DEVELOPMENT OF CEMENT CONCRETE WITH NON – BIODEGRADABLE WASTE PRODUCTS AND SUPPLEMENTARY MATERIALS –

AN EXPERIMENTAL INVESTIGATION

THESIS

Submitted in partial fulfillment
of the requirements for the degree of
DOCTOR OF PHILOSOPHY

by

MALAGAVELLI VENU
ID. No. 2004PHXF438H

Under the Supervision of
Prof. P. N. Rao

BIRLA INSTITUTE OF TECHNOLOGY AND SCIENCE, PILANI

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To my parents and wife
M. Nagabhushanam, M. Venkatalakshmamma and M. Kiranmai
with love, gratitude and deep respect.

BIRLA INSTITUTE OF TECHNOLOGY AND SCIENCE, PILANI

CERTIFICATE

This is to certify that the thesis entitled "**DEVELOPMENT OF CEMENT CONCRETE WITH NON – BIODEGRADABLE WASTE PRODUCTS AND SUPPLEMENTARY MATERIALS – AN EXPERIMENTAL INVESTIGATION**" and submitted by **MALAGAVELLI VENU, ID. No. 2004PHXF438H**, for the award of Ph.D. degree of the Institute, embodies the original work done by him under my supervision.

Signature of the supervisor:

Name in capital letters: **Dr. P. N. RAO**

Designation: Professor

Date: 10/04/2014

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DEVELOPMENT OF CEMENT CONCRETE WITH NON- BIODEGRADABLE WASTE PRODUCTS AND SUPPLEMENTARY MATERIALS – AN EXPERIMENTAL INVESTIGATION

ABSTRACT

Cement concrete has made tremendous strides in the past decade. The development of specifying a concrete according to its performance requirements, rather than the constituents and ingredients has opened innumerable opportunities for producers of concrete and users to design concrete to suit their specific requirements. The total annual concrete consumption in India is estimated at about 450 Million cubic meters (2012), which includes the consumption of concrete in infrastructure projects. Cement consumption in the world has increased exponentially since 1926 and is continuing to increase.

Cement production contributes to CO_2 emission because of calcinations of limestone during production of cement. $CaCO_3$ is calcined to CaO and CO_2 is released. Both embodied energy and direct emission contribute to total CO_2 emissions. Total amount of CO_2 emitted per ton of cement production ranges from 0.82 ton to 1.1 ton. Thus, control of this greenhouse gas emission is a major issue for sustainable concrete. Use of supplementary cementitious material, especially industrial waste products such as blast furnace slag and fly ash in concrete to reduce ordinary portland cement consumption is currently being considered as a major step towards achieving sustainability of cement concrete. The other side, many municipalities are facing the problem of disposal of non-biodegradable waste material like plastics.

Hence, the present experimental investigation is aimed at studying the behavior of concrete with locally available Ground Granulated Blast furnace Slag (GGBS) and ROBO sand (quarry dust) as supplementary materials for cement and fine aggregates respectively. Also the plastic waste i.e. High Density Polyethylene (HDPE), Polyethylene Terephthalate (PET), High Density Poly Propylene (HDPP) and POLYESTER in the form of fibers is added to the concrete for further study. For the present study, development of M25 and M30 cement concrete is being adopted. Detailed experimental investigation has been carried out to understand the behavior of concrete and results are compared with the conventional design mix concrete.

The results are quite encouraging with these supplementary materials in the concrete. The combination of GGBS as cement replacement and ROBO sand as fine aggregates can be replaced in the concrete by 50% and 25% respectively. Use of plastic waste as fibers (3.5%) in the concrete, the strength properties of concrete (load carrying capacity) is increased. Overall the compressive, split tensile and flexural strengths are increasing with the addition of non-biodegradable waste products as fibers and supplementary materials.

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LIST OF ABBREVIATIONS

AAF	Acid Attack Factor
ACI	American Concrete Institute
ADF	Acid Durability Factors
Al_2O_3	Alumina
AM	Aluminate Modulus
ASTM	American Society for Testing Materials
AR	Alkali Resistant Glass
BA	Bottom Ash
BFSA	Blast Furnace Slag Aggregates
Ca_2SO_4	Gypsum
$CaCl_2$	Calcium Chloride
C	Concrete without admixture
CA	Concrete with admixture (0% GGBS)
C_3A	Tricalcium Aluminate
CAG	CA+ 50% GGBS
CAH	Calcium Aluminate Hydrate
CAGR	CA + 50% GGBS + 25% ROBO Sand
CAGRHE	CA + 50% GGBS + 25% ROBO Sand + 3.5% HDPE
CAGRPE	CA + 50% GGBS + 25% ROBO Sand + 3.5% PET
CAGRHD	CA + 50% GGBS + 25% ROBO Sand + 3.5% HDPP
CAGRPO	CA + 50% GGBS + 25% ROBO Sand + 3.5% POLYESTER

CaO	Calcium oxide
CC	Conventional Concrete
CFA	Combustion Fly Ash
CIPET	Central Institute of Plastics Engineering and Technology
CO_2	Carbon dioxide
CCRP	Conditioned Crushed Rock Powder
CRP	Crushed Rock Powder
CSH	Calcium Silicate Hydrate gel
CSF	Condensed Silica Fume
CW	Colemanite ore Waste
FA	Coal Fly Ash
FB	Ground Fluidized Bed combustion fly ash
FRC	Fiber Reinforced Concrete
GFRP	Glass Fiber Reinforced Polymer
GGBS	Ground Granulated Blast Furnace Slag
GP	Glass Powder
HCl	Hydro Chloric acid
HDPE	High Density Polyethylene
HDPP	High Density Polypropylene
HPC	High Performance Concrete
H_2SO_4	Sulfuric Acid
HSC	High Strength Concrete
IS	Indian Standards

LSF	Lime Saturation Factor
MK	Metakaolin
Na_2SO_4	Sodium Sulphate
OPC	Ordinary Portland Cement
PCC	Plain Cement Concrete
PET	Polyethylene Terephthalate
POFA	Ground Palm Oil Fuel Ash
PP	Polypropylene
PUF	Polyurethane Formaldehyde
PVC	Polyvinyl Chloride
RCC	Reinforced Cement Concrete
RHA	Rice Husk Ash
RHBA	Ground Rice Husk–Bark Ash
RI	Reinforcing Index
RPC	Recycled Polyethylene terephthalate polymer Concrete
SCC	Self – Compacting Concrete
SCM	Supplementary Cementitious Materials
SiO_2	Silicon dioxide
SNP	Sulphonated Naphthalene Polymer
UFA	Ultrafine Fly Ash
USBR	United States Bureau of Reclamations
VSP	Vizag Steel Plant
WPLA	Waste bottles Lightweight Aggregate

WPLAC	Waste Polyethylene terephthalate bottles Lightweight Aggregate Concrete
w/b	Water binder ratio
w/c	Water cement ratio

LIST OF SYMBOLS

English Symbols

A	Material constant 1
a	Cross sectional area of the cube
B	Material constant 2
C	Material constant 3
C_u	Coefficient of uniformity
C_c	Coefficient curvature
D	Material constant 4
d	Diameter of the cylinder specimen
E_{it}	Initial tangent modulus of elasticity
E_0	Secant modulus at peak stress
E_{inf}	Slope at the inflection point of descending branch of the stress strain curve
f_c	Stress at any point on the stress strain curve
f_c'	Unconfined concrete compressive strength
f_{cf}'	compressive strength of fiber concrete
f_{cr}	Flexural strength of concrete
f_o''	Strength of confined concrete
f_t	Split tensile strength of concrete
k	Correction factor
k_1	Correction factor
k_2	Correction factor

l Length of the cylinder specimen

n Material parameter

P Compressive load at failure

w Concrete unit weight

X Normalized strain

Y Normalized stress

Greek Symbols

α_E Coarse aggregate coefficient

β Material parameter

ε Strain at any point on the stress strain curve

ε_d Strain corresponding to stress value of 0.3 f_c' in the descending part of the stress strain curve

ε_o'' Strain at peak stress of confined concrete

ε_{max} Strain when concrete stress is equal to 0.5 f_c' on the descending part of the stress strain curve

ε_0 Peak strain of unconfined concrete strength f_c'

ε_{of} strain corresponding to the compressive strength

ξ coefficient

φ Diameter of cylinder

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CHAPTER 1
INTRODUCTION

1.1 BACKGROUND

Concrete is an extraordinary and key structural material in the human history. It is an artificial compound generally made by mixing a binding material like cement with fine aggregate, coarse aggregate and water in suitable proportions. As written by Brunauer and Copeland (1964), "Man consumes no material except water in such tremendous quantities". With the development of human civilization, concrete will continue to be a dominant construction material in the future. The reasons are:

1. It can be readily moulded into durable structural items of various sizes and shapes.
2. It is possible to control the properties of concrete in a wide range by using appropriate ingredients and by applying special processing techniques.
3. It is possible to mechanize completely its preparation and placing processes.

However, the development of cement industry also introduces the major producer of greenhouse gases and energy user. This will create many environmental problems such as pollution, waste dumping, emission of dangerous gases, depletion of natural resources etc. This will lead to sustainability issues including environmental and social aspects. In general, sustainability embodies all the provisions necessary for excellent engineering solutions. Such solutions are those that contribute in a balanced measure to profitability, long-term community benefits and low environmental impact.

Aitcin (2000) estimated that, during the year 1900, 10 metric tons of cement produced globally. This quantity of cement estimated to have produced 40 metric cubic meters of concrete. The world population in 1900 was 1.65 billion. Therefore, the average consumption of concrete per person was 0.024 m^3. The estimates in 2004, puts cement production at 1700 metric tons per year (Gartner 2004), producing concrete of 6 billion m^3. That is at the average of 1 m^3 per person. Cement production has increased by 170 times, as world population has increased by 3.9 times. The average cement consumption of each person has increased from 6.25 kg per year to 268 kg per year, i.e. the average concrete consumption per person has increased by approximately forty two times. In India, the production of cement for the year 2011 – 2012 is 298 million tons. The amount of CO_2 emission from cement industry varies between 0.82 tons to 1.1 ton per ton of cement production (Malhotra).

On the other hand, the world is facing tremendous problems in terms of disposal of non-bio degradable waste products like plastics, which results in many environmental related issues. The estimated plastic waste generation in Andhra Pradesh is 28,888 tons per annum as per the 2011 records of central pollution control board, New Delhi. The disposal of this waste is another challenging problem. This is one of the main motivations to look forth for usage of non-bio degradable waste in the concrete.

1.2 OBJECTIVES OF THE STUDY

The primary and main focus of an experimental investigation is to ***study strength properties of concrete with locally available various supplementary materials for cement, aggregates***

(replacing partly) and usage of non-bio degradable waste products. More specifically, the research has the following objectives:

1. Identify the material which is suitable for substitution of cement and aggregate (partially).
2. Study the strength properties of concrete with supplementary materials for cement and aggregates.
3. Understand the behavior of concrete with non-bio degradable waste.
4. Investigate the durability properties of modified concrete.

1.3 ORGANISATION OF THESIS

The entire thesis is presented in nine chapters, including this chapter. The need for the present study and objectives are presented in **Chapter 1**.

Chapter-2 A review is made on the constituent materials including cement replacement materials and their influence on the properties of concrete. This is done to identify the most significant properties of structural concrete which would be affecting various properties of concrete. The techniques adopted by different researchers in producing different types of concretes are also discussed.

Chapter – 3 Properties of the concrete materials and specimen preparations are discussed as per the relevant IS codes. Experimental procedures and precautions used during the study are discussed as per the IS codes.

Chapter – 4 The various chemical admixtures, materials used, variables involved in concrete are discussed in detail. Three types of chemical admixtures of sulphonated naphthalene polymer are used for the study. The behavior of slump and compressive strength are studied. The behavior of these admixtures in concrete are compared with design mix of concrete without admixtures. Based on these results, the best admixture is selected for the further experimental studies.

Chapter – 5 Describes the properties of cement supplementary material (GGBS) and fine aggregate supplementary material (ROBO sand). For the optimization of GGBS as a partial replacement of cement in the concrete different combinations has been tried. After optimizing the GGBS in the concrete, different combinations of ROBO sand has been tried for the partial replacement of fine aggregate for getting final optimum mix design with GGBS and ROBO sand. The final optimum percentage of replacement levels in cement and fine aggregate are 50% of GGBS and 25% of ROBO sand respectively.

Chapter – 6 Emphases on properties of different non-bio degradable wastes used as fibers. Four types of fibers have been considered for the study i.e. HDPE, PET, HDPP and POLYESTER fibers. The different percentages of fibers varying from 0 to 6% with an increment of 0.5% are added to the concrete. The strength of concrete increases as the percentage of fiber increases up to 3.5% and then decreases as the percentage increases up to 6% of fibers.

Chapter – 7 Proposes the Stress – Strain relations of concrete without supplementary materials and with supplementary materials (GGBS, ROBO sand) and non – biodegradable waste plastic fibers. The proposed equation is in the form of $y = ((Ax + Bx^2)/(1 + Cx + Dx^2))$. A comparative study has been carried out for the different existing stress strain models of concrete like Desayi's and Krishnan's model and Saenz model.

Chapter – 8 Presents the durability properties (acid attack) concrete with and without supplementary materials and waste plastic fibers. The parameters considered in this study are weight loss, compressive strength loss, Acid Durability Factor and Acid Attack Factors. Three types of acids namely sulfuric acid, hydro chloric acid and sodium sulfate have been used for this study. The results shows that, concrete samples immersed in sulfuric acid are more affected when compared to other two acid attacks.

Chapter – 9 Summarizes the contributions made in the thesis together with a few suggestions for further research.

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CHAPTER 2
LITERATURE REVIEW

2.1 GENERAL

The following literature review has been organized with respect to three major issues: (1) supplementary materials for concrete, (2) Stress – strain behavior of concrete, and (3) Durability of concrete.

2.2 SUPPLEMENTARY MATERIALS FOR CONCRETE

Concrete is a building material composed of cement, fine aggregate, coarse aggregate and water. Supplementary Cementing Materials (SCM) has become an integral part of special concrete mix design. These may be naturally occurring materials, industrial wastes, or byproducts or the ones requiring less energy to manufacture. Some of the commonly used supplementary cementing materials are fly ash, silica fume, granulated blast furnace slag, rice husk ash and Metakaolin etc. It is being used very commonly as pozzolanic material in concrete, and has exhibited considerable influence in enhancing the mechanical and durability properties of concrete.

Mechanical properties of fiber reinforced light weight concrete composites have been studied by ***Perez-pena and Mobasher*** *(1994)*. They have used Polyvinyl Chloride (PVC), Polypropylene (PP) and alkali resistant glass as fibers in the concrete. Combinations of PP mesh/short alkali resistant (AR) glass did not show substantial improvement in the ultimate strength of the concrete. They found that significant improvement in the cracking strength and toughness of concrete by using the fibers in concrete.

Khatri et al. *(1995)* investigated the effects of different supplementary cementitious materials on the mechanical properties of High Performance Concrete (HPC). They have used fly ash (15 and 25%), silica fume (10%) and blast furnace slag (35 and 65%) with different combinations in the concrete having compressive strength 65MPa. They observed that, the effect of the addition of silica fume on early age compressive strength is more pronounced in low slag cement (35%) than in high slag cement (65%).

The authors have emphasized on compressive and flexural strength of concrete, but not studied about durability of the concrete.

Fouad Faroug et al. *(1999)* have given theoretical foundations governing the properties of concrete. The equation for fresh concrete is given as a modification of Bingham's equation. The shear resistance of the concrete mix is explained in terms of two constants—yield value and plastic viscosity. The complex nature of the yield value is described. It is pointed out that there are three different zones of behavior of fresh concrete, which arise as a result of the relation between shear stress and yield value of the mix. They also conducted experimental study that measures the effects of super plasticizer and water-to-cement (w/c) ratio of fresh concrete. The results showed that super plasticizers became less effective with increase in w/c ratio. The lower the w/c ratio, the more effective is the super plasticizer in increasing the mix workability.

The authors are mainly focused on the influence of the different types and dosages of super plasticizers on the properties of the concrete. They mainly discussed about the shear resistance and workability of concrete rather than strength of concrete.

Ganesh babu *(2000)* conducted the experiments to find out the efficiency of the Ground Granulated Blast Furnace Slag (GGBS) in concrete. They conducted the experiments with replacement levels of GGBS from 0 to 80% with 10, 30, 50, 60, 65, 70 and 80% as a partial replacement of cement in the concrete. Natural river sand and 10 to 20mm chips used in the concrete as fine aggregate and coarse aggregate respectively. They found that GGBS can be partially replaced with cement by 50 to 65%.

This paper mainly discussed the partial replacement of cement with GGBS. Authors are mainly concentrated on the compressive strength of concrete. However, not discussed about the tensile strength of the concrete, which is important for crack propagation and its durability.

Papadakis *(2002)* carried the experiments on concrete by using Supplementary Cementitious Materials (SCM) like lower calcium fly ashes, high-calcium fly ash of normal sulfur content and high-calcium fly ash of high sulfur content and natural pozzolanic materials like Milos earth (volcanic tuff from Milos Island) , diatomaceous earth (Samos Island). When the SCMs replace cement, the strength is reduced initially, but as time proceeds this gap is gradually eliminated and the strength increased.

This study is restricted to different fly ashes and volcanic ash only but not with any other materials like GGBS, etc.

Targan et al. *(2002),* conducted experimental investigation on the effects of supplementary cementing materials like bentonite, Colemanite ore Waste (CW), coal Fly Ash (FA) and coal Bottom Ash (BA) on the properties of cement and concrete. They studied systematically the effect of increasing bentonite addition (5–30%) with the constant FA (10%), BA (10%) and CW (4%) content on the properties of portland cement and concrete. They found that, with the replacement of bentonite up to 15%, the compressive strength of the concrete has significantly increased compared with control concrete at 7 days. The replacement of Portland cement beyond 15% of bentonite caused a reduction in the compressive strength. When bentonite was added to BA, CW or FA, the compressive strength of the concrete decreased with increasing bentonite content.

In this paper, authors are tried all the supplementary cementitious materials but not supplementary aggregate materials, which occupy more volume.

***Rajamane et al** (2003) and **Wang Ling et al.** (2004)* studied the properties of high performance concrete with partial replacement of cement by Ground Granulated Blast Furnace Slag (GGBS). A partial substitution of cement by GGBS eliminates higher shrinkage and greater evolution of heat of hydration, besides enhancing durability characteristics. The investigations carried out for a quantitative assessment of replacement levels of cement with GGBS on the compressive strength which were in the range of 70 MPa – 80 MPa at 28 days, and considerable imperviousness to chloride ions were obtained. They found that the compressive strength of concrete is increased from 10.2 MPa to 17.5 MPa at 28 days.

The authors focused on high strength concrete rather than normal concrete which are generally used in India.

***Gengying Li and Xiaohua Zhao** (2003)* conducted the experiments on the properties of the concrete incorporating 40% fly ash as partial replacement of cement and 25% fly ash + 15% GGBS as partial replacement of cement. The compressive strength gain from 28 days to 1 year is 23.3% with 25% fly ash and 15% GGBS whereas strength gain only with cement is 18.3%.

In this paper authors explained the compressive strength development only. This study can be extended to study the split tensile and flexural strengths also.

