seismic Hazard curves of North East region, India (Pallav *et al.* 2012)

5.5.2 Selected Building Frame

The typical building frame considered for numerical analysis in the present study is adopted from Haran *et al.* (2015). The selected frame is designed for the highest seismic zone (zone V with PGA of 0.36g) as per Indian standard IS 1893 (2002) considering medium soil conditions (N-value 10 to 30). The characteristic strength of concrete and steel are taken as 25MPa and 415MPa respectively. The buildings are assumed to be symmetric in plan, and hence a single plane frame is considered to be representative of the building along one direction. Typical bay width and column height in this study are selected as 5m and 3.2m respectively, as observed from the study of typical existing residential buildings. A configuration of four storeys and two bays is considered. The dead load of the slab (5 m × 5 m panel) including floor finishes is taken as 3.75 kN/m^2 and live load as 3 kN/m^2. The design base shear (V_B) is calculated as per equivalent static method (IS 1893, 2002). The structural analysis for all the vertical and lateral loads is carried out by ignoring the infill wall strength and stiffness (conventional). The design of the RC elements are carried out as per IS 456 (2000) and detailed as per IS 13920 (1993). In order to study the effect of variability in the compressive strength properties of concrete made by the partial replacement of cement by SF and FA, different building models considered to represent various practical cases with varying percentage of SF and FA. Buildings are named as XY, where X denotes 'SF' or 'FA' for silica fume or fly ash respectively. Y denotes the percentage of replacement of SF or FA. The building frame with normal concrete is represented as ‘C’. Fig. 5.14 shows the configuration of four storey two bay frame. Table 5.13 shows the design details of the selected frame.

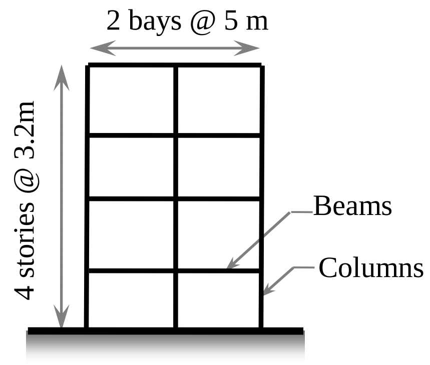


Figure 5.14: Selected four storey RC frame

Table 5.13: Design details of the selected building frame

Member	Floor no./ Storey no.	Width (mm)	Depth (mm)	Longitudinal Reinforcement detail	Transverse Reinforcement detail
Beam	1 to 3	300	450	[5-250 φ] (Top) + [4-20 φ] (Bottom)	10 φ@100 c/c
Beam	4	300	450	[5-250 φ] (Top) + [4-16 φ] (Bottom)	10 φ@100 c/c
Column	1-4	350	350	8-25 φ(Uniformly distributed)	10 φ@175 c/c

The most sensitive random variables [Damitri *et al.* 1997; Ozer 2006; Bal *et al*. 2008; Celik and Ellingwood 2010; Al Hafian and May 2012; and Seo *et al.* 2012], such as compressive strength of concrete and steel and global damping ratio in a constructed building frame are considered as random. Probabilistic model for compressive strength of concrete is obtained from the Section 5.7 of this thesis whereas this for compressive strength of steel and global damping ratio is considered from published literature. The mean and standard deviations (in terms of COV) of all the random variables are shown in Table 5.14.

Table 5.14: Concrete compressive strength of various buildings

Frame ID	Mean (MPa)	C.O.V (%)	Distribution	Source
Control	30.28	8.94	Lognormal	Present study
SF5	30.73	13.56	Lognormal	Present study
SF10	43.97	8.61	Lognormal	Present study
SF15	47.42	13.26	Lognormal	Present study
SF20	53.97	11.45	Lognormal	Present study
SF25	49.06	12.14	Lognormal	Present study
SF30	45.11	17.8	Lognormal	Present study
FA20	34.63	3.32	Lognormal	Present study
FA25	28.48	5.26	Lognormal	Present study
FA30	14.79	6.28	Lognormal	Present study
Steel	468.90	10.0	Normal	Ranganathan,1999
Global damping ratio	5%	40.0	Lognormal	Celik and Ellingwood, 2010

5.5.3 Results and Discussions of Seismic Risk Assessment

A set of 44 values of random variables are generated as per Latin Hypercube sampling technique to generate 44 computational models of frames for conducting nonlinear dynamic analysis. The 44 ground motions are scaled linearly from 0.1g to 1g and each of the 44 computational model is analysed for a particular earthquake (randomly selected) with a particular PGA. A total of 44 nonlinear dynamic time history analyses are performed and the maximum inter-storey displacement (ISD) for each storey are monitored. The inter-storey drifts (maximum of all storeys) along with the corresponding PGAs are plotted in a logarithmic graph for SF and FA buildings respectively in Figs. 5.15 and 5.16. Each point in the plot shown in Fig. 5.15 and 5.16 represents the PGA values and the corresponding percentage of maximum ISD in each of the 44 time history analysis for all the frames. A power law (refer Equation. 6) relationship, for each frames is fitted using regression analysis, which represents PSDM model for the corresponding frames. The PSDM model provides the most likely value of inter-storey drift (in mm) in the event of an earthquake of certain PGA (up to 1g) in each frame. Higher the value of inter-storey drifts, the higher will be the vulnerability of the building. The regression coefficients *a* and *b*, of the PSDMs are found for each frame and reported in Table 5.15.