Choi et al. *(2005)* investigated the influence of Waste Polyethylene Terephthalate (PET) bottles Lightweight Aggregate (WPLA) on the properties of concrete. Mixture proportions of concrete were planned so that the water–cement ratios were 0.45, 0.49 and 0.53, and the replacement ratios of WPLA were 0%, 25%, 50%, and 75% by volume of fine aggregate. They reported that slump value of Waste PET bottles Lightweight Aggregate Concrete (WPLAC) increased with the increase in water–cement ratio and the replacement ratio. The improvement ratios of workability represent 52%, 104%, and 123% in comparison with that of normal concrete at the water–cement ratios of 0.45, 0.49 and 0.53, respectively. This may be attributed to not only the spherical and smooth shape but also to the absorption of WPLA. Splitting tensile strength of concrete mixtures was observed to decrease with the increase in PET aggregates; and for a particular PET aggregate content, split tensile strength increased with the reduction in w/c ratio.

In this paper, the GGBS and PET are mixed together to form a kind of aggregate and used in the concrete. A study can be made by using PET fibers as crack arresters instead of using it as aggregates.

Ramazan and Rustem *(2006)* used Blast Furnace Slag Aggregates (BFSA) as partial replacement (70%) of coarse aggregates for the production of high strength concrete. Silica fume used as micro filler and pozzolonic admixture. They found that the compressive strengths of BFSA concretes were approximately 60 to 80% higher than the controlled / traditional concrete mix. *Further study can be made on concrete with supplementary materials for cement and fine aggregates.*

Vanchai et al. *(2007)* experimentally investigated the effects of pozzolan made from various by-product materials on mechanical properties of high-strength concrete. Ground pulverized coal Combustion Fly Ash (CFA), ground fluidized Bed Combustion Fly Ash (FB), ground Rice Husk–Bark Ash (RHBA), and ground Palm Oil Fuel Ash (POFA) were used in the experiments for partial replacement of cement to produce high – strength concrete. They concluded that concrete containing CFA, FB, RHBA, and POFA can be used as pozzolanic materials in the concrete with 28-day compressive strengths higher than 80 MPa.

In this paper, four different supplementary cementitious materials are used for the production of concrete except GGBS. Authors mainly concentrated on the compressive strength of concrete only. They have not studied the split tensile and flexural strength parameters.

Job Thomas and Ananth Ramaswamy *(2007)* have studied on mechanical properties of steel fiber reinforced concrete based on the results from an experimental program and an analytical assessment of the influence of addition of fibers on mechanical properties of concrete. Models derived based on the regression analysis of 60 test data for various mechanical properties of steel fiber reinforced concrete have been presented. The various strength properties studied are cube and cylinder compressive strength, split tensile strength, modulus of rupture and post cracking performance, modulus of elasticity, poisson's ratio and strain corresponding to peak compressive stress. The variables considered are grade of concrete i.e. normal strength (35MPa), moderately high strength (65MPa) and high strength concrete (85MPa) and the volume fraction of the fiber.

In this paper, steel fibers are used to find the strengths and development of mathematical model. It will be interesting to study further replacing steel fibers with non-bio degradable waste products fibers along with other supplementary materials.

Batayneh et al. *(2007)* investigated the effect of ground plastic on the strength properties of concrete. Concrete mixes of up to 20% of plastic particles are proportioned to partially replace the fine aggregates. The addition of the plastic particles led to a reduction in the strength properties. For a 20% replacement, the compressive strength shows a sharp reduction up to 72% of the original strength. With 5% replacement the compressive strength shows a 23% reduction. Similar behavior, but in a lower effect, was observed in both the split tensile and flexural strengths of the tested samples.

This paper mainly deals with the addition of fibers (source from construction waste) in the concrete. This study emphasis that the powder forms of plastics or plastic particles reduces strength properties of concrete.

Jo et al. *(2008)* studied the effect of recycled aggregates and resins on compressive strength of concrete. In recycled-PET polymer concrete with Recycled Concrete aggregates (RPC), a gradual reduction in strength was observed as the recycled aggregate content increased. This effect was due to the weaker bond of the old mortar adhering to the recycled concrete aggregate, which may have caused a reduction in the strength of the RPC. Studying the influence of resin on properties of concrete, it was observed that the compressive strength of RPC increased with resin content. However, beyond certain resin content (approximately 13–17% resin) the strength did not change appreciably with increasing resin content. The increase in strength with the use of resin was due to the voids in the old mortar attached to the recycled aggregate.

In this paper, authors are used the recycled PET and recycled concrete aggregate in the concrete. This study is limited to usage of recycled PET not direct usage of PET in the concrete as fibers. They have not used any supplementary cementitious material in the concrete.

Ilangovan et. al. *(2008)* studied on the feasibility of the usage of quarry rock dust as hundred percent substitutes for natural sand in concrete. Mix design has been developed using different mix design methods like Indian Standards (IS), American Concrete Institute (ACI), United States Bureau of Reclamations (USBR) etc. for both conventional and quarry dust concrete. Tests were conducted on cubes and beams to study the strength and durability of concrete using quarry dust. It is found that the compressive, flexural strength and durability studies of concrete made with the quarry rock dust are nearly 10% more than the conventional concrete.

Further an experimental investigation can be made to study the concrete with other Supplementary cementitious material and non-bio degradable waste fibers in the concrete.

Papayianni and Anastasiou *(2010)* conducted laboratory trials for the production of concrete with high volume industrial by products. In this investigation, the by-products used are high-calcium fly ash and ladle furnace slag as binders and electric arc furnace slag as aggregates. Fly ash is used as 50% by mass of the total binder and ladle furnace slag as 30% by mass of the total binder. Slag aggregates are used in replacement of both fine and coarse aggregates. From this experimental investigation, it has been observed that compressive strength of concrete containing industrial by products shown an increase of 13.68% than the regular concrete.

To improve strength further, crack arrestors like fibers can be placed. Most of the materials used are industrial by products, fibers of non-bio degradable waste can be incorporated.

Sivaraja et al. *(2010)* studied the mechanical strength of fibrous concrete with waste rural materials. Steel, nylon, plastic, tyre, coir and sugarcane bagasse are used as fibers in this experimental study with volume fraction 0.5, 1.0 and 1.5% and aspect ratios are 30, 60 and 90. It is observed that, the concrete mixed with rural waste fibers improved the mechanical strength. *This study can be extended by using supplementary materials in the concrete.*

Nagabhushana and Sharada bai *(2011)* investigated the properties of mortar and concrete in which Crushed Rock Powder (CRP) is used as a partial to full replacement for natural sand. For mortar, CRP is replaced at 20% 40%, 60%, 80% and 100%. The basic strength properties of concrete were investigated by replacing natural sand by CRP at replacement levels of 20%, 30% and 40%. This study reveals that in case of cement mortars, the natural sand can be replaced by CRP. The strength of mortar containing 40% CRP is much higher than normal mortar containing only sand as fine aggregate. Though the trend in variation of compressive strength with percentage of Conditioned Crushed Rock Powder (CCRP) was found to be similar to that of CRP mortar, the strength of CCRP mortar is less than that of CRP mortars. It is better to use CRP without removing the finer particles. For lean mortar mixes, CRP can be replaced up to 100%. For rich mortar mixes, CRP can be replaced up to 40%. The compressive strength, split tensile strength and flexural strengths of concrete are not affected with the replacement of sand by CRP as fine aggregate up to 40%. ***Rajendra*** *(2013)* studied the feasibility of artificial sand in the concrete. In these experiments natural sand is replaced with artificial sand with 0 to 100% with an increment of 20%. Based on the studies, it has been observed that the natural sand can be replaced with 60 to 80% of artificial sand.

Mostafizur Rahman et.al. *(2012),* Studied the potential of recycled waste polymeric materials as a substitute for aggregates in concrete has been investigated. Two different types of waste polymer, namely Polyurethane Formaldehyde (PUF) based packaging waste and high density polyethylene were recycled and used in the experiment. Concrete and masonry poly block specimens were prepared using recycled polymer materials, and test specimens were characterized. The effect of waste polymer materials on the mechanical, physical and properties of concrete and poly blocks has been investigated. The results show that the inclusion of waste polymer materials decrease density, porosity and water absorption of concrete and poly blocks significantly.

It clearly suggest that using recycled waste polymetric materials as substitute for aggregates decreases the compressive strength of concrete, whereas the compressive strength increases if it is used as fibers to arrest cracks.

Ing Lim et al. *(2012)* carried the experimental investigation on the effect of Ground Granulated Blast Furnace Slag (GGBS) on the mechanical behavior of polyvinyl alcohol fiber reinforced engineered cementitious composites. In this study authors have used GGBS with 20 and 40% of partial replacement of cement. It is reported that the compressive strength is increased by 43% compared to the nominal mix without GGBS. This experimental study concluded that the effect of ground granulate blast furnace slag replacement not only increased the strength but also created a better fiber bridging property.

Salahaldein *(2012)* conducted the experimental investigations on the effects of super plasticizing admixture on fresh and hardened properties of concrete. In this experimental study, the percentage dosages of admixture are 0.6, 0.8, 1.0 and 1.2 in the concrete. At 28 days, the compressive strength of concrete with 1% of admixture is more than the controlled concrete i.e. without admixture. It is also reported that, when the dosages of admixture is beyond 1%, the compressive strength of concrete reduces. This phenomenon occurs since over dosage of super plasticizing admixture will cause bleeding and segregation.

Vijaya Sarathy and Dhinakaran *(2013)* on ROBO sand usage in concrete, the compressive strength results shows decreasing trend of 40, 60 and 80% replacement levels with the fine aggregate. ***Rahamat and Reza*** *(2013)* investigated the mechanical properties of concrete containing waste Glass Powder (GP) and Rice Husk Ash (RHA) as partial replacement of cement in the concrete. They performed entire experimental investigations with the combination of both glass powder and rice husk ash like 5% GP and 5% RHA, 5% GP and 10% RHA etc. and compared with the control mix having compressive strength 45MPa.They found that the optimal combination based on 28 days compressive strength is 10% GP and 5% RHA.

Arkan Radi Ali *(2013)* conducted the experiments on the use of polypropylene fibers in the concrete. 2% of fibers in the concrete were used in this study. The compressive strength is increasing as the fiber content increases in the concrete. The compressive strength of concrete has been increased by 19% and 14.3% in 7 days and 28 days respectively. It is observed that the addition of fibers enhances the mechanical performance of concrete.

From the literature, it clearly shows that, there is a need to study the usage of locally available materials to supplement (as partial replacement) for cement and aggregates in the concrete. Also an admixture is to be identified with proper dosage for concrete to attain sufficient strength and durability.

2.3 STRESS – STRAIN BEHAVIOR OF CONCRETE

A simple equation proposed for the stress strain curve for the concrete (***Desayi and Krishnan***, 1964) in compression for the both ascending and descending portion of the curve. ***Saenz et al*** (1964) discussed the on the desayi and krishnan proposed equation and they proposed another euation based on the desayi model. They found that the good agreement between the experimental values and predicted values from the equation.

Empirical models of stress-strain relationships for normal weight high strength concrete (HSC) can be divided into two categories according to their expressional forms. One category is based on equations suggested by Popovics (1973) and the other category uses a form of equations proposed by Sargin et al. (1971), which are, respectively, listed in Tables 2.1 and 2.2.

Table: 2.1 Stress-Strain Models for Concrete Based on Popovics model

Researchers	Equations
Popovics (1973)	$f_c = \frac{f_c' \beta(\varepsilon/\varepsilon_o)}{[\beta - 1 + (\varepsilon/\varepsilon_o)^{\beta}]}$ $where\ \varepsilon_0 = 735(f_c')^{0.25} \times 10^{-6};\ \beta = 1.0 + 0.058 f_c'$
Tomaszewicz (1984)	$f_c = \frac{f_c' \beta(\varepsilon/\varepsilon_o)}{[\beta - 1 + (\varepsilon/\varepsilon_o)^{\beta}]} \quad 0 \le \varepsilon \le \varepsilon_o\ ;\quad f_c = \frac{f_c' \beta(\varepsilon/\varepsilon_o)}{[\beta - 1 + (\varepsilon/\varepsilon_o)^{k\beta}]} \quad \varepsilon > \varepsilon_o$ $where\ \varepsilon_0 = 700(f_c')^{0.31} \times 10^{-6};\ \beta = 8.32/[8.32 - (f_c')^{0.475}]\ ; k = f_c'/20$
Carreira and Chu (1985)	$f_c = \frac{f_c' \beta(\varepsilon/\varepsilon_o)}{[\beta - 1 + (\varepsilon/\varepsilon_o)^{\beta}]} where\ \varepsilon_0 = (1.680 + 7.1 f_c') \times 10^{-6};$ $\beta = 1/[1 - (f_c'/\varepsilon_o E_{it})] \quad and\ \ E_{it} = 0.0736 w^{1.51} (f_c')^{0.3}$

Hsu and Hsu (1994)	$f_c = \frac{nf_c'\beta(\varepsilon/\varepsilon_o)}{[n\beta - 1 + (\varepsilon/\varepsilon_o)^{n\beta}]}$ $where\ \beta = (f_c'/65.23)^3 + 2.59 \quad 0 \le \varepsilon \le \varepsilon_d$ $for\ 0 \le \varepsilon \le \varepsilon_0, n = 1; for\ \varepsilon_0 \le \varepsilon \le \varepsilon_d, n = 1\ if\ 0 < f_c' < 62\ MPa;$ $n = 2\ if\ 62 \le f_c' < 76\ MPa; n = 3\ if\ 76 \le f_c' < 90\ MPa\ and$ $n = 5\ if\ f_c' \ge 90\ MPa$ $\varepsilon > \varepsilon_d, \quad f_c = 0.3f_c' exp[-0.8(\varepsilon/\varepsilon_o - \varepsilon_d/\varepsilon_o)^{0.5}]$
Wee et al. (1996)	$0 \le \varepsilon \le \varepsilon_0, \ f_c = \frac{f_c'\beta(\varepsilon/\varepsilon_o)}{[\beta - 1 + (\varepsilon/\varepsilon_o)^{\beta}]}$ $\varepsilon > \varepsilon_0, \ f_c = \frac{k_1 f_c'\beta(\varepsilon/\varepsilon_o)}{[k_1\beta - 1 + (\varepsilon/\varepsilon_o)^{k_2\beta}]}$ $where\ \beta = 1/[1 - f_c'/\varepsilon_o E_{it}]$ $\varepsilon_o = 780(f_c')^{\frac{1}{4}} \times 10^{-6};\ E_{it} = 10200(f_c')^{\frac{1}{3}};$ $k_1 = (50/f_c')^3;\ k_2 = (50/f_c')^{1.3}$

Table: 2.2 Stress-Strain Models for concrete based on Sargin et al. model

Researchers	Equations
Sargin et al. (1971)	$f_c = \frac{f_c'[A(\varepsilon/\varepsilon_0) + (D-1)(\varepsilon/\varepsilon_0)^2]}{[1 + (A-2)(\varepsilon/\varepsilon_0) + D(\varepsilon/\varepsilon_0)^2]}$ $A = E_{it}\varepsilon_0/f_c'\ ; D = 0.65 - 7.25f_c' \times 10^{-3};\ E_{it} = 5975\sqrt{f_c'}\ ;\ \varepsilon_0 = 0.0024$
Wang et al. (1978)	$f_c = f_c' \left\{ \frac{A(\varepsilon/\varepsilon_0) + B(\varepsilon/\varepsilon_0)^2}{1 + C(\varepsilon/\varepsilon_0) + D(\varepsilon/\varepsilon_0)^2} \right\}$
Van Gysel and Taerwe (1996)	$0 \le \varepsilon \le \varepsilon_{max}, \ f_c = \frac{f_c'[(E_{it}/E_0)(\varepsilon/\varepsilon_0) - (\varepsilon/\varepsilon_0)^2]}{[1 + (E_{it}/E_0 - 2)(\varepsilon/\varepsilon_0)]}$ $\varepsilon > \varepsilon_{max}, \quad f_c = f_c'/\{1 + [(\varepsilon/\varepsilon_0 - 1)/(\varepsilon_{max}/\varepsilon_0 - 1)]^2\}$

	$\varepsilon_{max} = \varepsilon_0 \left\{ \left[\frac{[E_{it}/(2E_0)+1]}{2} + \frac{[E_{it}/(2E_0)+1]^2}{4} - \frac{1}{2} \right]^{1/2} \right\}$ $E_{it} = 21500\alpha_E(f_c'/10)^{1/3};\ \ \varepsilon_0 = 700(f_c')^{0.31} \times 10^{-6}$

$(f_c\,, \varepsilon)$ = coordinates of any point in the stress-strain curve

f_c' = unconfined concrete compressive strength

E_{it} = initial tangent modulus of elasticity; E_o = secant modulus at peak stress

k, k_1, and k_2 = correction factors; n, β = material parameters; w = concrete unit weight;

α_E = coarse aggregate coefficient (1.20 for basalt, dense limestone aggregates, 1.00 for quartzite aggregates, 0.90 for limestone aggregates and 0.70 for sandstone aggregates)

ε_d = strain corresponds to a stress value of 0.3 f_c' in the descending part of the stress-strain curve;

ε_{max} = Concrete strain when concrete stress is equal to 0.5 f_c' on the descending part of the stress - strain curve;

ε_0 = Peak strain of unconfined concrete strength f_c' and ξ = Coefficient.

The complete stress strain behavior of steel fiber reinforced concrete studied by ***Sameer et al*** (1992) the concrete compressive strength ranging from 35 MPa to 85 MPa. In the experimental investigations, three fiber volume fractions of 30 kg/m^3, 45 kg/m^3, and 60 kg/m^3 and three aspect ratios of 60, 75, and 100 are investigated. They have proposed a simple equation to predict the complete stress strain curve and the proposed equation provides a good correlation between predicted and experimental results.

$$\frac{f_c}{f'_{cf}} = \frac{\beta \frac{\varepsilon_c}{\varepsilon_{of}}}{\beta - 1 + \left(\frac{\varepsilon_c}{\varepsilon_{of}}\right)^{\beta}}$$

Where f'_{cf}= compressive strength of fiber concrete

ε_{of}= strain corresponding to the compressive strength

f_c , ε_c= stress and strain values on the curve

$\beta = 1.093 + \frac{0.074 E_{inf}}{10^4}$ $E_{inf} = 10^4[9.610(RI)^{-0.926}]$ RI = Reinforcing Index

E_{inf} = Slope at the inflection point of descending branch of the stress strain curve

Mansur et al (1997) conducted experiments to predict the stress strain behavior of confined high strength and fiber concrete. The test parameters include tie diameter, tie spacing, concrete core area, and casting direction of specimens. The results indicate that the initial tangent modulus and initial Poisson's ratio of the concrete are not affected by confinement. The confinement enhances both the peak stress and strain at peak stress. The confined fiber concrete exhibits larger strain at peak stress and have higher post peak ductility. Based on test data, an analytical model is proposed to generate the complete stress-strain curves of high-strength concrete confined by lateral ties. The proposed model has been found to agree well with the stress-strain curves generated experimentally. The ascending portion of curve is given by

$$f = f_o^{"} \left\{ \frac{\beta \frac{\varepsilon}{\varepsilon_o^{"}}}{\beta - 1 + \left(\frac{\varepsilon}{\varepsilon_o^{"}}\right)^{\beta}} \right\} \quad where \;\; \beta = \frac{1}{1 - \frac{f_o^{"}}{\varepsilon_o^{"} E_{it}}}$$

Where f and ε = concrete stress and strain; and β = material parameter

$f_o^{"}$= Strength of confined concrete $\varepsilon_o^{"}$= Strain at peak stress of confined concrete

Similarly for the descending portion of curve is given with two correction factors k1 and k2

$$f = f_o^{"} \left\{ \frac{k_1 \beta \frac{\varepsilon}{\varepsilon_o^{"}}}{k_1 \beta - 1 + \left(\frac{\varepsilon}{\varepsilon_o^{"}}\right)^{k_2 \beta}} \right\} \quad where \quad \beta = \frac{1}{1 - \frac{f_o^{"}}{\varepsilon_o^{"} E_{it}}}$$

For confined non-fiber concrete k1 and k2 is given by

$$k_1 = 2.77 \left\{\frac{\rho_s f_y}{f_0}\right\} \quad and \quad k_2 = 2.19 \left\{\frac{\rho_s f_y}{f_0}\right\} + 0.17$$

For confined fiber concrete is given by

$$k_1 = 3.33 \left\{\frac{\rho_s f_y}{f_0}\right\} + 0.12 \quad and \quad k_2 = 1.62 \left\{\frac{\rho_s f_y}{f_0}\right\} + 0.35$$

ρ_s is the volumetric ratio of confining steel; and f_0 is the peak stress of unconfined concrete.