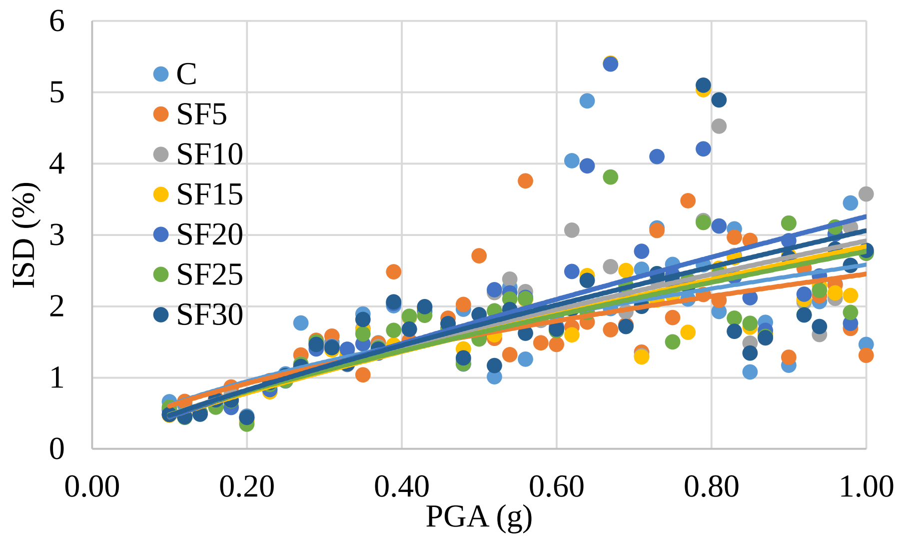


Figure 5.15: PSDM models for building frames using SF concrete

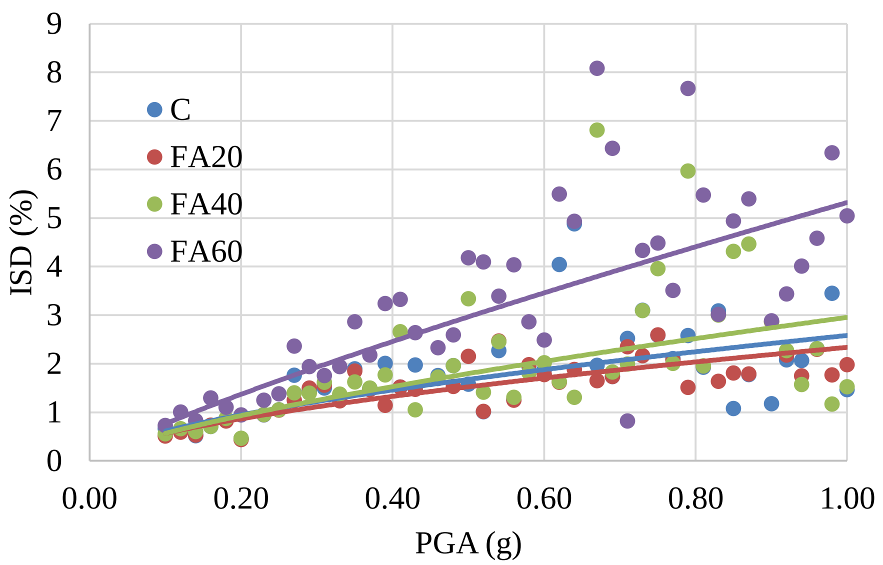
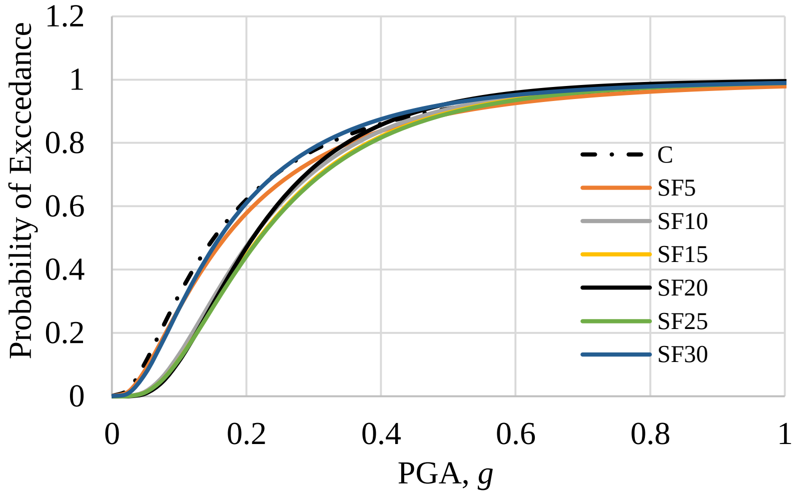


Figure 5.16: PSDM models for building frames using FA concrete

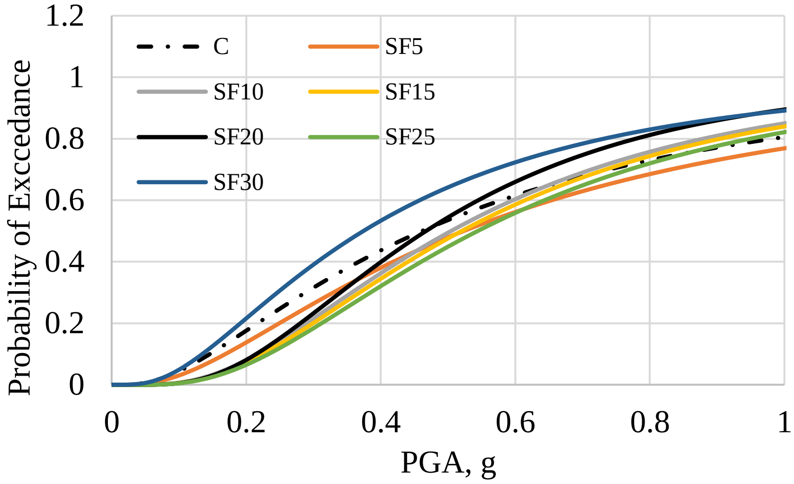
Table: 5.15: PSDM models for all the frames

Frame ID	$a(PGA)^b$	a	b
C	$2.58(PGA)^{0.62}$	2.58	0.62
SF5	$2.45(PGA)^{0.61}$	2.45	0.61
SF10	$2.92(PGA)^{0.79}$	2.92	0.79
SF15	$2.84(PGA)^{0.80}$	2.84	0.80
SF20	$3.26(PGA)^{0.86}$	3.26	0.86
SF25	$2.77(PGA)^{0.77}$	2.77	0.77
SF30	$3.06(PGA)^{0.81}$	3.06	0.81
FA20	$2.34(PGA)^{0.62}$	2.34	0.62
FA40	$2.95(PGA)^{0.72}$	2.95	0.72
FA60	$5.32(PGA)^{0.84}$	5.32	0.84

Having computed the PSDM models, the dispersions of inter-storey drifts $\beta_{D/IM}$ from the best fitted PSDM, the dispersion component β_{comp} are calculated. In order to study the performance of selected cases of buildings, the fragility curves are developed for all the frames for each performance limit states as shown in Figs. 5.17 and 5.18 for SF and FA frames. Figs. 5.17a and 5.17b show the fragility curves at SD and CP performance levels respectively for SF frames.

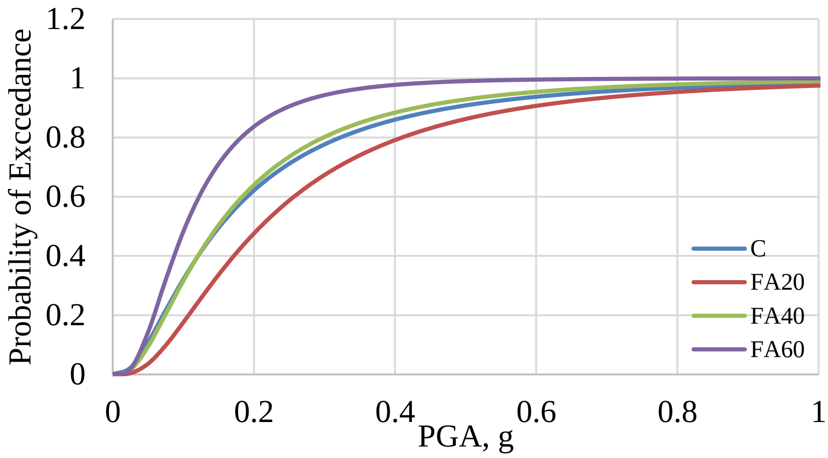


(a) At Significant Damage (SD)

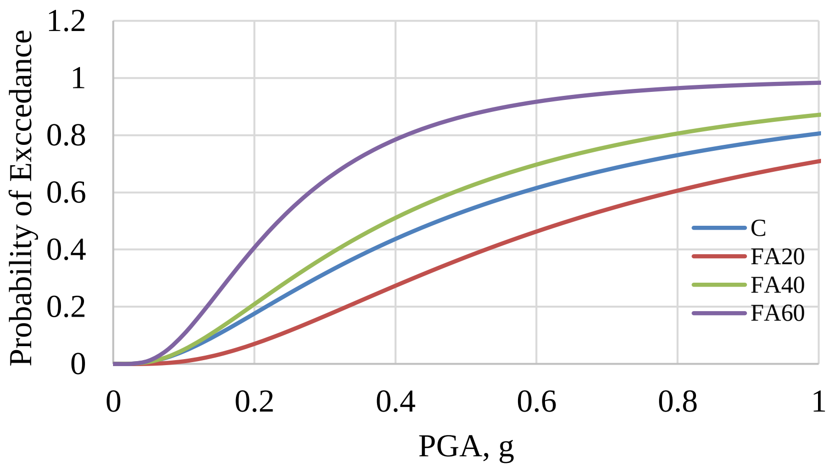


(b) At Collapse Prevention (CP)

Figure 5.17: Fragility curves for SF building frames



(a) At Significant Damage (SD)

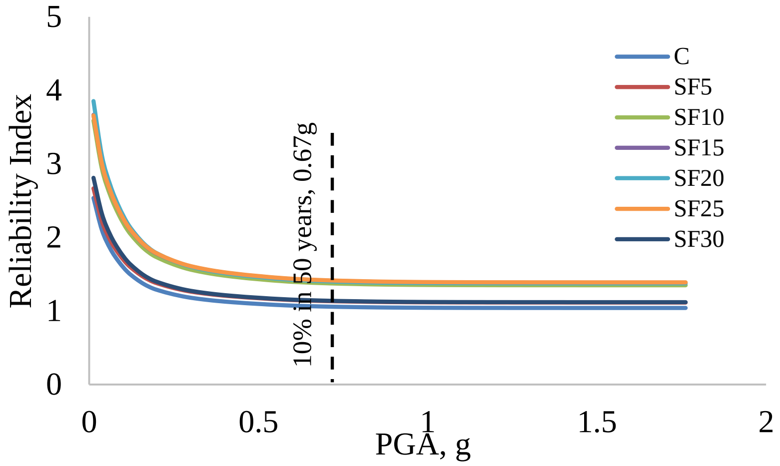


(b) At Collapse Prevention (CP)

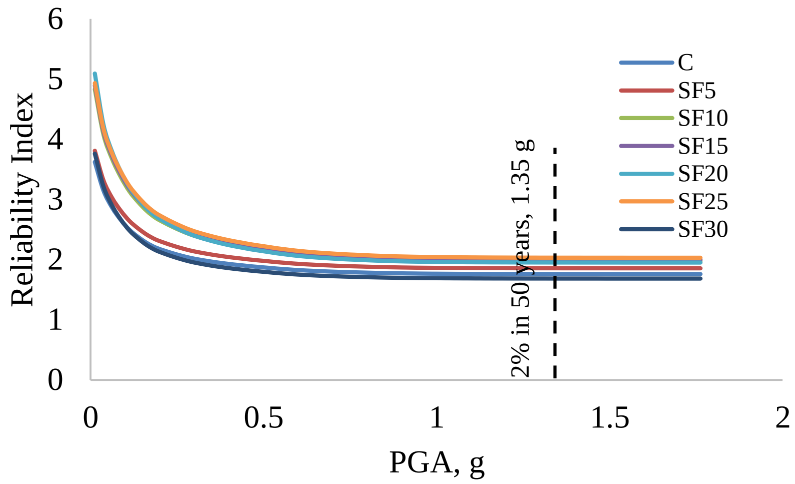
Figure 5.18: Fragility curves for FA building frames

It can be seen from Figs. 5.17a and 5.17b that the SF frames with 15%, 20% and 25% partial replacement of SF are found to be performing better than other frames for all performance limit states. Figs. 5.18a and 5.18b show the fragility curves at SD and CP performance levels respectively for FA frames. These figures show that the seismic performance of frames with 20% partial replacement of FA is found to be better than other frames. The frames having 60% FA replacement is found to be more vulnerable than the frame with normal concrete.