Zhao-Hui Lu and Yan-Gang Zhao (2010), developed a new empirical model with emphasis on the softening branch is proposed to generate the complete stress-strain relationship for high strength concrete based on the published experimental data. The proposed equation is given by

$$f_c = \frac{f_c' [\,(E_{it}/E_0)(\varepsilon/\varepsilon_0) - (\varepsilon/\varepsilon_0)^2]}{[1 + (E_{it}/E_0 - 2)(\varepsilon/\varepsilon_0)]}$$

Where $(f_c\,, \varepsilon)$ = coordinates of any point in the stress-strain curve;

f_c' = Unconfined concrete compressive strength; E_{it} = initial tangent modulus of elasticity;

E_0 = secant modulus at peak stress $E_0 = f_c'/\varepsilon_0$;

ε_0 = peak strain of unconfined concrete strength f_c'

Giriprasad *(2012)* conducted the experiments on stress strain behavior of high strength self-compacting concrete. The proposed equation is in the form of $y = Ax/(1 + Bx^2)$

From the studies on stress - strain behavior of concrete, there is need to develop the stress - strain relationships for the proposed concrete mix with supplementary materials as partial replacement of cement and fine aggregate along with fibers.

2.4 DURABILITY OF CONCRETE

An important engineering property of concrete is durability. It determines the service life of concrete structures significantly. Durability of concrete is very important especially when it is exposed to marine environments, sulphate and hydrochloric acids etc.

Vladimir Zivica and Adolf Bajza (2001, 2002 and 2004) clearly said the principle of acid attack, factors of rate of acidic attack and protection measures and Methods of testing. ***Al-Tamimi and Sonebi*** (2003) studied the self-compacting concrete exposed to acidic solutions. They investigated the acid resistance of Self-Compacting Concrete (SCC) and Conventional Concrete (CC), immersed up to 18 weeks at 20°C in sulfuric and hydrochloric acid solutions. They found that the SCC performed better than the conventional concrete in sulfuric solution but was slightly more vulnerable to hydrochloric acid attack compared to conventional concrete.

Vladimir Zivica (2006) studied the long-term effect on the action of various organic substances on the cement-based material. The result shows the different aggressiveness of solutions of phenols, carboxylic acids and sulphonic compounds. Petroleum and mineral oil have been shown as unaggressive media. It is observed that increased expansions followed by crack propagation at the action of acidic media were adjudged to the cooperation of the mechanism of acidic attack and dispersive effect of surface activity of organic substances.

Hanifi Binici (2006) investigated the sulfate resistance of plain and blended cement using GGBS and natural pozzolan. It was observed that the sulfate resistances of blended cements were significantly higher both against sodium sulfate and magnesium sulfate attacks than references cement. Final strength reductions for finer mixes attacked by magnesium sulfate were marginally lower than those attacked by sodium sulfate.

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Nabil (2006) studied the durability of Metakaolin (MK) concrete to sulphate attack. The degree of sulfate attack was evaluated by measuring expansion of concrete prisms, compressive strength reduction of concrete cubes, and visual inspection of concrete specimens to cracks. The study showed that MK replacement of cement increased the sulfate resistance of concrete. The sulfate resistance of MK concrete increased with increasing the MK replacement level. The sulfate resistance of MK concrete at water binder ratio (w/b) of 0.5 was found higher than that at w/b ratio of 0.6.

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Serdar Aydın et al (2007) conducted the experiments on sulfuric acid resistance of high volume fly ash concrete. Under standard curing conditions, the strength values of high-volume fly ash concretes were satisfactory. Test results indicate that sulfuric acid resistance of steam-cured concrete has improved significantly by incorporation of fly ash and long-term strength values decreased significantly for concrete mixtures.

Murthi and Sivakumar (2008) conducted the experiments on acid resistance of ternary blended concrete (20% fly ash and 8% silica fume) immersed up to 32 weeks in sulfuric acid and hydro chloric acid. They found that the ternary blended concrete prepared by 20% fly ash and 8% silica fume performed better acid resistance than the ordinary plain concrete and binary blended concrete.

Martin *(2012)* studied the performance of concrete incorporating GGBS in aggressive waste water environments. The cement has been replaced by 50 and 70% with GGBS in this experimental study. Sulfate expansion study and sulfuric acid tests are conducted in this study. It was concluded in respect of sulfate attack that resistance of Portland cement binders is greatly enhanced by the use of high quantities of GGBS.

Bassel Hanayneh et al. (2012) investigated the effect of micro silica, water proofer and super plasticizer on the durability of concrete to phosphoric acid attack. The degree of acid attack was evaluated by measuring the percentage changes in weight of concrete cubes. The results showed that the combined effect of micro silica and water proofer was the best to enhance the durability of concrete to phosphoric acid attack.

Sesha Phani et al (2013), conducted experiment on the effects of mineral admixtures on the durability properties of high strength self-compacting concrete. They have prepared 100mm cubes and immersed in the 10% acidic solution. The experimental result shows that, there is considerable weight loss and compressive strength loss of concrete cubes immersed in acidic solution.

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Sunil pratap reddy et al (2010, 2013), investigated the durability performance of bacterial concrete. Durability studies reveal the percentage weight reduction and percentage strength loss when cubes are immersed in 5% HCl indicating that bacterial concrete has less weight reduction and strength loss than the conventional concrete.

Based on the above studies it is desirable to have an experimental investigation, on durability of the concrete, whether it is normal or modified concrete, having partial or full supplementary materials for cement and aggregates. Further the concrete with fibers of non-bio degradable waste to be checked for acid attacks.

2.5 NEED FOR THE STUDY

From the entire literature, there is a scope to look after the supplementary materials for cement, fine aggregate and coarse aggregate in the concrete as a partial replacement. If these supplementary materials are from industrial waste, then this will reduce the environmental concerns also. Based on the above study the following objectives are identified to carry an experimental investigation and analysis.

(i) Identify the material from industrial wastes which can be partially replaced with cement and fine aggregate.

(ii) Detailed study of strength properties of concrete with these supplementary materials.

(iii) Formulation of stress- strain behavior of concrete with supplementary materials.

(iv) Durability studies of concrete with supplementary materials.

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CHAPTER 3

MATERIAL PROPERTIES

3.1 INTRODUCTION

Concrete is a highly durable and most frequently used human-made material in the world. Concrete is a conglomerate material composed of three essential elements. The three elements include some type of cementitious material, most often – cement, water, with which the cementitious material will react and fillers or some material which will occupy considerable volume, most often aggregate. Concrete is inexpensive, readily moulded into complicated shapes and has adequate compressive strength and stiffness.

There are two varieties of concretes namely Plain Cement Concrete (PCC) and Reinforced Cement Concrete (RCC). The ingredients of Plain Cement Concrete are cement, aggregate and water. PCC is very good in compression but weak in tension. To overcome this, structural members are provided with reinforcement for example steel bars This chapter mainly focused on the material properties used in the concrete, mix design, properties of fresh and hardened concrete.

3.2 MATERIALS

Cement, fine aggregate (sand), coarse aggregate water and admixtures are the essential ingredients of the concrete.

3.2.1 Cement: Cement is a binding material in the concrete. The main functions of cement are

1. Fill up the voids existing in the fine aggregate and make concrete impermeable.
2. It provides the strength to the concrete on setting and hardening.

3. It binds the aggregates into solid mass by virtue of its setting and hardening properties when mixed with water.

There are wide varieties of cements available in the market and each type is used under certain conditions due its special properties.

The use of additives, changing chemical composition and use of different raw materials have resulted in the availability of many types of cements to cater the need of the construction industries for the specific purposes. These cements are classified as Portland cements and non-Portland cements. The distinction is mainly based on the method of manufacture (Shetty, 2008).

Types:

1. Ordinary Portland cement (33, 43 and 53 grades)
2. Rapid hardening cement (IS 8041: 1990)
3. Sulphate resisting cement (IS 12330:1988)
4. Portland slag cement (IS 455:1989)
5. Portland Pozzolana Cement (PPC) (IS 1489:1991)
6. High alumina cement (IS 6452: 1989)
7. Concrete sleeper grade cement (IRS-T 40: 1985)

ASTM Classification of Cement:

As per American Society for Testing Materials (ASTM) the cement is designated as Type I to V (ASTM C150).

Type I: used for the general construction purposes

Type II: used concrete construction is exposed to moderate sulphate action or moderate heat of hydration is required.

Type III: used to get high early strength

Type IV: Used when low heat of hydration is required

Type V: Used when high sulphate resistance required

Ordinary Portland Cement (OPC) of 43 grade is used in the experimental analysis. Cement is procured from Zuari industries. The physical and chemical properties of four different samples of cement are given in the tables 3.1 to 3.3.

Table: 3.1 Physical and Chemical Properties of Cement Sample No.1

Physical properties #	**Test Results**	**Limits as per IS 8112 - 1989**
Fineness (m^2/Kg) (Specific Surface)	296	225 minimum
Initial Setting Time (minutes)	140	30 minimum
Final Setting Time (minutes)	245	600 maximum
Soundness By Le-chatelier By Auto Clave	 1.50 mm 0.04%	 10mm maximum 0.8% maximum
Compressive Strength (MPa) 3 days 7 days 28 days	 32 41.3 59	 23 minimum 33 minimum 43 minimum
Chemical Properties #		
LSF $\frac{Cao-0.7SO_3}{2.8SiO_2+1.2Al_2O_3+0.65Fe_2O_3}$	0.89	0.66 to1.02 Max
AM (Al_2O_3/Fe_2O_3)	1.26	0.66 minimum
Insoluble Residue (% by Mass)	1.20	3
Magnesia (% by Mass)	1.30	6 maximum
Sulphuric Anhydrate (By Mass)	2.18	3 maximum
Total Loss in Ignition (%)	1.56	5 maximum
Total Chlorides (%)	0.012	0.05 maximum

Courtesy ZUARI Industries

Table: 3.2 Physical and Chemical Properties of Cement Sample No.2

Physical properties #	**Test Results**	**Limits as per IS 8112 - 1989**
Fineness (m^2/Kg) (Specific Surface)	298	225 minimum
Initial Setting Time (minutes)	140	30 minimum
Final Setting Time (minutes)	245	600 maximum
Soundness By Le-chatelier By Auto Clave	 3.5 0.05%	 10mm maximum 0.8% maximum
Compressive Strength (MPa) 3 days 7 days 28 days	 33 43 59.3	 23 minimum 33 minimum 43 minimum
Chemical Properties #		
LSF $\frac{Cao-0.7SO_3}{2.8SiO_2+1.2Al_2O_3+0.65Fe_2O_3}$	0.90	0.66 to1.02 Max
AM (Al_2O_3/Fe_2O_3)	1.20	0.66 minimum
Insoluble Residue (% by Mass)	0.92	3
Magnesia (% by Mass)	1.30	6 maximum
Sulphuric Anhydrate (By Mass)	1.86	3 maximum
Total Loss in Ignition (%)	1.46	5 maximum
Total Chlorides (%)	0.012	0.05 maximum

Courtesy ZUARI Industries

Table: 3.3 Physical and Chemical Properties of Cement Sample No.3

Physical properties #	**Test Results**	**Limits as per IS 8112 - 1989**
Fineness (m^2/Kg) (Specific Surface)	294	225 minimum
Initial Setting Time (minutes)	160	30 minimum
Final Setting Time (minutes)	275	600 maximum
Soundness By Le-chatelier	 2.0	 10mm maximum

By Auto Clave	0.05%	0.8% maximum
Compressive Strength (MPa) 3 days 7 days 28 days	 32 42 56.8	 23 minimum 33 minimum 43 minimum
Chemical Properties #		
LSF $\frac{Cao-0.7SO_3}{2.8SiO_2+1.2Al_2O_3+0.65Fe_2O_3}$	0.90	0.66 to1.02 Max
AM (Al_2O_3/Fe_2O_3)	1.24	0.66 minimum
Insoluble Residue (% by Mass)	1.2	3
Magnesia (% by Mass)	1.34	6 maximum
Sulphuric Anhydrate (By Mass)	1.92	3 maximum
Total Loss in Ignition (%)	1.86	5 maximum
Total Chlorides (%)	0.012	0.05 maximum

Courtesy ZUARI Industries

3.2.2 Aggregates: Aggregates are important constituents of concrete. The aggregates occupy 70 to 80% of the volume of concrete and their impact on various characteristics and properties of concrete is considerable. Aggregates can be classified based on sources like natural and artificial aggregates. The natural aggregates are sand, gravel, crushed rock such as granite, quartzite, basalt etc. whereas the artificial aggregates are broken brick, air cooled slag, sintered fly ash, bloated clay etc. The aggregate can also be classified on the basis of the size of the aggregates as Fine aggregate and Coarse aggregate. The size of the aggregate bigger than 4.75 mm is considered as Coarse Aggregate and aggregate whose size is 4.75 mm and less is considered as Fine aggregate.

3.2.2.1 Fine aggregate: Generally river sand is considered as fine aggregate in the concrete. Sand consists of small angular/rounded grains of silica. The main functions of the fine aggregates are:

1. Sand fills the voids existing in the coarse aggregate.
2. It reduces the shrinkage and cracking of concrete.
3. By varying the proportion of sand, concrete can be prepared economically for required strength.
4. Sand helps in hardening of cement by allowing the water through its voids.
5. To form hard mass of silicates, due to some chemical reaction between silica of sand and the constituents of cement.

The following precautions must be take care while selecting the aggregate:

1. It should consist of coarse, angular, sharp and hard grains.
2. It must be clean and free from coatings of clay and silt.
3. It should not contain any organic matter.
4. It should be free from hygroscopic salts.
5. It should be chemically inert.
6. It must be strong and durable.
7. The size of the sand grains should pass through 4.75mm IS sieve and should be entirely retained on 75 micron IS sieve.

Locally available river sand from Karimnagar, Andhra Pradesh, which is free from organic impurities, is used in the experiments. The specific gravity of sand is 2.62 and water adsorption is 0.3%. The sieve analysis results of three different samples of sand are given in tables 3.4 to 3.6.

Quantity of sample 1: 1000gm

Table: 3.4 Sieve Analysis of Fine Aggregate Sample No.1

IS Sieve	Weight retained	%of weight retained	Cumulative % of weight retained	% of passing	Limits as per IS 383-1970 IS 2386-1963
10	0	0	0	100	100
4.75	47	4.7	4.7	95.3	90 – 100
2.36	89	8.9	13.6	86.4	75 – 100
1.18	123	12.3	25.9	74.1	55 – 90
600	303	30.3	56.2	43.8	35 – 59
300	241	24.1	80.3	19.7	8 – 30
150	173	17.3	97.6	2.4	0 – 10
Total cumulative % of weight retained			278.3		

Results: The sample is in zone II within limits
Fineness modulus: Total cumulative % of weight retained/100 = 2.78

Quantity of sample 2: 1000 gm

Table: 3.5 Sieve Analysis of Fine Aggregate Sample No.2

IS Sieve	Weight retained	%of weight retained	Cumulative % of weight retained	% of passing	Limits as per IS 383-1970 IS 2386-1963
10	0	0	0	100	100
4.75	66.4	6.64	6.64	93.36	90 – 100
2.36	126.1	12.61	19.25	80.75	75 – 100
1.18	122.5	12.25	31.5	68.5	55 – 90
600	235	23.5	55	45	35 – 59
300	326	32.6	87.6	12.4	8 – 30
150	124	12.4	100	0	0 – 10
Total cumulative % of weight retained			299.99		

Results: The sample is in zone II within limits
Fineness modulus: Total cumulative % of weight retained/100 = 2.99

Quantity of sample: 1000 gm

Table: 3.6 Sieve Analysis of Fine Aggregate Sample No.3

IS Sieve	Weight retained (gm)	% of weight retained	Cumulative % of weight retained	% of passing	Limits as per IS 383-1970 IS 2386-1963
10	0	0	0	100	100
4.75	11	1.1	1.1	98.9	90 – 100
2.36	66	6.6	7.7	92.3	75 – 100
1.18	287	28.7	36.4	63.6	55 – 90
600	379	37.9	74.3	25.7	35 – 59
300	230	23.0	97.3	2.7	8 – 30
150	20	2.0	99.3	0.7	0 – 10
Total cumulative % of weight retained			316.10		

Results: The sample is in zone II within limits
Fineness modulus: Total cumulative % of weight retained/100 = 3.16

3.2.2.2 Coarse aggregate: Crushed stone, gravel and broken bricks are some of the materials used as coarse aggregate in the concrete depending on the situation. The functions of the coarse aggregate is given below

1. It makes solid and hard mass of concrete with cement and sand.
2. Coarse aggregate increases the crushing strength of concrete.
3. It reduces the cost of concrete, since it occupies major volume.

The coarse aggregates used in this experimental investigation are 20 mm & 12 mm size, crushed and angular in shape. The aggregates are free from dust. The specific gravity of coarse aggregate is 2.65 and water adsorption is 0.3%. The sieve analysis results of three different samples of coarse aggregate are given in tables 3.7 to 3.9.

Quantity of sample 1: 5000 gm (20 mm size)

Table: 3.7 Sieve Analysis of Coarse Aggregate Sample No.1

IS Sieve	Weight retained	% weight retained	Cumulative % weight retained	% passing	Limits as per IS 383 – 1970 IS 2386 – 1963
80	0	0	0	100	100
40	0	0	0	100	100
20	936	18.72	18.72	81.2	85– 100
10	4044	80.88	99.6	0.4	0 – 20
4.75	20	0.4	100	0	0 – 5
2.36	0	0	100	0	0
1.18	0	0	100	0	0
600	0	0	100	0	0
300	0	0	100	0	0
150	0	0	100	0	0
Total cumulative % of weight retained			718.32		

Quantity of sample 2: 5000 gm (20 mm size)

Table: 3.8 Sieve Analysis of Coarse Aggregate Sample No.2

IS Sieve	Weight retained	%of weight retained	Cumulative % of weight retained	% of passing	Limits as per IS 383 – 1970 IS 2386 – 1963
80	0	0	0	100	100
40	0	0	0	100	100
20	813	16.26	16.26	83.74	85 – 100
10	4152	83.04	99.3	0.7	0 – 20

4.75	35	0.7	100	0	0 – 5
2.36	0	0	100	0	0
1.18	0	0	100	0	0
600	0	0	100	0	0
300	0	0	100	0	0
150	0	0	100	0	0
Total cumulative % of weight retained			715.56		

Quantity of sample 3: 5000 gm (20 mm size)

Table: 3.9 Sieve Analysis of Coarse Aggregate Sample No.3

IS Sieve	Weight retained	% weight retained	Cumulative % weight retained	% passing	Limits as per IS 383 – 1970 IS 2386 – 1963
80	0	0	0	100	100
40	0	0	0	100	100
20	790	15.8	15.8	84.2	85– 100
10	3970	79.4	95.2	4.8	0 – 20
4.75	240	4.8	100	0	0 – 5
2.36	0	0	100	0	0
1.18	0	0	100	0	0
600	0	0	100	0	0
300	0	0	100	0	0
150	0	0	100	0	0
Total cumulative % of weight retained			711		

3.2.3 Water: Water plays a vital role in the mixing, laying, compaction, setting and hardening of concrete. The strength of concrete directly depends on the quantity and quality of water used in the mix. The main uses of the water in the concrete are listed below:

1. Water wets the surface of the aggregates.
2. It facilitates the spreading of cement over the fine aggregate.
3. Water acts as a lubricant for the aggregate and makes the mix workable.
4. Water is only the ingredient that reacts chemically with cement (hydration of cement) and thus setting and hardening of cement takes place.

The following precautions were taken while selecting the water

1. Water should be fresh and clean.
2. It should be free from organic impurities, harmful salts, greasy and oil substances.

Water samples were collected from bore well (BITS, Pilani – Hyderabad Campus) and its properties are shown in tables 3.10 and 3.11.

Table: 3.10 Water Sample 1 Test Results

S. No.	Parameter	Results	Limits as per IS 456 – 2000
1	pH	6.3	6.5 – 8.5
2	Chlorides (mg/l)	45	2000 (PCC); 500 (RCC)
3	Alkalinity (ml)	6	< 25
4	Sulphates (mg/l)	105	400
5	Florides (mg/l)	0.04	1.5
6	Organic Solids (mg/l)	43	200
7	Inorganic Solids (mg/l)	115	3000

Table: 3.11 Water Sample 2 Test Results

S. No	Parameter	Results	Limits as per code IS 456 – 2000
1	pH	6.33	6.5 – 8.5
2	Chlorides (mg/l)	30	2000 (PCC); 500 (RCC)
3	Alkalinity (ml)	5	< 25
4	Sulphates (mg/l)	121	400
5	Florides (mg/l)	0.02	1.5
6	Organic Solids (mg/l)	40	200
7	Inorganic Solids (mg/l)	120	3000

3.3 MIX DESIGN

Mix design is the process of selecting suitable ingredients of the concrete and determining their proportions with object of producing concrete of certain maximum strength and durability as economical as possible. The concrete mix is designed as per IS 10262 – 2009, IS 456-2000 and SP 23. Target mean strength for M25 grade concrete is 33.25 N/mm^2. Target mean strength for M30 grade concrete is 38.25 N/mm^2. Table – 3.12 and 3.13 represents the mix proportion quantities for one cubic meter and cement bag.

Table: 3.12 Mix Proportion Quantities M25 concrete

Mix Constituents	For one cum of concrete (Kg)
Cement	327
Water	147
Fine aggregate	723
Coarse aggregate	1246

Admixture	3.270
Water cement ratio	0.45
Workability in mm after 45 min	85
3days average compressive strength (N/mm^2)	23.41
7days average compressive strength (N/mm^2)	29.59
28days average compressive strength (N/mm^2)	36.3

Table: 3.13 Mix Proportion Quantities M30 concrete

Mix Constituents	For one m^3 of concrete (kg)
Cement	350
Water	147
Fine aggregate	704
Coarse aggregate	1245
Admixture	3.5
Water cement ratio	0.42
Workability in mm after 45 min (Slump cone)	90
3days average compressive strength (N/mm^2)	25.21
7days average compressive strength (N/mm^2)	31.11
28days average compressive strength (N/mm^2)	38.81

3.4 EXPERIMENTAL PROGRAM

The proportioning of quantity of both cement and aggregate is done by weight as per the concrete mix design. The water and the admixture are measured by volume. All measuring equipments were maintained in clean serviceable condition with their accuracy periodically checked. The workability tests are carried out immediately after mixing of concrete using the slump cone test. The specimens are used according to the specification laid down in IS 516:1959. Standard cast iron cube moulds of size 150x150x150mm, cylinder moulds of size 150X300mm and beam size for flexure test is 100X100X500mm are used in the preparation of test specimens. The moulds have been cleaned to remove dust particles and applied with mineral oil on all sides before the concrete is poured into the mould. The admixture is mixed with the constituents of concrete at the time of adding water. Full blending of the admixture and the concrete is ensured by mixing for a period of at least two minutes. Thoroughly mixed concrete is filled into the mould and compacted in three equal layers. Excess concrete is removed with trowel after proper compaction and top surface is smoothened. Overdose may also cause increase in air entrainment, which will tend to reduce the strength of the mix. After casting, the specimens are stored in the laboratory with room temperature for 24 hours from the time of addition of water to the ingredients. After this period, the specimens are removed from the moulds and immediately submerged in the clean and fresh water tank. The specimens are cured for 28 days.

3.5 PROPERTIES OF HARDENED CONCRETE

The properties of hardend concrete depends on the mix proportions, curing conditions and environment. The strength of concrete is basically referred to compressive strength of concrete and it depends on cement paste strength, interfacial bonding and aggregate strength. This strength can be affected by the water cement ratio, type of ingredients, mix proportions, curing, temperature and age of concrete.

3.5.1 Compressive strength of concrete: Compressive strength is one of the most important and useful properties of concrete in most of the structural applications. Compressive strength of concrete is calculated by dividing load by area of the specimens. Generally cube size 150X150X150 mm size samples are used for finding the compressive strength.

$$f_c = P/a$$

Where f_c = Compressive strength of concrete

P = Compressive load at failure in N or KN

a = Cross sectional area of the cube in mm^2

3.5.2 Split tensile strength of concrete: Direct tension tests of concrete are seldom carried out, mainly because the specimen holding device introduce secondary stresses that cannot be ignored. The most commonly used tests for estimating the tensile strengths of concrete are the ASTM C496 (IS 5816 – 1999) splitting tensile strength. The 150Φ and 300mm long cylindrical specimen is subjected to compression loads along two axial lines which are diametrically opposite. The load is applied continuously at a constant rate until the specimen fails. The compressive stress produces a transverse tensile stress, which is uniform along the vertical diameter. The splitting tensile strength is computed by the formula:

$$f_t = \frac{2P}{\pi l d}$$

Where f_t = Split tensile strength of concrete

P = Compressive load at failure in N or KN

d = Diameter of the cylinder specimen in mm

l = Length of the cylinder specimen in mm

3.5.3 Flexural strength of concrete: The flexural strength is more important than the compressive strength in the design of concrete mixes to be used in the construction of roads and airport pavements. The flexural strength of concrete is determined by subjecting a plain concrete beam to flexure under transverse loads. The theoretical maximum tensile stress reached in the bottom fiber of a standard test beam is often referred to as the modulus of rupture. The magnitude of which depends on the dimensions of the beam and the type of loading. The beam is tested using load frame of 20 KN capacity. The bed of the testing machine is provided with two steel rollers on which the specimen is supported. The distance between these rollers is kept at 40cm. The load is applied through two similar rollers mounded at the third point of the supporting span that are spaced 13.33 cm apart. The load is applied to the two rollers through another roller contact with the top face of the machine.

The specimen is placed in such a manner that the load is applied without shock. The axis of the specimen is carefully aligned with the axis of the loading frame. The rate of loading 180kg/min i.e. extreme fiber stress increases at approximately 0.07 $kg/mm^2/min$. The load is divided equally between the two loading rollers. The load is increase until the specimen fails and the maximum load applied at failure is calculated.

Also the central deflections are noted with the help of a deflectometer until failure of the specimen corresponding to the loads at an interval of 20 divisions on proving ring.

$$Flexural\ strength\ (f_{cr}) = Pl/bd^2\ \ when\ a = 13.33cm\ to\ 26.6cm$$

$$Flexural\ strength\ (f_{cr}) = 3Pa/bd^2\ \ when\ a = 11.0cm\ to\ 13.33cm$$

Where P = Maximum load at failure in N or KN

l = Length of the specimen in mm (50 cm)

b and d = breadth and depth of specimens (10 cm X 10 cm)

a = Distance between the line of fracture and the nearer support, measure on the center line of the tensile side of the specimen.

Fig.3.1 Concrete cube during testing

Fig. 3.2 Concrete in flexural strength

Fig. 3.3 Concrete cubes after Failure

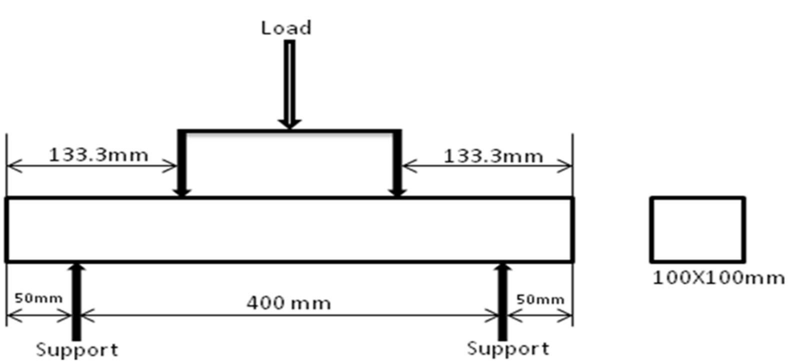

Fig: 3.4 Experimental setup for the flexure test

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CHAPTER 4
CONCRETE WITH CHEMICAL ADMIXTURES

4.1 INTRODUCTION

Admixtures are chemical compounds in the form of powder or fluids that are added to the concrete to get certain characteristics not obtainable with plain concrete mixes. Admixtures are added to the concrete mix before or during mixing to modify one or more properties of fresh and/or hardened concrete. The properties of concrete commonly modified are rate of hydration or setting times, workability, dispersion and air entrainment (Gambhir, 2008). Most of the admixtures are supplied in ready to use liquid form and are added to concrete in small quantities. The effectiveness of an admixture depends on several factors including type and quantity of cement, water content, mixing time, slump, temperatures of concrete and air.

Admixtures are used for the following purposes:

1. increase the workability
2. improve flow ability and pumpability of concrete
3. increase the strength of concrete

Admixtures are two types

1. Chemical admixtures
2. Mineral admixtures

Chemical admixtures that have been used in concrete mixes are numerous and include chemicals to entrain air, accelerate/retard setting times, reduce amount of mixing water, etc.

Mineral admixtures are Fly Ash, Ground Granulated Blast Furnace Slag (GGBS), Micro Silica and Metakaolin, etc.

The concrete is prepared at batching plant and transported to the site of construction by transient mixers. The time varies from site to site based upon the distance and pumping conditions, etc. Generally it varies from 1 hour to 10hours. The present study has been carried to test the workability and compressive strength of the concrete by using three different admixtures. A comparative analysis of these admixtures is made.

4.2 ADMIXTURES

To improve the pumpability of concrete, three different types of admixtures namely Sulphonated Naphthalene Polymer (SNP) 1, SNP 2 and SNP 3 are considered. These selected admixtures are chloride free. Table 4.1 specifies the properties of these admixtures.

4.2.1 SNP 1: It is a super-plasticizing admixture based on selected sulphonated naphthalene polymers. It disperses the fine particles in the concrete mix, enabling the water content of the concrete to perform more effectively. The water reduction is possible with SNP 1, which also increases the strength of the concrete mix.

4.2.2 SNP 2: It is an highly effective dual action liquid super plasticizer for the production of free flowing concrete or as a substantial water-reducing agent for promoting high early ultimate strength. This consists of aqueous solution of anionic formaldehyde - polycondensate, naphthalene sulphonic acid and sodium salt. It decreases the amount of vibration period for compaction required, normal setting without retardation and reduces risk of segregation.

4.2.3 SNP 3: It has lingosulphonate base. It is suitable for high performance concrete to produce pumpable concrete. The workability increases without extra water. It improves cohesion, minimizes segregation and gives better finish.

Table: 4.1 Properties of Admixtures

S. No.	Test Parameter	SNP 1	SNP 2	SNP 3
1	pH	7.60	7.69	7.8
2	Specific gravity	1.20	1.189	1.179
3	Solid Content (%)	42.16	36	33.54
4	Appearance	Brown liquid	Dark Brown liquid	Brown liquid

4.2.4 Sample preparation

The concrete mix is designed as per IS 10262 – 2009, IS 456-2000 and SP 23. Target mean strength for M25 grade concrete is 33.25 MPa. Target mean strength for M30 grade concrete is 38.25 MPa. The quantity of admixture is taken as 1% by weight of cement. Standard cast iron cube moulds of size 150x150x150 mm are used in the preparation of concrete cubes. The admixture is mixed with the constituents of concrete at the time of adding water. Full blending of the admixture and the concrete is ensured by mixing for a period of at least two minutes. Before casting the cubes, slump test is performed. The results of this slump test are given in the Table 4.2. After casting, the cube specimens are stored in the laboratory at room temperature for 24 hours from the time of addition of water to the ingredients. After this period, the specimens are removed from the moulds, immediately submerged in the clean and fresh water tank for curing. Three samples of each admixture are tested for 7 and 28 days compressive strength. Figure 4.1 to 4.4 shows the preparation and testing of cubes.

Fig. 4.1 Sample Preparation

Fig. 4.2 Cubes after curing

Fig. 4.3 Compression Testing Machine

Fig. 4.4 Concrete Cubes after testing

4.3 RESULTS & DISCUSSIONS

The experiments are conducted for M25 and M30 design concrete mixes. The slump and compressive strength of concrete cubes are presented and a comparative analysis is made.

4.3.1 M25 concrete

Slump tests are conducted for all samples of concrete without and with admixtures (SNP 1, SNP 2 and SNP 3). The results are given below Table.4.2.

Table: 4.2 Slump test results for M25 concrete

Samples	Without admixture	SNP 1	SNP 2	SNP 3
1	75	90	60	90
2	85	100	70	90
3	50	60	70	95
4	65	70	60	95
5	50	60	70	94
6	60	70	70	90
7	60	60	60	90
8	70	65	60	89
9	70	90	90	89
Average	65.00	73.89	67.78	91.33
Maximum	85	100	90	95
Minimum	50	60	60	89
SD	11.46	15.37	9.72	2.55

The maximum and minimum slumps are 85 and 50 mm respectively with an average slump of 68.75 mm for concrete without admixture. The maximum and minimum slumps of concrete with SNP 1 admixture are 100 and 60 mm respectively, with an average slump of 73.5 mm. The maximum and minimum slumps of concrete by using SNP 2 admixture are 90 and 60 mm respectively. The average slump is 70 mm.

The maximum and minimum slumps of concrete by using SNP 3 admixture are 95 and 89 mm respectively. The average slump is 91.2mm. It is observed that among the admixtures, SNP 3 gives better slump when compared to other two admixtures. The degree of workability of the concrete is medium as per the IS 456 – 2000.

Table: 4.3 Compressive strength test results for M25 concrete

S. No.	without admixture		SNP1		SNP 2		SNP 3	
	7 Days	28 Days	7 Days	28 Days	7 Days	28 Days	7 Days	28 Days
1	18.96	28.44	25.1	36.42	26.95	36.62	22.79	37.12
2	19.4	29.48	24.88	36.14	26.81	36.72	23.18	37.06
3	18.81	28.88	25.03	35.7	27.99	36.45	22.81	36.91
4	18.66	29.47	27.7	36.68	26.95	36.18	23.01	36.89
5	18.81	27.4	27.7	36.49	26.96	36.93	22.71	37.08
6	21.54	28.73	27.1	34.92	29.33	36.12	22.81	36.78
7	21.81	25.1	26.97	36.44	29.18	36.78	22.68	37.05
8	22.37	26.96	26.06	36.88	26.21	36.29	22.59	36.75
9	19.41	27.7	25.1	34.1	24.73	34.92	22.98	36.91
Average	19.97	28.02	26.18	35.97	27.23	36.33	22.84	36.95
Maximum	22.37	29.48	27.7	36.88	29.33	36.93	23.18	37.12
Minimum	18.66	25.1	24.88	34.1	24.73	34.92	22.59	36.75
SD	1.49	1.40	1.20	0.92	1.44	0.60	0.18	0.13

The maximum, minimum and average 7 and 28 days compressive strengths without admixtures are 22.37, 18.66, 19.97 and 29.48, 25.1, 28.02 MPa respectively, which are presented in Table 4.3. The maximum, minimum and average 7 and 28 days compressive strength of concrete cubes with SNP 1 admixture are 27.7, 24.88, 26.18 and 36.88, 34.1, 35.97 MPa respectively (Table 4.3).

The maximum, minimum and average 7 days compressive strength of concrete cubes with SNP 2 admixture are 29.33, 24.73, 27.23 MPa and with that of 28 days are 36.93, 34.92 and 36.33 MPa respectively (Table 4.3). The maximum, minimum and average 7 days compressive strengths for SNP 3 admixture are 23.18, 22.59 and 22.84 MPa and with that of 28 days are 37.12, 36.75 and 36.95 MPa respectively (Table 4.3).

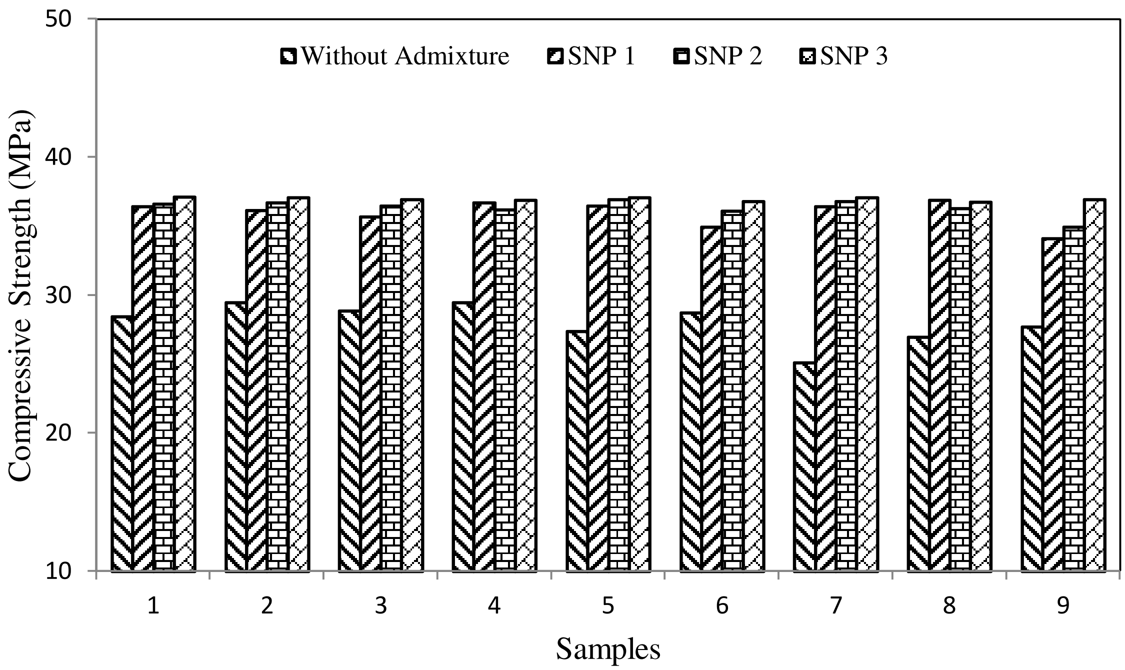

Fig. 4.5 Compressive Strength of M25 concrete with and without Admixtures.

From the Fig. 4.5, it is observed that 28 days compressive strengths of the concrete are varying abnormally in case of SNP 1 when compared to SNP 2 and SNP 3 admixtures. Design concrete mix M25 with SNP3 admixture gives more consistent and uniform values of compressive strength when compared to SNP 2, SNP 1.