In order to understand the performance of each frame quantitatively, the seismic reliability indices are calculated for each frame. The reliability indices are estimated by combining the fragility curve for a particular limit states with hazard curves using the Eq.5.2. In present study, hazard curve of North East India is chosen for reliability index estimation. Reliability index is calculated for two performance objectives, PO-I and PO-II namely, Significant Damage (SD) performance level at an earthquake having 10% probability of occurrence in 50 years (PO-I) and Collapse Prevention (CP) performance level at an earthquake having 2% probability of occurrence in 50 years (PO-II). PGAs at 10% and 2% probability of occurrence are obtained from hazard curve fig. 5.13 as 0.67g and 1.35g respectively. Reliability indices for different performance levels in terms of various PGA values are presented in Figs. 5.19 and 5.20 for SF and FA buildings respectively. Figs. 5.19a and 5.19b show the variation of reliability indices for different PGAs for all SF frames at SD and CP performance levels respectively. Similarly, 5.20a and 5.20b show the variation of reliability indices for different PGAs for all FA frames at SD and CP respectively. The PGAs corresponding to PO-I and PO-II performance levels are marked in these figures for the calculation of reliability indices at each performance objectives. The reliability indices for all frames at PO-I and PO-II performance objectives are tabulated in Table 5.16. The reliability index values at PO-I and PO-II for SF15, SF20 and SF25 frames are found to be higher, which means that, adding SF in the range of 15% to 25% may yield better seismic performance. Therefore, for economy and durability point of view 15% to 20% may be considered for the practical applications. The reliability index values at PO-I and PO-II for the frame FA40 is found to the highest, which indicates that replacement of about 20- 40% may be considered to be the appropriate replacement ratio for better performance.

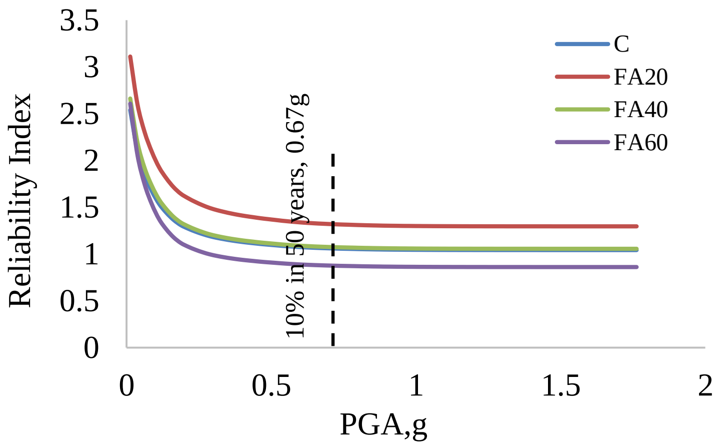


(a) At SD for SF building frames

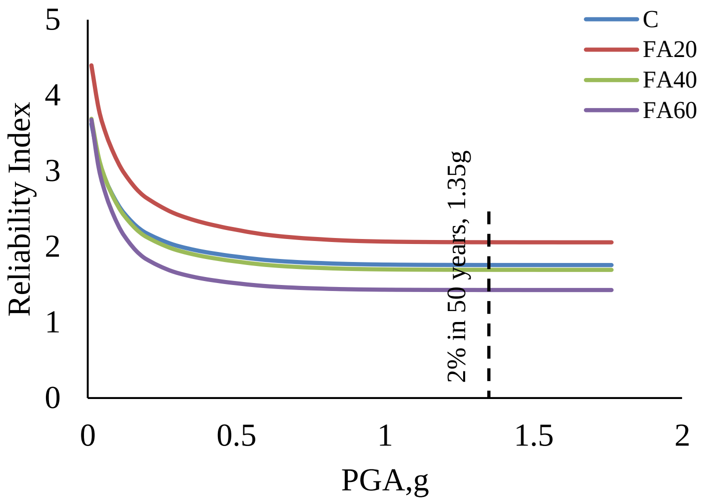


(b) At CP for SF building frames

Figure 5.19: Reliability curves for SF building frames



(a) At SD for FA building frames



(a) At CP for FA building frames

Figure 5.20: Reliability curves for FA building frames

Table 5.16. Reliability index (P_f) for SF and FA building frames

Frame ID	PO-II β_{Pf} (P_f)	PO-III β_{Pf} (P_f)
C	1.07 (1.423E-01)	1.76 (3.920E-02)
SF5	1.14 (1.271E-01)	1.85 (3.216E-02)
SF10	1.39 (8.226E-02)	1.98 (2.385E-02)
SF15	1.42 (7.780E-02)	2.00 (2.275E-02)
SF20	1.41 (7.927E-02)	1.95 (2.559E-02)
SF25	1.42 (7.780E-02)	2.03 (2.118E-02)
SF30	1.14 (1.271E-01)	1.68 (4.648E-02)
FA20	1.07 (1.423E-01)	1.76 (3.920E-02)
FA40	1.33 (9.176E-02)	2.06 (1.970E-02)
FA60	1.08 (1.401E-01)	1.69 (4.551E-02)