4.3.2 M30 concrete

Slump tests are conducted using slump cone for all samples of concrete without and with admixtures (SNP 1, SNP 2 and SNP 3). The results are given below in Table 4.4.

Table: 4.4 Slump test results for M30 concrete

Samples	without Admixture	SNP 1	SNP 2	SNP 3
1	68	80	68	100
2	63	58	65	98
3	74	63	62	95
4	80	90	85	95
5	72	90	90	95
6	63	85	85	93
7	58	73	88	97
8	75	58	92	100
9	80	90	68	90
Average	70.33	76.33	78.11	95.89
Maximum	80	90	92	100
Minimum	58	58	62	90
SD	7.38	12.95	11.37	3.07

The maximum and minimum slumps are 80 and 58 mm respectively with an average slump of 70.33 mm for concrete without admixture. The maximum and minimum slumps of concrete with SNP 1 admixture are 90 and 58 mm respectively, with an average slump of 76.33 mm. The maximum and minimum slumps of concrete by using SNP 2 admixture are 92 and 62 mm respectively. The average slump is 78.11 mm.

The maximum and minimum slumps of concrete by using SNP 3 admixture are 100 and 90 mm respectively. The average slump is 95.89 mm. It is observed that among the admixtures, SNP 3 gives better slump when compared to other two admixtures. The degree of workability of the concrete is medium as per the IS 456 – 2000.

The maximum, minimum and average 7 and 28 days compressive strengths without admixtures are 22.37, 19.66, 20.62 and 32.88, 28.93, 30.96 MPa respectively which are represented in Table

4.5. The maximum, minimum and average 7 and 28 days compressive strength of concrete cubes with SNP 1 admixture are 27.85, 26.95 and 26.95, and 37.8, 34.9 and 36.25 MPa respectively (Table 4.5).

Table: 4.5 Compressive strength test results for M30 concrete

S. No.	without admixture		SNP1		SNP 2		SNP 3	
	7 Days	28 Days	7 Days	28 Days	7 Days	28 Days	7 Days	28 Days
1	20.14	30.44	26.7	37.12	23.25	37.62	23.08	38.3
2	20.4	31.88	26.4	37.8	28.56	37.78	23.18	37.9
3	19.81	32.88	27.1	34.9	26.36	37.33	23.28	38.29
4	20	30.77	27.5	35.1	28.14	36.9	23.32	38.46
5	19.66	29.47	27.1	34.9	27.99	32.9	23.15	38.21
6	19.81	31.89	26.9	35.9	27.7	34.07	23.18	37.85
7	21.54	32.73	27.85	36.82	25.47	36.9	23.25	38.35
8	21.81	28.93	26.91	36.79	22.08	34.62	23.41	38.5
9	22.37	29.63	26.07	36.92	23.33	34.9	22.98	38.48
Average	20.62	30.96	26.95	36.25	25.88	35.89	23.20	38.26
Maximum	22.37	32.88	27.85	37.8	28.56	37.78	23.41	38.5
Minimum	19.66	28.93	26.07	34.9	22.08	32.9	22.98	37.85
STDEV	1.01	1.46	0.54	1.08	2.46	1.79	0.13	0.24

The maximum, minimum and average 7 days compressive strength of concrete cubes with SNP 2 admixture are 28.56, 22.08 and 25.88 MPa, and with that of 28 days are 37.78, 32.9 and 35.89 MPa respectively (Table 4.5). The maximum, minimum and average 7 days compressive strengths for SNP 3 admixture are 23.41, 22.98 and 23.20 MPa, and with that of 28 days are 38.5, 37.85 and 38.26 MPa respectively (Table 4.5).

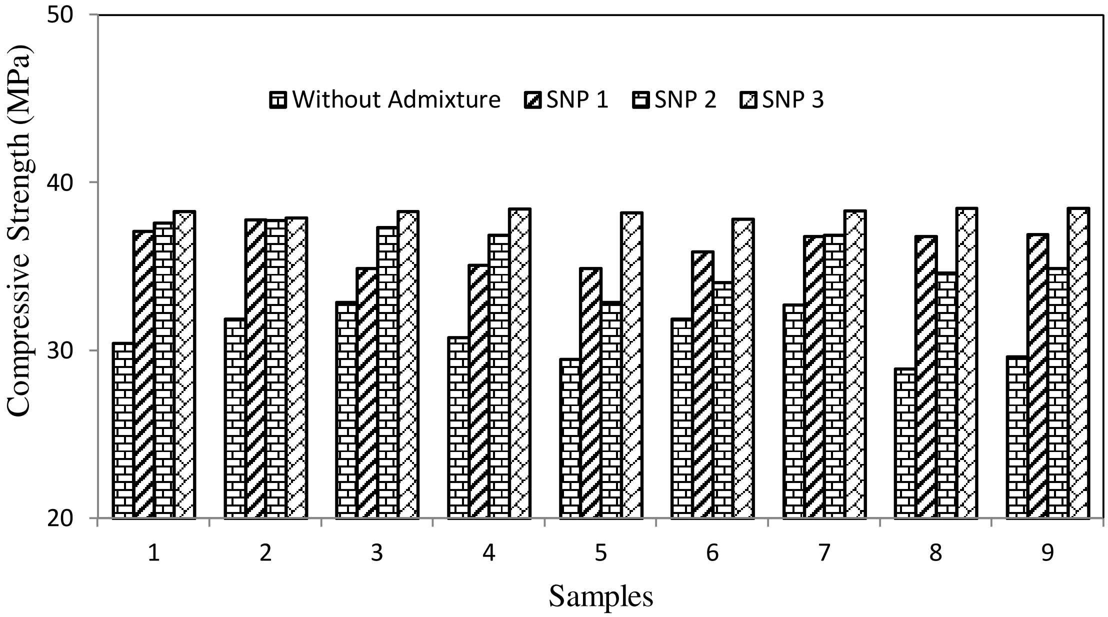

Fig. 4.6 Compressive Strength of M30 concrete with and without Admixtures

From the Fig. 4.6, Design concrete mix M30 with SNP3 admixture gives more consistent and uniform values of compressive strength when compared to SNP 2, SNP 1.

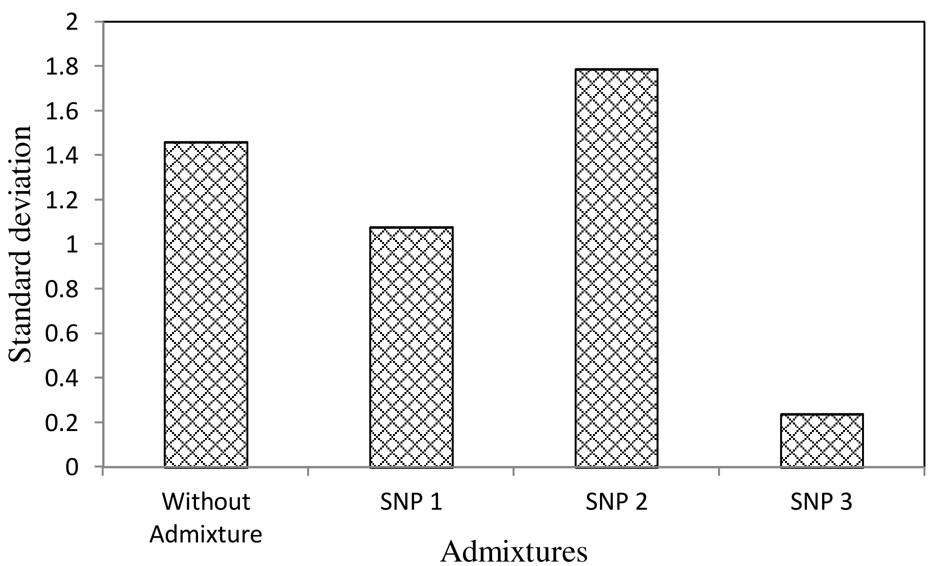

Fig: 4.7 Standard deviation of compressive strength

From Fig. 4.7, the standard deviations of 28 days compressive strength of concrete cubes are 1.08, 1.79 and 0.24 by using SNP 1, SNP 2 and SNP 3 admixtures respectively. The degree of control of these admixtures are laboratory precision SNP 1, SNP 2 and for SNP 3 (Shetty, M. S., 2000).

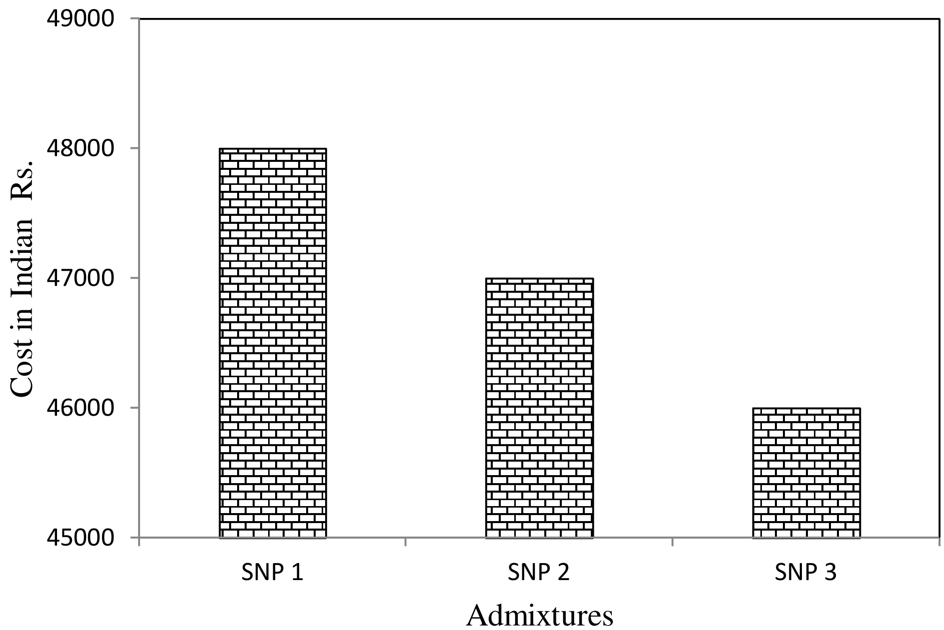

Fig. 4.8 Cost of Admixtures per tonne

The cost analysis is made and reported in Fig. 4.8 in terms of cost per tonne. It observed that the cost of SNP 1 admixture is more when compared to SNP 2 and SNP 3 admixtures.

4.4 SUMMARY

From the experimental investigation, it is observed that the SNP 3 admixture is having better homogeneity and missibility compared to SNP 1 and SNP 2 which can be seen with great clarity in the consistency of test results with SNP 3.

Based on the experiments conducted, the following points can be summarized:

1. Workability of concrete with the admixture SNP 3 is uniform compared to other admixtures.
2. The average slump of measuring workability of concrete with SNP 3 admixture is near to the designed value of the concrete.
3. The average 28 days compressive strength of M25 and M30 concrete by using SNP 3 admixture is increased by 30 and 25.8% respectively compared to concrete without admixture.
4. The concrete with admixture SNP 3 is consistent and uniform in giving the experimental results. It is also evident from standard deviation which is of laboratory precision.

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CHAPTER 5
CONCRETE WITH GROUND GRANULATED BLAST FURNACE SLAG AND ROBO SAND

5.1 INTRODUCTION

Cost of concrete is attributed to the cost of its ingredients which are scarce and expensive. This leads to usage of economical and locally available supplementary materials in its production. Also research on sustainable construction materials suggests extensive use of industrial waste products. Fly ash and Ground Granulated Blast furnace Slag (GGBS) can be used as supplementary materials for the cement and ROBO Sand can be used as a supplementary material instead of fine aggregate. Fly ash produced from thermal power plants and GGBS produced from steel plants. In this study GGBS is used as supplementary material for cement and ROBO Sand is supplementary material for fine aggregate. These materials maximize the reuse of industrial solid waste products in the production of concrete.

5.2 GROUND GRANULATED BLAST FURNACE SLAG (GGBS)

GGBS is obtained by quenching molten iron slag (a by-product of iron and steel making) from a blast furnace in water or steam, to produce a glassy, granular product that is then dried and ground into a fine powder. GGBS is used to make durable concrete structures in combination with ordinary portland cement and/or other pozzolanic materials. GGBS has been widely used for its superiority in concrete durability. Use of GGBS significantly reduces the risk of damages caused by alkali-silica reaction, higher resistance to chloride, and provides higher resistance to attacks by sulfate and other chemicals. GGBS is procured from Vizag Steel Plant (VSP). The fineness modulus of GGBS using blaine's fineness is 320 m^2/kg and other properties of GGBS are given in table 5.1.

Table: 5.1 Properties of GGBS

CONSTITUENTS #	PERCENTAGE CONTENTS	
	Requirement IS 12089-1987	Percentage contents
Calcium Oxide (CaO)	1.4 Max.	1.32
Silica (SiO_2)	30-38	33.31
Alumina (Al_2O_3)	15-25	18.55
Ferric Oxide (Fe_2O_3)	0.5-2.0	0.7
Magnesium Oxide (MgO)	17 Max.	10.72
Manganese Oxide (MnO)	1-5	0.49
Sodium Oxide (Na_2O)	0.5-1	0.5
Glass	85-98	91

Courtesy Vizag Steel Plant (VSP)

5.3 ROBO SAND

ROBO sand obtained from local granite crushers is used in concrete to cast the cubes, cylinders and beams. The bulk density of ROBO sand is 1768kg/m^3. The specific gravity and fineness modulus of ROBO sand are 2.66 and 2.94 respectively. Sieve analysis results are shown in table 5.2 and figure 5.1 represents the grain size distribution of ROBO sand.

Quantity of sample: 1000 gm

Table: 5.2 Sieve analysis of ROBO sand

IS Sieve	Weight retained	% of weight retained	Cumulative % of weight retained	% of passing	Limits as per IS 383-1970 IS 2386-1963
10	0	0	0	100	100
4.75	40	4	4	96	90 – 100
2.36	125	12.5	16.5	83.5	75 – 100
1.18	140	14	30.5	69.5	55 – 90
600	300	30	60.5	39.5	35 – 59
300	230	23	83.5	16.5	8 – 30
150	160	16	99.5	0.5	0 – 10
Total cumulative % of weight retained			294.5		

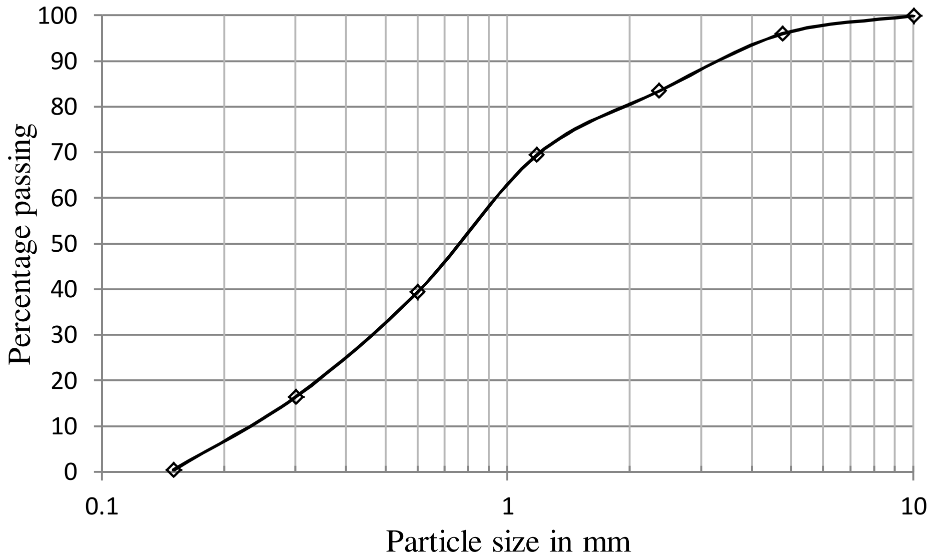

Fig.5.1 Grain size distribution curve for ROBO Sand

The coefficient of uniformity (C_u) and coefficient of curvature (C_c) of the ROBO sand is 4.087 and 1.02 respectively. Based on gradation curve, C_u and C_c the ROBO sand is well graded and confirming to zone II. The sieve analysis conducted for the combined gradation of 80% of natural sand and 20% of ROBO sand. The combined gradation is confirming to zone II.

Experimental investigations are carried to study the behavior of concrete with GGBS as partial replacement of cement and ROBO sand as partial replacement of fine aggregate.

The total experimental investigations are carried out in different phases.

Phase I: Development of two grades of concrete with optimized quantity of GGBS and studies on strength properties of modified concrete.

Phase II: Study the effect of ROBO sand on GGBS concrete and its strength properties.

Phase III: The behavior of modified concrete by using different non-bio degradable waste plastic fibers like High Density Polyethylene (HDPE), High Density Poly Propylene (HDPP), Poly Ethylene Terephthalate (PET) and Polyester fibers and its fresh and hardened properties. Phase III will be discussed in the chapter 6.

Phase IV: formulation of stress strain relation of modified concrete. This phase will be discussed in the chapter 7.

Phase V: Durability studies of modified concrete. This phase will be discussed in chapter 8

5.4 MIX DESIGN AND TRIAL MIX PROPORTIONS OF CONCRETE

Different trial mixes are attempted in the laboratory to get a concrete mix, which gives required strength properties of M25 and M30 design concrete mixes.

5.4.1 GGBS

The whole mixing process was carried out in a concrete mixer. Coarse aggregate, fine aggregate, cement and GGBS were put in the concrete mixer first and mixed in the dry state for few seconds. Later admixture thoroughly mixed with water was added to the material in the concrete mixer. Then it was allowed to mix thoroughly, mixed till a mixture of uniform colour and consistency were achieved. To produce concrete with GGBS, different percentages of GGBS are added to the mix along with coarse aggregate. The mix proportions of concrete mix are shown in tables 5.3 and 5.4 for M25 and M30 respectively.

5.4.2 ROBO sand

ROBO sand is used as partial replacement for fine aggregate in small quantities to the above optimized GGBS mix. The mix proportion of GGBS – ROBO sand concrete is shown in tables 5.11 and 5.12 for M25 and M30 respectively.

Based on the strength properties of modified concrete mixes finally, two concrete mix proportions (i.e. one for M25 and one for M30) with GGBS and ROBO Sand, with relatively high compressive strengths are selected for further investigations. Table 5.19 shows the final mix designs for further investigations.

5.5 EXPERIMENTAL RESULTS AND DISCUSSIONS

During the initial stage of work the cube specimens of the two grades M25 and M30 are cast with cement replaced by 10% to 60% of GGBS (at an increment of 5%) and the specimens are tested for compressive strength at 3 days, 7 days and 28 days. The Compressive strengths are compared with that of corresponding grade concrete specimens without GGBS and found that at 50% GGBS replacement for M25 and M30 grade concrete mixes the strength was maximum.

Based on optimum GGBS percentages arrived from 3 days, 7 days and 28 days cube compressive strengths, the strengths of cylinders and beams are studied for 10% to 60% of GGBS replacement at an increment of 5%.