5.6 Conclusions

The study on the statistical variations is carried out using the experimental data and considering several two parameter probability distribution functions with an aim to describe the variability of the mechanical properties of SF and FA concrete. Several two-parameter distributions are selected to find the best-fit model that describes the experimental data closely. Based on the limited set of data and using three goodness-of-fit tests, such as minimum KS distance, minimum CS and maximum LK values, most appropriate statistical distributions for the selected parameters are proposed. The three selected statistical criteria (KS, CS and LK) are not always found to be in agreement with a single distribution for some of the concrete mixes. In such cases the closest fit model is selected based on the KS distance and LK value [Chen *et al*. 2014]. For other mixes, a single distribution is found to meet all the three validating criteria. This study proposed lognormal distribution function as the distribution model that closely describes the variations of different mechanical properties of SF and FA concrete for practical point of view. Further, performance of typical selected buildings using SF and FA concrete is evaluated through fragility curves and reliability indices incorporating the proposed probability distributions and variability of material properties. It is found that 15% to 20% of partial replacement of cement with SF and 20-40% partial replacement of cement with FA may yield better performance of frames.

Chapter 6

Summary and Conclusions

6.1 Summary

The following primary objectives of this research were identified from a detailed literature review (discussed in chapter 2) conducted on the materials that can be used in concrete by recycling for sustainability.

- To study the relationship of *w/c* ratio and compressive strength, the effects of age and number of recycling on the properties of RCA concrete.
- To study the enhancement of engineering properties of RCA concrete using bacteria.
- To investigate the mechanical properties of low to medium strength SF concrete incorporated with 10% additional cement quantity as per the construction practice.
- To describe the variability in the properties of both SF and FA concrete.

Eco-friendliness and sustainability of engineering constructions indirectly improve when more recycled waste materials incorporate in concrete. The first part of the present study focused on properties of concrete made using RCA. The relationship between the water-to-cement (*w*/*c*) ratio and the compressive strength of RCA concrete was studied to understand its behaviour compared to normal concrete. Influence of age and number of recycling are some parameters that can affect the quality of RCA concrete. A number of experimental tests were conducted to investigate on these aspects.

It is reported by many studies that mineral precipitation by bacteria can enhance the properties of normal concrete. Present study used to ureolytic-type bacteria, *B.subtilis* and *B. sphaericus* to enhance the properties of RCA concrete. The properties such as compressive strength, durability and shrinkage of RCA concrete incorporated with bacteria were investigated in the present study. Microstructural analyses, XRD and field emission scanning electron microscopy (FESEM) were also conducted to understand the morphology of bacterial concrete. In order to understand the effect of bacteria on the properties of mortar, further investigations were carried out on cement mortar incorporating *B. sphaericus*. (All the experimental work such as making of RCA, bacteria culture, specimen preparation and testing are explained in chapter 3)

It is reported that SF improves the properties of concrete due to its pozzolanic activity. Most of the previous studies on the SF concrete focused on Portland cement and high strength concrete. International codes suggest an increase of 10% cement when SF concrete is used in construction practice. The present study investigated the mechanical properties of low-medium strength SF concrete made as per the above construction practice using slag cement. Similar studies are also carried out using FA. (Chapter 4 presents the mechanical properties of SF and FA concrete).

Uncertainties in loading and the capacities of structural members are not avoidable. The fluctuating nature of the mechanical properties (compressive strength, flexural strength and tensile splitting strength) can have a significant impact on the performance of structures. The probability distribution models that can represent the fluctuating nature of normal concrete is available in literature. The present study attempted to propose the probabilistic descriptions of the mechanical properties of SF and FA concrete using three statistical goodness of fit tests. Several two-parameter distributions are selected to find the best-fit model that describes the experimental data closely (Chapter 6 of this thesis explains the descriptions of variability in the mechanical properties of SF and FA concrete using various probability distribution functions). The proposed probability distributions are used to study probabilistic performance of typical buildings made of SF and FA concrete in a probabilistic frame work.

6.2 Conclusions

The following specific conclusions are drawn from the present research:

- It was found that the RCA concrete requires a threshold minimum quantity of water depending upon the parent adhered mortar to contribute to the strength. This minimum quantity of water in terms of *w/c* ratio for one year old and two year old RCA concrete were about 0.37 and 0.42 respectively. In order to obtain higher compressive strength for RCA (than NCA), *w/c* ratio should be higher than the above mentioned threshold limits.
- The compressive strength of concrete prepared from older (RC-2) aggregate was found to be about 6% lower in comparison with RC-1. The split tensile strength

and flexural strength of RC-2 concrete are about 14 to 28% and 6% to 21% lesser than that of RC-1 concrete respectively.

- The successive recycling decreases the quality of concrete due to higher water absorption of the recycled aggregate. The compressive strength of concrete after two times of recycling was about 2% less than that of one time of recycling. It was found that the capillary water absorption of N2-RC-1 is about 9 times more than both RC-1 and NCA concrete. Further recycling found to be increasing the air content of RCA concrete. The successive recycling reduces the splitting tensile strength and flexural strength by 6% and 12% respectively.
- Properties of RCA concrete such as compressive strength, capillary water absorption and drying shrinkage are improved by the addition of *Bacillus subtilis* and *Bacillus sphaericus*. The compressive strength of RCA concrete at 28 days is found to be increased by about 21% for *B. subtilis* (B-3a) and 36% for *B. sphaericus* (B-3b) with respect to RCA control mix at an optimum cell concentration of 10^6 cells/ml. Calcium carbonate precipitation by *B. subtilis* and *B. sphaericus* in the form of calcite is confirmed through microstructure analysis using SEM, EDX and XRD. *B. subtilis* and *B. sphaericus* can reduce the drying shrinkage strain and capillary water absorption of RCA concrete and thereby enhances the durability.
- The compressive and splitting tensile strength of concrete increases gradually from a SF dosage of 5%, to reach an optimum value at 20%. A cap of about 21% in comparison with previous studies is found at 20% replacement. This cap can be attributed to the use of 10% extra cement. The flexural strength of concrete prism increases gradually with increase of SF doses, to reach an optimum value at 25%. A cap of about 10% in comparison with previous studies is observed at 20% replacement.
- The present study proposed lognormal distribution function for the description of variability of mechanical properties such as compressive strength, flexural strength and tensile splitting strength of SF and FA concrete based on three statistical goodness of fit tests.
- Incorporating the proposed probability distributions, the seismic performance of selected typical buildings using SF and FA concrete in a probabilistic framework were evaluated through fragility curves and reliability indices. It was found that

15% to 20% of partial replacement of cement with SF and 20-40% partial replacement of cement with FA may yield better seismic performance of frames.