5.5.1 GGBS Concrete Mix proportions

Table: 5.3 Mix proportions of M25 Concrete

MIX No.	GGBS % (bwc)	Cement Kg	F.A Kg	C.A Kg	GGBS kg	S.P %(bwp)
A-1	0	327	723	1246	0	1
A-2	10	294.3	723	1246	32.7	1
A-3	15	278	723	1246	49.05	1
A-4	20	261.2	723	1246	65.4	1
A-5	25	245.3	723	1246	81.75	1
A-6	30	228.9	723	1246	98.1	1
A-7	35	212.6	723	1246	114.5	1
A-8	40	196.2	723	1246	130.8	1
A-9	45	179.9	723	1246	147.2	1

A-10	50	163.5	723	1246	163.5	1
A-11	55	147.2	723	1246	179.9	1
A-12	60	130.8	723	1246	196.2	1

bwp-by weight of powder   bwc-by weight of cement

The average slump for all the mixtures are 85mm using 1% of admixture

Table: 5.4 Mix proportions of M30 Concrete

MIX No.	GGBS % (bwc)	Cement Kg	F.A Kg	C.A Kg	GGBS kg	S.P %(bwp)
B-1	0	350	704	1245	0	1
B-2	10	315	704	1245	35.8	1
B-3	15	297.5	704	1245	53.8	1
B-4	20	280	704	1245	71.6	1
B-5	25	262.5	704	1245	89.5	1
B-6	30	245	704	1245	107.4	1
B-7	35	227.5	704	1245	125.3	1
B-8	40	210	704	1245	163.5	1
B-9	45	192.5	704	1245	161.1	1
B-10	50	175	704	1245	175	1
B-11	55	157.5	704	1245	196.9	1
B-12	60	140	704	1245	214.8	1

The average slump for all the mixtures are 90mm using 1% of admixture

5.5.1.1 Compressive Strength

The Compressive strength at 3, 7 and 28 days for M25 and M30 grades of concrete is shown in the tables 5.5 and 5.6, figures 5.2 to 5.5.

Table: 5.5 Compressive Strengths of M25 Concrete

Mix. No	% of GGBS Replacement	Compressive Strength (MPa)		
		3- Days	7- Days	28-Days
A-1	0	15.84	21.54	36.81
A-2	10	12.32	18.25	36.92
A-3	15	12.69	18.69	37.23

A-4	20	12.91	18.95	37.89
A-5	25	13.21	19.13	38.12
A-6	30	13.65	19.82	38.65
A-7	35	13.92	20.12	39.02
A-8	40	14.08	20.82	39.82
A-9	45	14.25	21.65	40.56
A-10	50	14.52	22.91	41.21
A-11	55	14.41	22.41	40.65
A-12	60	14.13	21.92	40.01

Table: 5.6 Compressive Strengths of M30 Concrete

Mix. No	% of GGBS Replacement	Compressive Strength (Mpa)		
		3- Days	7- Days	28-Days
B-1	0	17.82	23.26	38.15
B-2	10	14.35	20.32	38.98
B-3	15	14.47	20.86	39.21
B-4	20	15.19	21.64	39.62
B-5	25	15.21	22.35	40.01
B-6	30	15.46	22.98	40.98
B-7	35	15.55	23.68	41.68
B-8	40	15.61	23.98	42.32
B-9	45	15.82	24.28	42.98
B-10	50	16.31	24.98	43.21
B-11	55	16.12	23.89	42.3
B-12	60	15.92	23.01	41.7

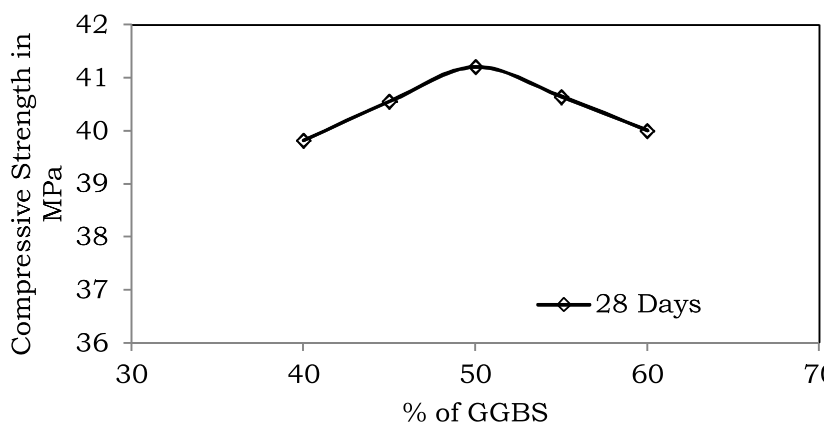

Fig. 5.2 Compressive Strength of M25 concrete with various % of GGBS

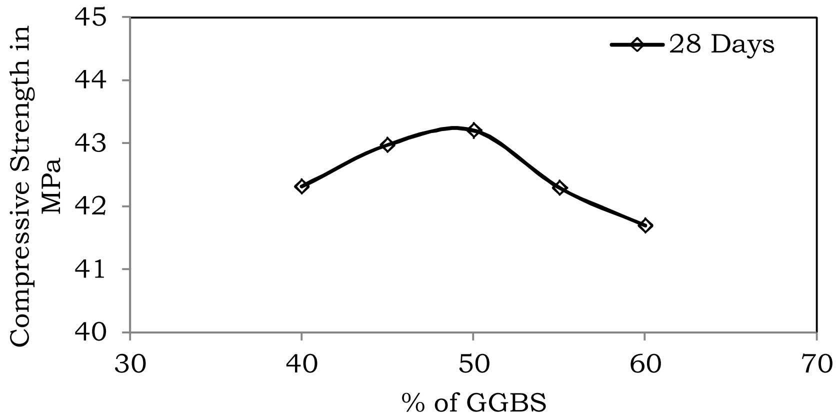

Fig. 5.3 Compressive Strength of M30 concrete with various % of GGBS

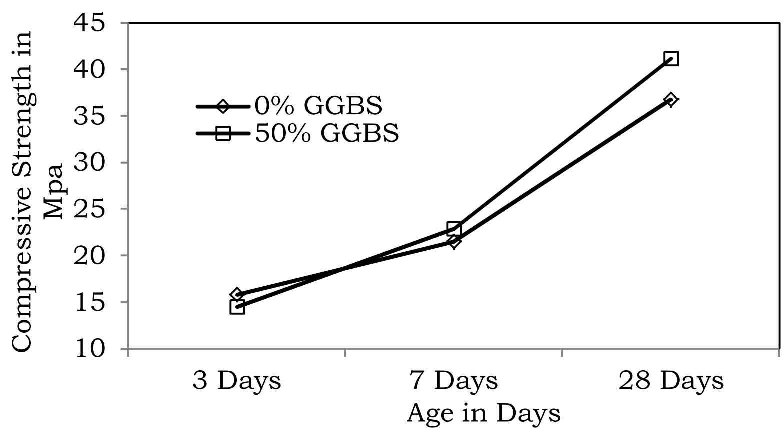

Fig. 5.4 Compressive strength of M25 grade concrete with age for mix with optimum GGBS and without GGBS

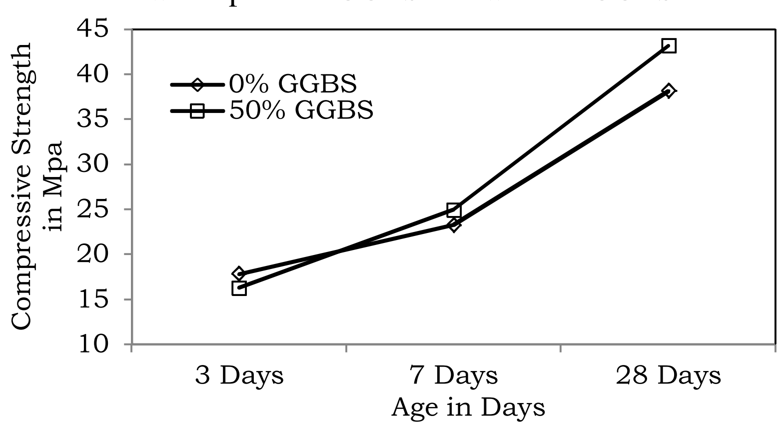

Fig. 5.5 Compressive strength of M30 grade concrete with age for mix with optimum GGBS and without GGBS

The results are shown in tables 5.5 and 5.6, figures 5.2 to 5.5. An improvement of 6.36% and 11.95% in M25 mix and 7.3% and 13.2% in M30 mix was observed when 7days and 28 days compressive strengths of concrete mixes produced with 50% GGBS are compared with other mixes without GGBS.

5.5.1.2 Split Tensile Strength

The Split tensile strength at 28 days for M25 and M30 grades concrete is shown in following tables 5.7 and 5.8 respectively. Figure 5.6 and 5.7 shows the variation of split tensile strength to the percentage of GGBS replacement.

Table: 5.7 28 days split tensile strengths of M25 Concrete

Mix. No	% of GGBS Replacement	Split Tensile strength (Mpa)
A-1	0	3.21
A-2	10	3.23
A-3	15	3.28
A-4	20	3.31
A-5	25	3.35
A-6	30	3.37
A-7	35	3.4
A-8	40	3.42
A-9	45	3.45
A-10	50	3.47
A-11	55	3.44
A-12	60	3.41

Table: 5.8 28 days split tensile strengths of M30 Concrete

Mix. No	% of GGBS Replacement	Split Tensile strength (Mpa)
B-1	0	3.34
B-2	10	3.37
B-3	15	3.39
B-4	20	3.42
B-5	25	3.45
B-6	30	3.47
B-7	35	3.5

B-8	40	3.53
B-9	45	3.59
B-10	50	3.63
B-11	55	3.58
B-12	60	3.52

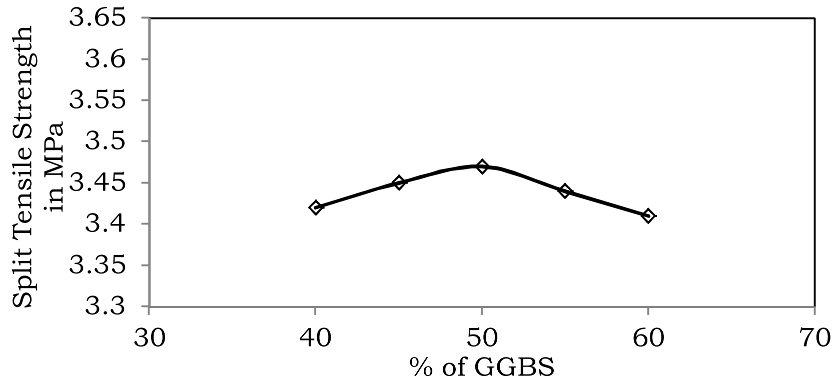

Fig. 5.6 Split Tensile Strength of M25 concrete
With various % of GGBS

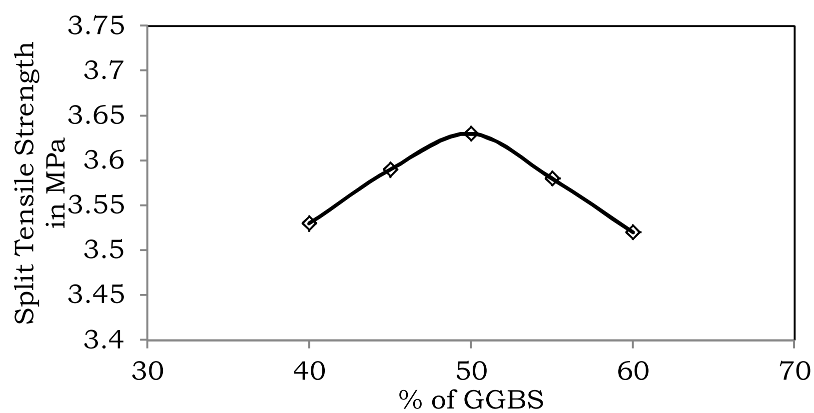

Fig. 5.7 Split Tensile Strength of M30 concrete
with various % of GGBS

The 28 days split tensile strengths of concrete mix with 50% GGBS compared to that of mix without GGBS is increased by 8.09% and 8.68% in M25 and M30 grade of concrete respectively.

5.5.1.3 Flexural Strength

The Flexural Strength at 28 days for M25 and M30 grades concrete is shown in tables 5.9 and 5.10 respectively.

Table: 5.9 28 days flexural strengths of M25 Concrete

Mix. No	% of GGBS Replacement	Flexural strength (Mpa)
A-1	0	4.23
A-2	10	4.25
A-3	15	4.29
A-4	20	4.32
A-5	25	4.36
A-6	30	4.39
A-7	35	4.41
A-8	40	4.42
A-9	45	4.46
A-10	50	4.48
A-11	55	4.47
A-12	60	4.45

Table: 5.10 28 days flexural strengths of M30 Concrete

Mix. No	% of GGBS Replacement	Flexural strength (Mpa)
B-1	0	4.33
B-2	10	4.37
B-3	15	4.4
B-4	20	4.43
B-5	25	4.46
B-6	30	4.49
B-7	35	4.52
B-8	40	4.56
B-9	45	4.61
B-10	50	4.65
B-11	55	4.6
B-12	60	4.57

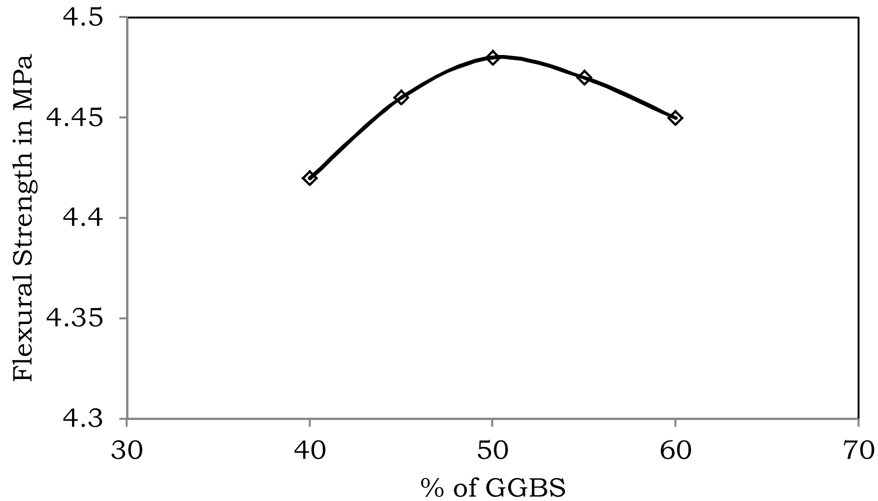

Fig. 5.8 Flexural Strength of M25 Concrete
With various % of GGBS

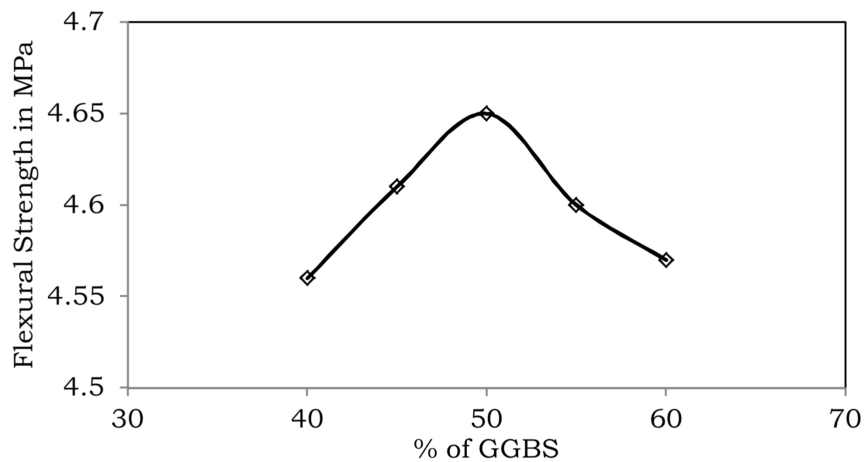

Fig. 5.9 Flexural Strength of M30 Concrete
With various % of GGBS

The increase is 5.91% and 7.39% for M25 and M30 GGBS mixes respectively, thus indicating a considerable increase in the flexural strength of concrete mix made with GGBS when compared to designed concrete mixes. The flexural strength variation is shown in figures 5.8 and 5.9.

The GGBS generally reduces the water demand and improves workability. The factors influencing the reactivity of GGBS are the chemical composition of slag and the glass content which is shown in table 5.1. The presence of GGBS in the mix improves workability and makes the mix more mobile but cohesive. This is the consequence of a better dispersion of the cementitious particles and of the surface characteristics of the GGBS particles; however it is more sensitive to variations in the water content than ordinary cement concrete.

It is observed that there is an increase in the peaks strength properties for different design mixes made with GGBS mixes. Addition of GGBS control the initiation of micro cracks, improve the first crack load, the ultimate load of concrete specimens under flexure. They are also effective in resisting deformation at all stages of loading from first crack to failure.

With the addition of GGBS to the concrete, the initial hydration of GGBS is very slow. It depends mainly upon the breakdown of the glass present in GGBS by the hydroxyl ions released during the hydration of cement. This can be observed in 3 days compressive strength, as it is decreases by maximum 8.3% for the both mixes.

5.5.2 GGBS and ROBO Sand Combined Mixes

Further ROBO sand is used as replacement by 5 to 40% with an incremental value of 5% for fine aggregate. The combined GGBS and ROBO Sand concrete mix cubes are tested in the laboratory. The results are shown in tables 5.11 and 5.12.

Table: 5.11 Mix proportions of M25 Concrete

Mix ID	Cement kg.	F.A kg	C.A kg	GGBS kg	ROBO sand	
					% of FA	Quantity kg
M1	163.5	723	1246	163.5	0	0
M2	163.5	686.85	1246	163.5	5	36.15

M3	163.5	650.7	1246	163.5	10	72.3
M4	163.5	614.55	1246	163.5	15	108.45
M5	163.5	578.4	1246	163.5	20	144.6
M6	163.5	542.25	1246	163.5	25	180.75
M7	163.5	506.1	1246	163.5	30	216.9
M8	163.5	469.95	1246	163.5	35	253.05
M9	163.5	433.8	1246	163.5	40	289.2

The average slump of all the mixes is 60mm using 1% of admixture. But the slump required for the mix is 85mm. So, dosage of admixture is increased 1 % to 1.5% of bwp, the average slump is 87mm.

Table: 5.12 Mix proportions of M30 Concrete

Mix ID	Cement kg.	F.A kg	C.A kg	GGBS kg	ROBO sand	
					% of FA	quantity kg
V1	175	704	1245	175	0	0
V2	175	668.8	1245	175	5	35.2
V3	175	633.6	1245	175	10	70.4
V4	175	598.4	1245	175	15	105.6
V5	175	563.2	1245	175	20	140.8
V6	175	528	1245	175	25	176
V7	175	492.8	1245	175	30	211.2
V8	175	457.6	1245	175	35	246.4
V9	175	422.4	1245	175	40	281.6

The average slump of all the mixes is 59mm using 1% of admixture. But the slump required for the mix is 90 mm. So, dosage of admixture is increased 1% to 1.5% of bwp, the average slump is 92mm. For the further studies 1.5% admixture is used.

5.5.2.1 Compressive strength

The results of compressive strength of modified concrete i.e. 50% of GGBS and 0 to 30% ROBO Sand had shown in table 5.13 and 5.14. Figure 5.10 and 5.11shows the variations in compressive strengths of concrete.