6.3 Main Contribution of Research

The following enlisted are the main contributions of this thesis:

1. The relationship between *w/c* ratio and compressive strength of RCA concrete was found out experimentally considering the age and number of recycling of RCAs.
2. The present study found that the strength and durability properties of the RCA concrete and cement mortar are improved by incorporation of two bacteria, *Bacillus Sphaericus* and *Bacillus subtilis*. Optimum dosage of bacteria for the maximum improvement in the strength was obtained.
3. The present study confirms that the partial replacement of cement by SF and FA can improve strength even for PSC.
4. Probability distribution models are proposed for the description of variability of the mechanical properties of SF and fly-ash concrete by replacement of cement. Effect of the variability in the mechanical properties of the concrete on the performance of RC frames is studied.

6.4 Future Scope of the Research Work

The following are the scope for the extension of the present work:

1. The present study can be extended to develop the design code provisions required for RCA concrete in line with that of normal concrete.
2. The present study considered RCA concrete having two ages, one year and two year. This study can be extended to consider much older demolished concrete to represent more realistic situations.
3. This study can be extended to propose stress versus strain relationship of the RCA concrete considering age and number of recycling as different parameters.
4. The stress versus strain relationship of the RCA concrete incorporating bacteria concrete is not available in literature. This study can be continued in this direction.
5. The present study used SF and FA from a single source. The present study can be extended to develop the variability descriptions among various sources.

6. This study can be extended to make the bacterial concrete to more commercial friendly.
7. The cost benefit analysis for the use of recycled materials can be studied.
8. A systematic guideline to design sustainable concrete mix using RCA/SF/FA or mineral precipitating bacteria can be arrived at through specific studies for each of these materials separately.

Appendix A

Bacterial Concrete

A.1 Introduction

In the present study *Bacillus sphaericus* (*B. sphaericus*), a non-contageous ureolytic bacteria, is used to improve the strength and durability of the cement mortar through bio-mineralization. Bio-mineralization is defined as a biologically induced mineral precipitation in which an organism creates a local micro-environment with conditions that allow optimal extracellular chemical precipitation of mineral phases. This is induced by heterogeneous nucleation on bacterial cell walls [Ramachandran *et al.* 2001; and Bachmeier *et al.* 2002]. This section presents the mechanism of bio-mineralization briefly.

A.2 Mechanism of Bacterial Precipitation

The microbial precipitation of $CaCO_3$ is determined by several factors including: the concentration of dissolved inorganic carbon, the pH, the concentration of calcium ions and the presence of nucleation sites. The first three factors are provided by the metabolism of the bacteria while the cell wall of the bacteria will act as a nucleation site. The bacteria produce urease which catalyses the hydrolysis of urea ($CO(NH_2)_2$) into ammonium (NH_4^+) and carbonate (CO_3^{2-}). First, one mol of urea is hydrolysed intracellular to one mol of carbamate and one mol of ammonia (Eq. A.1). Carbamate spontaneously hydrolyses to form additionally one mol of ammonia and carbonic acid (Eq. A.2). These products subsequently form one mol of bicarbonate and two mol of ammonium and hydroxide ions (Eqs. A.3 and A.4). The last two reactions give rise to a pH increase, which in turn shifts the bicarbonate equilibrium, resulting in the formation of carbonate ions (Eq. A.5)

$$CO\,(NH_2)_{2+}\,H_2O \rightarrow NH_2COOH + NH_3 \tag{A.1}$$

$$NH_2COOH + H_2O \rightarrow NH_3 + H_2CO_3 \tag{A.2}$$

$$H_2CO_3 \leftrightarrow HCO_3^- + H^+ \tag{A.3}$$

$$2\,NH_3 + 2\,H_2O \rightarrow 2NH_4^+ + 2OH^- \tag{A.4}$$

$$HCO_3^- + H^+ + 2NH_4^+ + 2OH^- \rightarrow CO_3^{2-} + 2NH_4^+ + 2\,H_2O \tag{A.5}$$

Since the cell wall of the bacteria is negatively charged, the bacteria draw cations from the environment, including Ca^{2+}, to deposit on their cell surface. The Ca^{2+}-ions subsequently react with the CO_3^{2-}-ions, leading to the precipitation of $CaCO_3$ at the cell surface that serves as a nucleation site (Eqs. A.6 and A.7).

$$Ca^{2+} + Cell \rightarrow Cell\text{-}Ca^{2+} \tag{A.6}$$

$$Cell\text{-}Ca^{2+} + CO_3^{2-} \rightarrow Cell\text{-}CaCO_3\downarrow \tag{A.7}$$

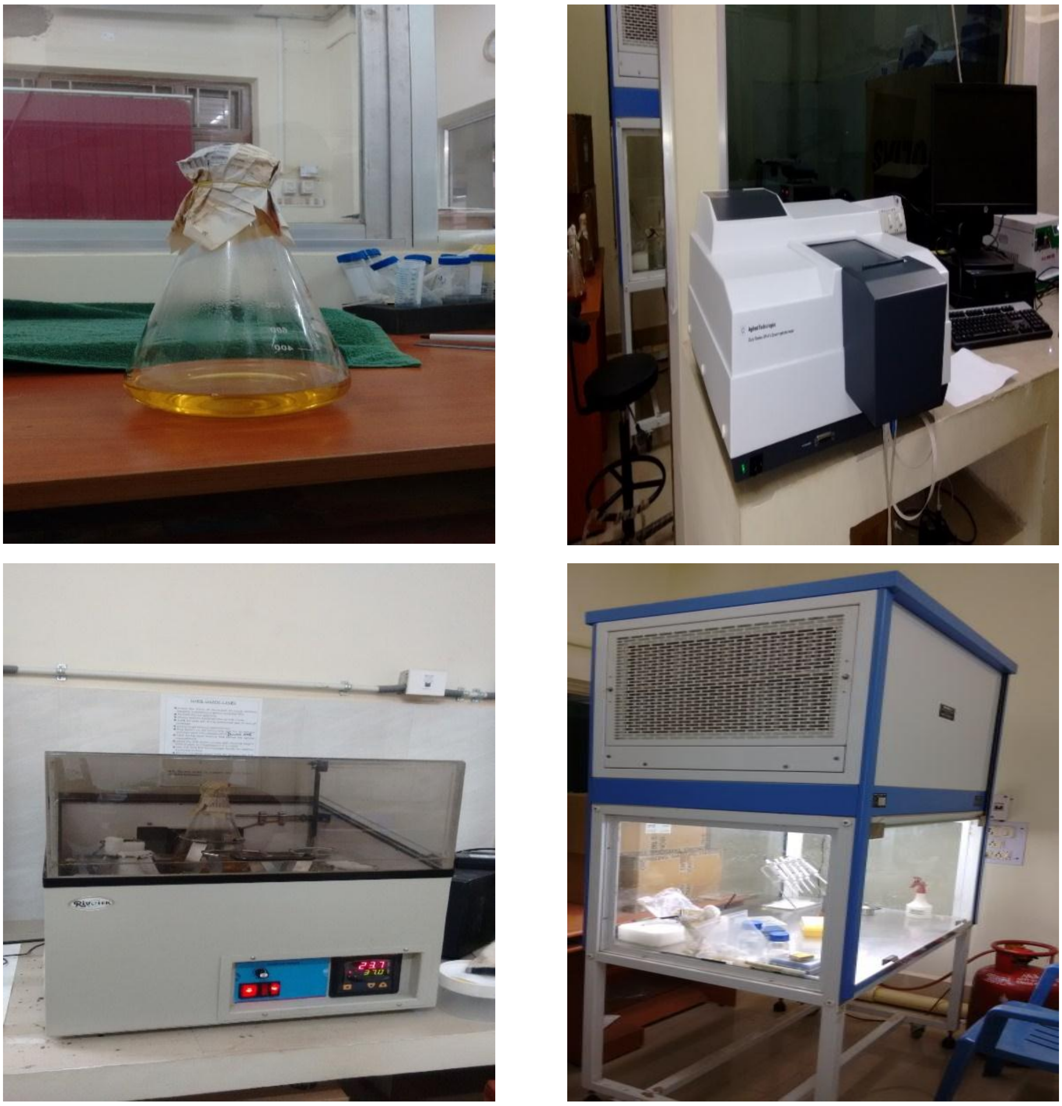


Figure A.1**:** Instruments used for bacteria culture

Appendix B

Probability Distributions and Goodness-of-Fit Tests

B.1 Introduction

Different two parameter probability functions are adopted in Chapter 5 of this thesis for description of the variation in different mechanical properties of brick, mortar and brick masonry. Best fitted probability distribution function is arrived through different goodness-of-fit tests. The probability distribution functions considered are normal, lognormal, gamma and Weibull. The goodness-of-fit test includes Kolmogorov-Smirnov (KS), Log-likelihood (LK) and minimum Chi-square criterion (CS). This section describes selected probability distribution functions and the goodness-of-fit tests briefly.

B.2 Probability Distributions

B.2.1 Normal Distribution

Normal distribution (also known as Gaussian distribution) is one of the most commonly used continuous type of distribution. It is a special case of binomial distribution and is suitably applicable for large data. Normal distribution is a two-parameter probability distribution function that consists of mean (μ) and standard deviation (σ) as its parameters. This distribution is applicable in situations where the random variable is dependent on several other independent variables.

The probability density function (PDF) of the normal distribution is given by Eq.B.1

$$N(x;\mu,\sigma)=\frac{1}{\sigma x\left(\sqrt{2\pi}\right)}\exp\frac{-(x-\mu)^2}{2\sigma^2} \tag{B.1}$$

The cumulative distribution function (CDF) of the normal distribution is given by Eq.B.2

$$N(x;\mu,\sigma)=\frac{1}{\sigma x\left(\sqrt{2\pi}\right)}\int_{-\infty}^{x}\frac{(-t-\mu)^2}{2\sigma^2}dt \tag{B.2}$$

Where x, μ and σ are random variable, mean and standard deviation respectively. The normal distribution is symmetric and uni-modal about the mean. The curve has maximum value at $x = \mu$ and point of inflexion at $x = \mu + \sigma$. The PDF of normal distribution range from -∞ to +∞ while the CDF is from 0 to 1 which is used in goodness-of-_t tests. Because of simplicity, many researchers prefer considering normal distribution than other distributions. This distribution could be used for both positive and negative outcomes. However engineering properties such as compressive strength or weight is always positive in that case the range of normal distribution can be truncated to the requirement and called as truncated normal distribution.

B.2.2 Lognormal Distribution

Lognormal distribution is similar to normal distribution and both are interrelated to each other. This distribution can be appropriately used when the outcomes are non-negatives. In such cases the data is skewed and not symmetrical like normal distribution. It is a two-parameter distribution having mean (μ) and variance (σ) as its parameters.

If a variate X is such that log(X) is normally distributed, then the distribution of X is said to be lognormal. The range of lognormal distribution is from 0 to +∞. Lognormal distribution finds its application in many engineering fields because it captures the non-negative values.