Table: 5.13 Compressive Strengths of M25 Concrete

Mix. No	% of ROBO sand Replacement	Compressive Strength (MPa)		
		3- Days	7- Days	28-Days
M1	0	14.52	22.91	41.21
M2	5	14.62	23.12	41.68
M3	10	14.74	23.43	41.89
M4	15	14.81	23.89	42.56
M5	20	14.92	24.05	42.92
M6	25	15.12	24.12	43.56
M7	30	15.01	24.09	43.01
M8	35	14.48	23.92	42.24
M9	40	14.09	23.56	41.72

Table: 5.14 Compressive Strengths of M30 Concrete

Mix. No	% of ROBO sand Replacement	Compressive Strength (MPa)		
		3- Days	7- Days	28-Days
V1	0	16.31	24.98	43.21
V2	5	16.51	25.42	43.78
V3	10	16.76	25.64	44.02
V4	15	16.82	25.92	44.68
V5	20	16.91	26.19	45.01
V6	25	17.02	26.45	45.71
V7	30	16.72	26.11	45.12
V8	35	16.26	25.89	44.8
V9	40	15.91	25.21	44.29

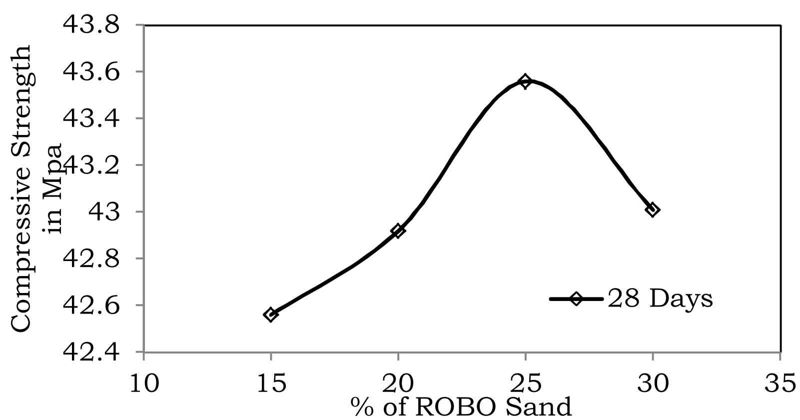

Fig. 5.10 Compressive Strength of M25 Concrete with various % of ROBO Sand

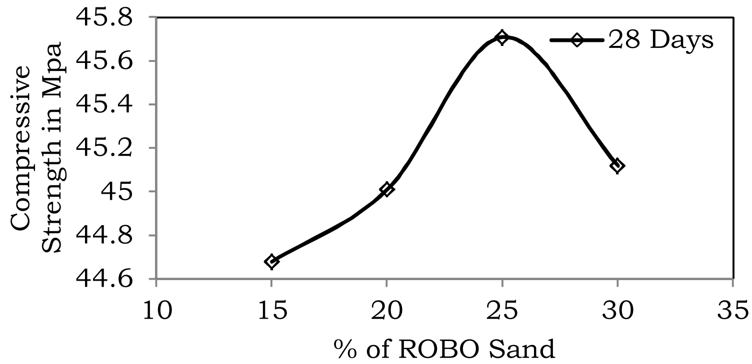

Fig. 5.11 Compressive Strength of M30 Concrete
with various % of ROBO Sand

5.5.2.2 Split Tensile Strength

The split tensile strength of modified concrete using ROBO sand are given tables 5.15 and 5.16 and figures 5.12 and 5.13 shows the variation of split tensile strength to percentage of replacement of ROBO Sand.

Table: 5.15 28 days split tensile strengths of M25 Concrete

Mix. No	% of ROBO sand Replacement	Split Tensile strength (Mpa)
M1	0	3.47
M2	5	3.49
M3	10	3.51
M4	15	3.53
M5	20	3.56
M6	25	3.59
M7	30	3.55
M8	35	3.54
M9	40	3.5

Table: 5.16 28 days split tensile strengths of M30 Concrete

Mix. No	% of ROBO sand Replacement	Split Tensile strength (Mpa)
V1	0	3.63
V2	5	3.67
V3	10	3.69
V4	15	3.71
V5	20	3.73

V6	25	3.75
V7	30	3.72
V8	35	3.74
V9	40	3.72

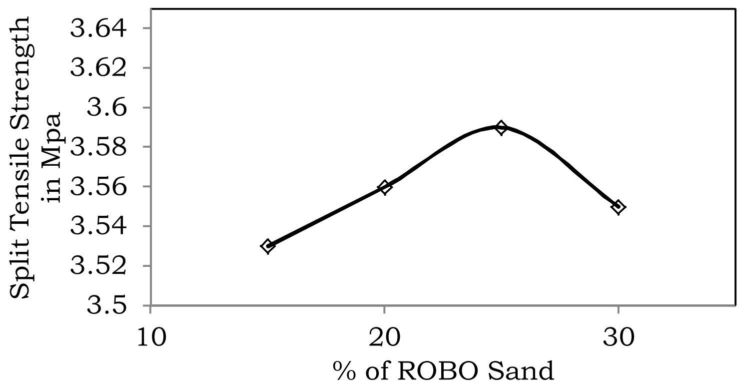

Fig. 5.12 Split Tensile Strength of M25 Concrete
With various % of ROBO Sand

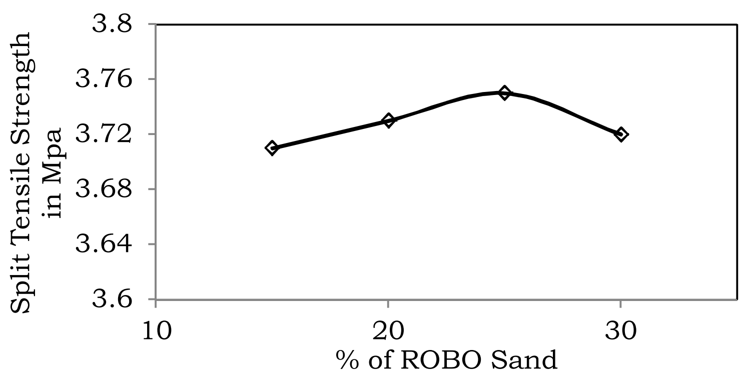

Fig. 5.13 Split Tensile Strength of M30 Concrete
With various % of ROBO Sand

5.5.2.3 Flexural Strength

The results of flexural strength of modified concrete are given table 5.17 and 5.18. The figure 5.14 and 5.15 shows the variation of flexural strength to the percentage of replacement of ROBO Sand.

Table: 5.17 28 days flexural strengths of M25 Concrete

Mix. No	% of ROBO sand Replacement	Flexural strength (Mpa)
M1	0	4.48
M2	5	4.51

M3	10	4.54
M4	15	4.55
M5	20	4.58
M6	25	4.6
M7	30	4.56
M8	35	4.53
M9	40	4.49

Table: 5.18 28 days flexural strengths of M30 Concrete

Mix. No	% of ROBO sand Replacement	Flexural strength (Mpa)
V1	0	4.65
V2	5	4.68
V3	10	4.7
V4	15	4.72
V5	20	4.74
V6	25	4.77
V7	30	4.73
V8	35	4.71
V9	40	4.69

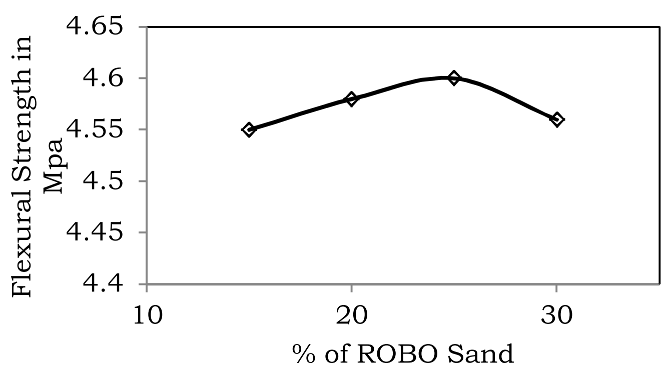

Fig. 5.14 Flexural Strength of M25 Concrete with various % of ROBO Sand

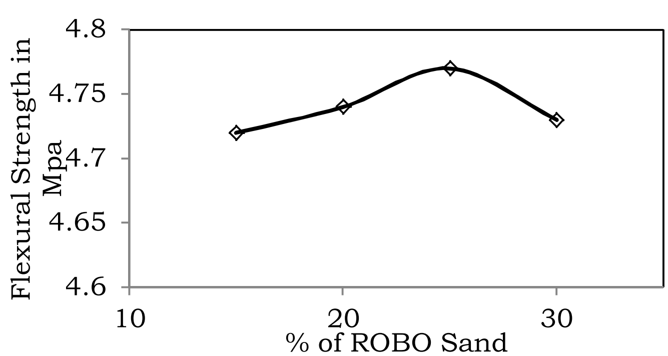

Fig. 5.15 Flexural Strength of M30 Concrete with various % of ROBO Sand

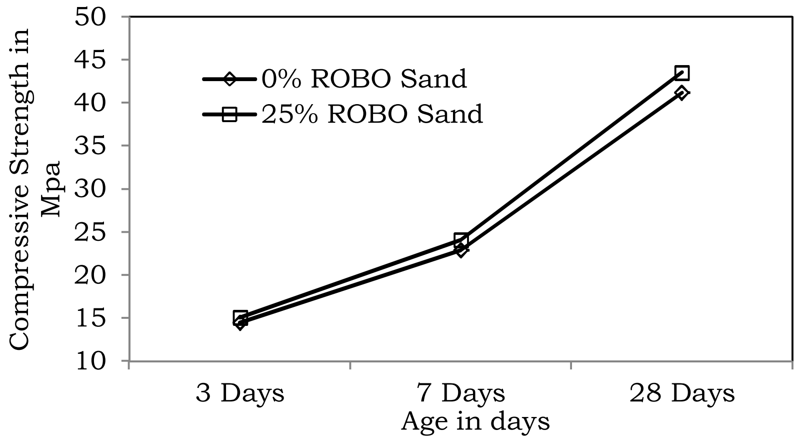

Fig. 5.16 Compressive strength of M25 grade concrete with age for mix with optimum ROBO sand and without ROBO sand

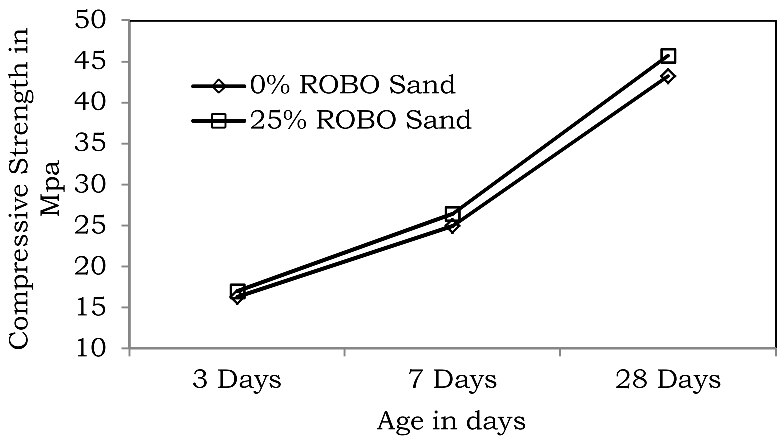

Fig. 5.17 Compressive strength of M30 grade concrete with age for mix with optimum ROBO sand and without ROBO sand

From tables 5.13 and 5.14, figures 5.10 and 5.11, it is observed that the increase in 3days, 7days and 28days strengths are 4.13%, 5.28% and 5.7% respectively for M25. Similarly the increase in 3days, 7days and 28 days strengths are 4.35%, 5.88% and 5.78% respectively for M30 concrete. From the tables 5.15 and 5.16, figures 5.12 and 5.13 the increase in split tensile strength of GGBS - ROBO sand concrete mixes at the age of 28days are 3.45% and 3.3% of M25 and M30 respectively. Similarly the increase in the flexural strengths at the age of 28days are 2.67% and

2.58% of M25 and M30 concrete mixes respectively when compared with conventional concrete without GGBS - ROBO sand. The results of flexural strengths are presented in tables 5.17 and 5.18, figures 5.14 and 5.15.

The observation shows that the increase in strengths when concrete mixes produced with GGBS – ROBO sand. The percentage increase in strength increased with increase in the amount of ROBO sand but only up to 25% percentage of replacement with fine aggregate. The increase in the strength of concrete is due to the aggregate shape i.e. ROBO sand is of more flake compared to river sand.

Finally two concrete mixes which satisfied strength properties of concrete are selected and taken for further investigations. These mix proportions are given in table 5.19.

Table: 5.19 Final mix proportions for further Investigations

Mix	Mix ID	Cement kg	F.A kg	C.A kg	GGBS kg	ROBO sand Quantity kg
M25	M6	163.5	542.25	1246	163.5	180.75
M30	V6	175	528	1245	175	176

5.6 SUMMARY

When port land cement and water are mixed, a chemical reaction called hydration initiates, resulting in the creation of calcium-silicate-hydrate (CSH) and calcium hydroxide (CH). CSH is a gel that is responsible for strength development in port land cement pastes. CH is a byproduct of the hydration process that does not significantly contribute to strength development in normal port land cement mixtures. Silicates in GGBS combine with the CH byproduct of hydration and form additional CSH. This in turn leads to a denser, harder cementitious paste, which increases ultimate strength as compared to 100% port land cement systems.

From the experimental study it is found to be the optimum use of GGBS in concrete is 50%. From the literature, Wang Ling et al. 2004 and Rajamane et al. 2003 conducted the experiments on the use of GGBS in the concrete. They found that, cement can be replaced with GGBS by 50% in the concrete having compressive strength of 50, 70, 80 MPa. So for the normal concrete also cement can be replaced with GGBS up to 50%.

By using ROBO Sand in the concrete, concrete strength is gaining because of its rough surface texture which causes better interlocking and bonding characteristics. In case of river sand the particle grains are smooth and round (figures below).

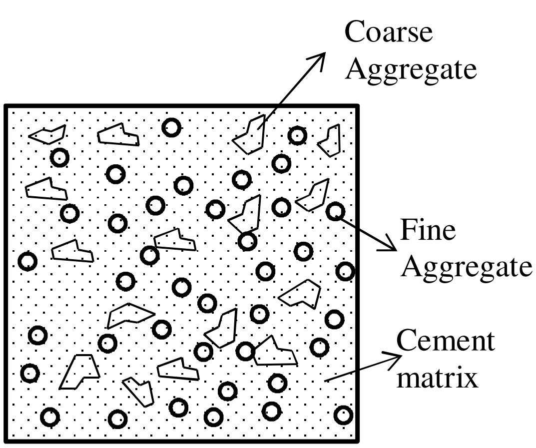

Fig. 5.18 Concrete without ROBO sand

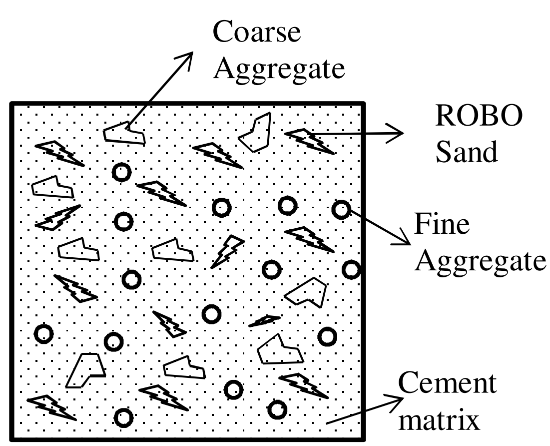

Fig. 5.19 Concrete with ROBO sand

As per the experiments conducted by Vijaya Sarathy 2013 on ROBO sand usage in concrete, the compressive strength results shows decreasing trend of 40, 60 and 80% replacement levels with the fine aggregate. This concludes that the optimum use of ROBO sand may be below 40%. In the present study the experiments are conducted below 40% replacement of ROBO sand. Based on this study the optimum percentage of replacement of fine aggregate with ROBO sand is found to be 25% (less than 40%, hence in line with previous studies).

Based on the experimental investigation studies the following points can be summarized.

1. From the experimental results 50% of cement can be replaced with GGBS.
2. The increase in compressive strength, split tensile strength and flexural strength of concrete with 50% GGBS are 11.95%, 8.09% and 5.91% for M25 concrete, for M30 13.26%, 8.68% and 7.39% respectively compared to normal concrete without GGBS.
3. The percentage of increase in the compressive strength are 5.70% at the age of 28 days and the percentage of increase in the split tensile and flexural strengths are 3.45% and 2.68% at the age of 28 days for M25 concrete with 50% GGBS and 25% ROBO Sand.
4. Similarly for M30 modified concrete (50% GGBS+25% ROBO Sand) compressive, split tensile and flexural concretes at the age of 28days are 5.79%, 3.3% and 2.58% respectively.

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CHAPTER 6
FIBER REINFORCED CONCRETE

6.1 INTRODUCTION

Concrete is a heterogeneous composite material made up of cement, fine aggregate, coarse aggregate and water mixed in a desired proportion based on the strength requirements. Plain concrete is strong in compression and weak in tension. The reinforcement in the concrete is used to increase the tensile strength and ductility of members. Fiber Reinforced Concrete (FRC) is an emerging field in the area of Concrete Technology. The addition of fibers in concrete would act as crack inhibitors and substantially improve the tensile strength, cracking resistance, impact resistance and ductility of concrete. The general form of fiber composites will be in the use of short discontinuous fibers. Generally economic considerations will dictate the choice and volume of percentage of fibers used.

Following a normal growth in population, the amount and type of waste materials have increased accordingly. Many of the non-decaying waste materials will remain in the environment for hundreds, or may be perhaps thousands of years. The non-decaying waste materials cause a waste disposal crisis, thereby further contributing to the environmental problems. The problem of waste accumulation exists worldwide, specifically in the densely populated areas. Most of these materials are left as stockpiles, landfill material or illegally dumped in selected areas. An attempt has been made to study using solid waste materials (water bottles, polythene bags, disposable glasses, cement bags, cool drink bottles etc.) as fibers in the concrete.

6.2 ROLE OF FIBERS

The crack, which starts from the bottom most layers, will progress slowly in upward direction, and its growth will be resisted by the bridging fiber. These fibers help to carry the load, thereby increasing the tensile strength of material and arrest the propagation of crack.

At the ultimate stage either the fiber gets pulled out from the matrix or yielding of fibers occurs. This slow progress of crack would lead to a ductile failure and it will give sufficient time between the onset of flexure crack and ultimate failure.

6.3 FACTORS INFLUENCING THE FIBERS IN THE CONCRETE

The effect of fiber reinforced on the matrix and the efficient transfer of stress between the matrix and fiber depends on many factors. Many of these factors intimately inter dependent and exercise a profound but complex influence on the properties of the composite. The following are the factors influencing in the concrete by using fibers.

1. Aspect ratio
2. Orientation of fibers
3. Volume of fibers
4. Spacing of fibers

6.3.1 Aspect ratio: It is the ratio of the length of fiber to the diameter/width of fiber. In the present study the aspect ratio (length/width) of fibers are 20. At most care has been taken to maintain and cut the fibers with same aspect ratio in the workshop. Figures 6.1 and 6.2 shows the length and width of the fiber. Length of the fiber is 50mm and width is 2.5mm, aspect ratio is 50/2.5 is 20.

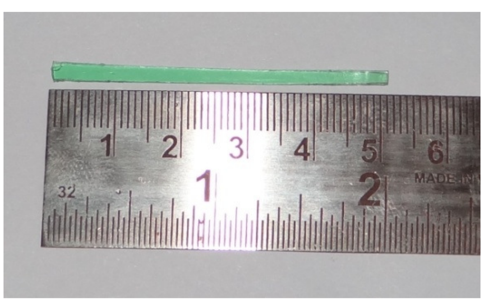

Fig. 6.1 Length of the fiber

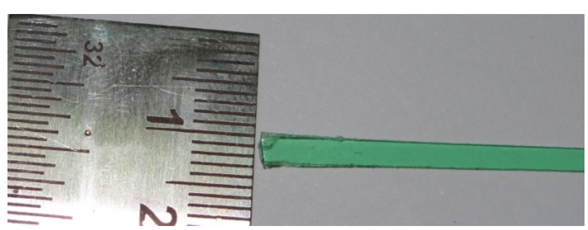

Fig. 6.2 width of the fiber

6.3.2 Orientation of Fibers: One of the differences in conventional reinforcement and fiber reinforcement is that, the conventional reinforcement bars are oriented in the desired direction while fibers are randomly oriented in the concrete.