The PDF of the lognormal distribution is given by Eq. B.3

$$L(x;\mu,\sigma) = \frac{1}{\sigma x\left(\sqrt{2\pi}\right)} \exp \frac{-(\log x - \mu)^2}{2\sigma^2} \tag{B.3}$$

when x ≥ 0 otherwise 0

The CDF of the lognormal distribution is given by Eq. B.4

$$L(x;\mu,\sigma) = \frac{1}{\sigma x\left(\sqrt{2\pi}\right)} \int_0^x \frac{\exp \frac{-(\log x - \mu)^2}{2\sigma^2}}{t} dt \tag{B.4}$$

B.2.3 Gamma Distribution

Gamma distribution (also known as Erlang distribution) is used to model data which is only positive and is derived from gamma function. It is a two-parameter distribution with positive parameters σ and λ. The mean (μ) and variance (σ^2) of gamma distribution are given in Eq. B.5

$$\mu = \frac{\alpha}{\lambda} \qquad \alpha^2 = \frac{\alpha}{\lambda^2} \tag{B.5}$$

where α and λ are called as shape and scale factors respectively

The PDF of the gamma distribution is given by Eq. B.6

$$G(x;\alpha,\lambda) = \lambda^{\alpha} e^{-\lambda x} \frac{x^{(\alpha-1)}}{(\alpha-1)} \tag{B.6}$$

when x ≥0 otherwise 0

The CDF of the gamma distribution is given by Eq. B.7

$$N(x;\mu,\alpha) = \frac{1}{\lambda^{\alpha}\Gamma(\alpha)} \int_0^x t^{\alpha-1} e^{\frac{-t}{\lambda}} dt \tag{B.7}$$

By changing the shape and scale parameters of gamma distribution, curves with different shapes can be generated fitting to the outcome. This makes gamma distribution reliable and flexible. Many distributions such as normal, exponential and chi-square can be derived from shape and scale parameters of gamma distribution.

B.2.4 Weibull Distribution

Weibull distribution is one of the versatile and widely used distributions. The application of Weibull involves reliability and life testing such as to model the time of failure or life of a component. This distribution has two parameters α and β which are its shape and scale parameter respectively. The distribution can also have another parameter depending on its location called as location parameter (γ).

The PDF of the Weibull distribution is given by Eq. B.8

$$W(x;\alpha,\beta) = \frac{\alpha}{\beta^{\alpha}} x^{\alpha-1} e^{\left(\frac{-x}{\beta}\right)^{\alpha}} \tag{B.8}$$

when x ≥ 0 otherwise 0

The CDF of the Weibull distribution is given by Eq. B.9

$$W(x;\alpha,\beta) = \int_0^x \beta\alpha^{-\beta} t^{\beta-1} e^{\left(\frac{-t}{\alpha}\right)^{\beta}} dt \tag{B.9}$$

B.3 Goodness-of-Fit Tests

B.3.1 Kolmogorov-Sminrov Test

Kolmogorov-Sminrov (KS) test is based on the statistic that measures the deviation of the observed cumulative distribution from the hypothesized cumulative distribution function as shown in Fig. B.1. The main advantage of KS test is that it can be performed on less number of samples. It utilizes the unaltered and un-aggregated form of data, as binning or lumping of data is not necessary. The only disadvantage of KS test is that it is valid only for continuous distributions.

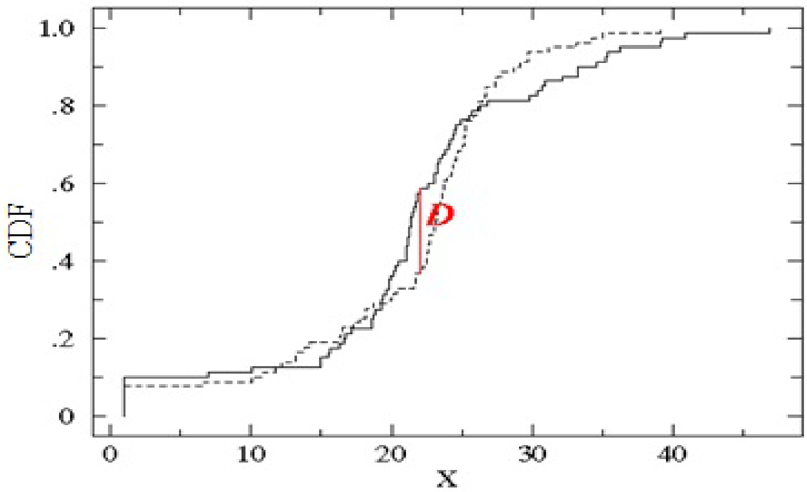


Figure B.1: KS test plot showing deviation between observed and hypothesizes CDF

B.3.2 Chi-square Test

Chi-square (CS) test measures the difference between the frequencies of observed samples and hypothesized samples. CS test is essentially a large sample test and yields better results if sample size is more than 50. In this test the data is divided in to a number of bins and the frequencies are derived. These frequencies are compared with the frequencies from the hypothesized distribution. Then the chi-square value is calculated from the sum of deviations between the frequencies. A significance level is assumed and the critical chi-square value is obtained from the chi-square distribution table. If the observed chi-square value is less than critical chi-square at assumed significance level then the hypothesis is accepted else it is rejected. The main disadvantage of CS test is that it involves dumping of data into bins for calculating the frequency which is influenced by the change in bins.

B.3.3 Log-likelihood Test

Log-likelihood (LK) test is used to measure the goodness-of-fit of two models. The test is based on log-likelihood ratio that expresses the number of times the data of one model is more likely than other. In this study the log-likelihood values are determined from MATLAB which returns negative log-likelihood values.

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3. Sahoo KK, Sarkar P and Davis R. (2016). Artificial neural networks for prediction of compressive strength of recycled aggregate concrete, *International Journal of Research in Chemical, Metallurgical and Civil Engineering*, **3(1)**: 81-85, DOI: 10.15242/IJRCMCE.IAE0316414.

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