6.3.3 Volume of fibers: The strength of the composite concrete largely depends on the quantity of fibers used in it. Use of higher percentage of fibers is likely to cause segregation of concrete and fibers can form like ball in the mix this will affect the strength of concrete.

6.4 CLASSIFICATION OF FIBERS

Fibers can be classified in to two categories.

1. Based on modulus of elasticity
2. Based on the material

Based on the modulus of elasticity the fibers can further classified as:

a. Hard intrusion of fibers: fibers having higher elastic modulus than the cement matrix can be termed as hard intrusion fibers.

b. Soft intrusion fibers: these fibers having lower elastic modulus than the cement matrix can be termed as soft intrusion fibers.

Based on the materials:

i. Steel fibers: Steel fibers are probably the only fibers that can be used for long time load bearing applications. They are stable in cement matrix and need no longer to be a design or cost inhibiting factors. Steel fibers are classified as collected steel fibers and epoxy coated steel fiber.

ii. Glass fibers: glass fibers in a process in which molten glass is drawn in the form of filaments. Glass fiber reinforced cementitious composites have been developed mainly for the production of sheet components with a paste or mortar matrix and about 5% fiber content. The other application have been considered either by making reinforcing bars with plastic or by making similar short, rigid units impregnated with epoxy, to be depressed in concrete during mixing.

iii. Carbon fibers: carbon fibers are very expensive but the strength and stiffness characteristics are superior to steel. Carbon fibers are inert in aggressive environments, abrasion, resistant and stable at high temperatures with relatively high stiffness. However carbon fibers are more vulnerable than the glass fibers to surface damage and subsequent weakening and must be used in the clumped form i.e. embedded in or sized with resin coating.

iv. Synthetic fibers: They are classified as polypropylene fiber, nylon, polyethylene, polyester and rayon fibers etc. Plain twisted fibers, button ended are the form of polypropylene fibers. The potential market for polypropylene reinforced cement is principally as a substitute for the asbestos cement.

v. Natural Fibers: They are classified as wood, coconut, bamboo, jute, sugarcane bars, mineral wool, rock wool and vegetable fibers like elephant grass, water reed etc.

6.5 FIBERS USED IN THE PRESENT EXPERIMENTAL WORK

The following fibers are used in the present experimental work:

i. High Density Polyethylene (HDPE) fibers

ii. Poly Ethylene Teraphthalate (PET) fibers

iii. High Density Poly Propylene (HDPP) fibers

iv. Polyester fibers

6.5.1 High Density Polyethylene Fiber (HDPE): HDPE fiber is a relatively straight chain structure, but, as its name implies, exhibits a higher density. It is naturally milky white in appearance and finds wide application in blow molded bottles for milk, water and fruit juices. HDPE, pigmented with a variety of colorants, is used for packaging toiletries, detergents and similar products. The chemical composition of polyethylene is defined in ASTM D1248-84 as a product of ethylene polymerization with a bulk density of 0.94gm/cm^3 or higher. This fiber is used to prepare Milk, water and juice containers, grocery bags, toys, liquid detergent bottles. The recycled products of this fiber is Recycling bins, benches, bird feeders, retractable pens, clipboards, fly swatters, dog houses, vitamin bottles, floor tile, and liquid laundry detergent containers. Fig. 6.3 represents the Chemical structure and table 6.1 gives the properties of HDPE fiber.

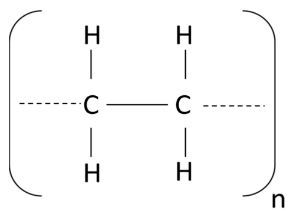

Fig: 6.3 Chemical structure of HDPE fiber

Table: 6.1 Properties of HDPE Fiber

S. No	Test	Result (CIPET)
1	Tensile load at yield	67.79Kg
2	Density	1.22gm/cc
3	Identification	High Density Polyethylene

6.5.2 Poly Ethylene Teraphthalate (PET): This is a thermo plastic resin of the polyester family that is used to make beverage, food and other liquid containers. PET blends are engineered plastics with excellent processing characteristics and high strength and rigidity for a broad range of applications unlike other plastics. This is most important raw material used in man-made fibers. Depending on its processing and thermal history, it may exist both as an amorphous and semi crystalline material. It can be synthesized by transesterification reaction between ethylene glycol and dimethyl terephthalate. It is manufactured under the names Arnite, Impet & rynite, Hostaphan, Melinex & Mylar Films and Darcon Terylene & Treivive fibres. Fig. 6.2 represents the Chemical structure and table 6.4 gives the properties of PET fiber.

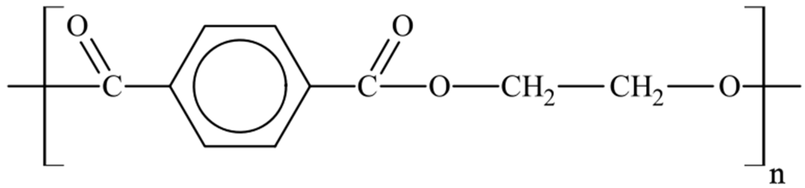

Fig: 6.4 Chemical structure of PET fiber

Table: 6.2 Properties of PET Fiber

S. No	Test	Result (CIPET)
1	Tensile load at yield	72.03Kg
2	Density	1.43 gm/cc
3	Identification	Polyethylene Teraphthalate (PET)

6.5.3 High density Polypropylene Fiber (HDPP): HDPP is a linear polymer with the chemical composition of polypropylene (CH_3) N and defined by ASTM D638 as a product of propylene polymerization with a bulk density of 0.036gm/cm^3 or higher. It has a high melting point, yet is readily heat-sealable. In film form it may or may not be oriented (stretched). It is also relatively inexpensive. PP is found in everything from flexible and rigid packaging to fibers and large molded parts for automotive and consumer products. This fiber is used to prepare the ketchup bottles, yogurt containers and margarine tubs, medicine bottles. The recycled products of this fiber are Signal lights, battery cables, brooms and brushes, ice scrapers, oil funnels, landscape borders, bicycle racks. Fig. 6.5 represents the Chemical structure and table 6.3 gives the properties of HDPP fiber.

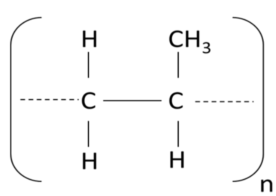

Fig: 6.5 Chemical structure of HDPP fiber

Table: 6.3 Properties of HDPP Fiber

S. No	Test	Result (CIPET)
1	Tensile load at yield	64.41Kg
2	Density	0.88 gm/cc
3	Identification	Polypropylene (PP)

6.5.4 Polyester Fiber: This polymer used for making many soft drink bottles and it is becoming increasingly common to recycle them after use by re-melting them and extruding it as fiber. The Table 6.4 gives the properties of polyester fiber.

Table: 6.4 Properties of Polyester fiber

S. No	Test	Result (CIPET)
1	Tensile load at yield	60.22Kg
2	Density	1.35 gm/cc
3	Identification	Polyester fiber

6.6 EXPERIMENTAL INVESTIGATION

Fibers are added to the concrete during dry mixing of materials. The Mixing has been done carefully to get uniform dispersion of fibers and to prevent segregation or balling of the fibers. That is, the fibers are randomly distributed in the concrete during mixing. Specimens are prepared using design mix given table 5.19 with fiber percentages starting from 0 to 6% with an increment of 0.5 by volume of cement. Cubes 150X150X150 mm, cylinders with 150φ X 300mm and beams of size 100X100X500 mm are prepared. The samples are kept in a sump for curing.

6.6.1 Experimental Results: Slump tests are conducted for finding the workability of the concrete. The average slump of 80mm attained for all fibers whereas targeted slump is 85mm.

6.6.1.1 Strength Properties of HDPE Concrete

The experimental results of compressive strength, split tensile strength and flexural strength of modified concrete i.e. with the addition of HDPE fiber in the concrete are given below tables and figures.

Table: 6.5 Compressive Strength of M25 concrete with HDPE fibers

% fiber	3 Days	7 Days	28 Days
0	15.12	24.12	43.56
0.5	15.31	24.41	43.81
1	15.42	24.86	44.46
1.5	15.61	25.12	44.91
2	15.81	25.38	45.85
2.5	15.89	25.64	46.24
3	15.93	25.89	47.01
3.5	16.02	26.12	47.51
4	15.94	25.91	46.81
4.5	15.88	25.76	46.12
5	15.76	25.39	45.24
5.5	15.61	24.91	44.87
6	15.54	24.67	44.08

Table: 6.6 Compressive Strength of M30 concrete with HDPE fibers

% fiber	3 Days	7 Days	28 Days
0	17.02	26.45	45.71
0.5	17.16	26.89	45.98
1	17.21	27.24	46.58
1.5	17.39	27.78	47.12
2	17.56	27.91	47.86
2.5	17.81	28.01	48.35
3	17.95	28.4	49.01
3.5	18.11	28.65	49.75
4	17.91	28.39	48.92
4.5	17.72	27.89	48.4
5	17.59	27.51	47.68
5.5	17.31	27.31	47.11
6	17.12	27.01	46.51

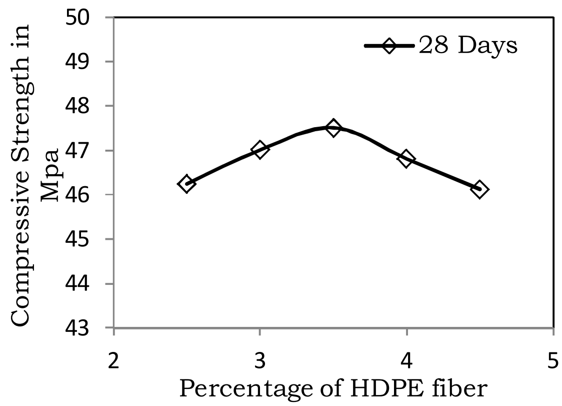

Fig. 6.6 Compressive Strength of M25 concrete with HDPE fibers

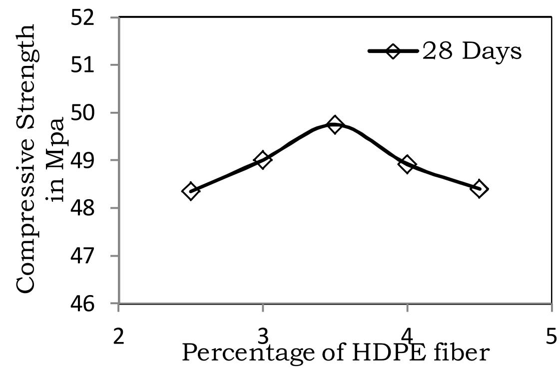

Fig. 6.7 Compressive Strength of M30 concrete with HDPE fibers

Table: 6.7 Split tensile and Flexural Strengths M25 concrete with HDPE Concrete

% fiber	Split tensile strength at 28 Days	Flexural strength at 28 Days
0	3.59	4.6
0.5	3.62	4.64
1	3.63	4.69
1.5	3.65	4.75
2	3.69	4.78
2.5	3.72	4.81
3	3.78	4.84
3.5	3.81	4.86
4	3.76	4.83
4.5	3.7	4.79
5	3.68	4.73
5.5	3.63	4.68
6	3.58	4.63

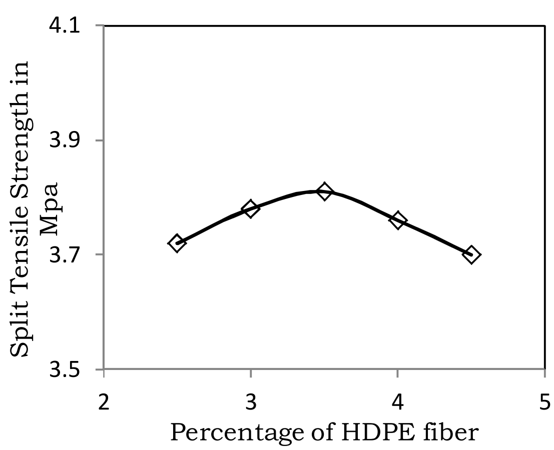

Fig. 6.8 Split Tensile Strength of M25 concrete with HDPE Concrete

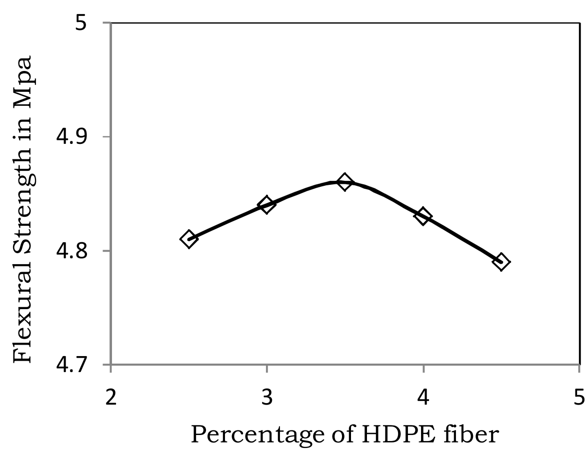

Fig. 6.9 Flexural Strength ofM25 concrete with HDPE Concrete

Table: 6.8 Split tensile and Flexural Strengths M30 concrete with HDPE Concrete

% fiber	Split tensile strength at 28 Days	Flexural strength at 28 Days
0	3.75	4.77
0.5	3.79	4.8
1	3.85	4.86
1.5	3.89	4.89
2	3.92	4.91
2.5	3.96	4.95
3	3.98	4.99
3.5	4.01	5.05
4	3.97	5.01
4.5	3.91	4.98
5	3.84	4.94
5.5	3.77	4.91
6	3.71	4.88

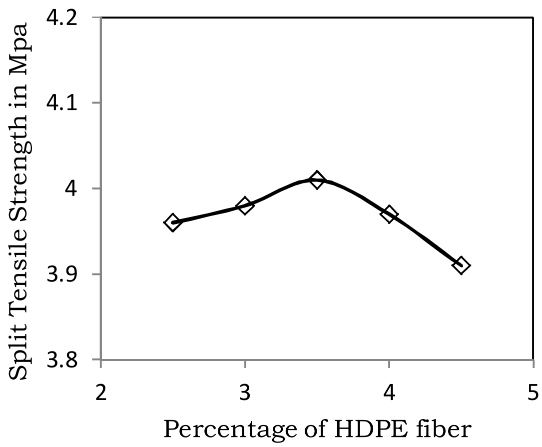

Fig. 6.10 Split Tensile Strength of M30 concrete with HDPE Concrete

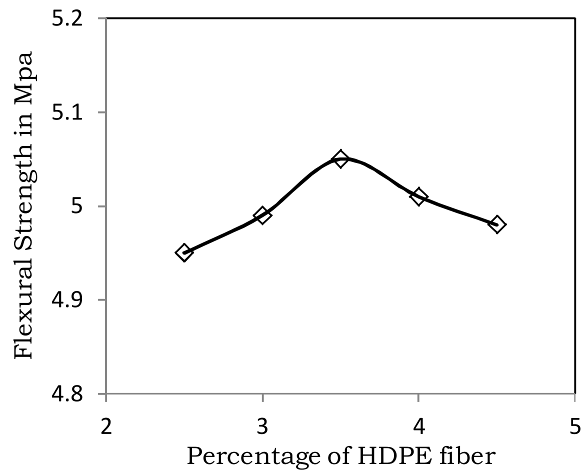

Fig. 6.11 Flexural Strength of M30 concrete with HDPE Concrete

The results of modified HDPE fiber reinforced concrete are shown in tables 6.5 to 6.8, figures 6.6 to 6.11. Based on the results the strength properties of fiber reinforced concrete are increasing as percentage of fiber increases up to 3.5%, later the strength is decreasing. An improvement of 5.95%, 8.29% and 9.06% in M25 design mix and 6.4%, 8.31% and 8.83% in M30 design mix are observed when 3days, 7days and 28 days compressive strengths of modified concrete at 3.5% HDPE fibers in the concrete. Similarly, the improvement in the split tensile and flexural strengths is 6.12% and 5.65% of M25 concrete and 6.93% and 5.87% for M30 concrete respectively.

6.6.1.2 Strength Properties of PET Concrete

The experimental results of PET fiber reinforced concrete i.e. compressive strength, split tensile strength and flexural strength are given tables 6.9 to 6.12 and figures 6.10 to 6.15.

Table: 6.9 Compressive Strength of M25 concrete with PET fibers

% fiber	3 Days	7 Days	28 Days
0	15.12	24.12	43.56
0.5	15.26	24.29	43.82
1	15.32	24.65	44.12
1.5	15.56	24.97	44.68
2	15.71	25.56	45.23
2.5	15.86	25.78	45.96
3	15.94	25.96	46.81
3.5	16.04	26.18	47.59
4	15.9	25.89	46.75
4.5	15.81	25.61	45.89
5	15.68	25.01	45.1
5.5	15.5	24.81	44.56
6	15.31	24.42	44.01

Table: 6.10 Compressive Strength of M30 concrete with PET fibers

% fiber	3 Days	7 Days	28 Days
0	17.02	26.45	45.71
0.5	17.21	26.81	45.98
1	17.45	27.05	46.56
1.5	17.68	27.56	46.92
2	17.79	27.89	47.69
2.5	17.86	28.12	48.23
3	17.95	28.46	48.96
3.5	18.15	28.7	49.8
4	17.91	28.41	49.01
4.5	17.81	28.05	48.65
5	17.7	27.85	48.12
5.5	17.49	27.49	47.61
6	17.3	26.91	46.89

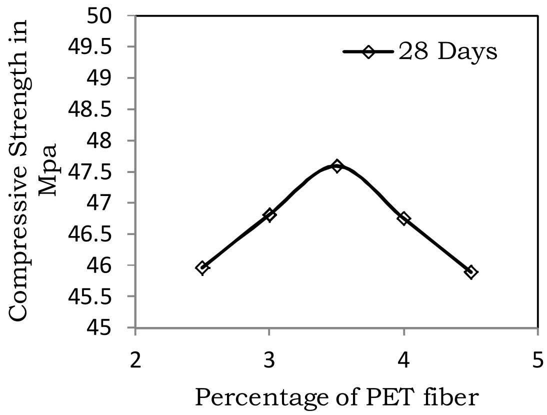

Fig. 6.12 Compressive Strength of M25 concrete with PET fibers

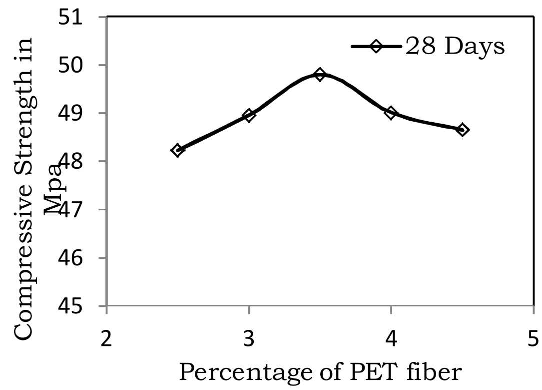

Fig. 6.13 Compressive Strength of M30 concrete with PET fibers

Table: 6.11 Split tensile and Flexural Strengths M25 concrete with PET fibers

% fiber	Split tensile strength at 28 Days	Flexural strength at 28 Days
0	3.59	4.6
0.5	3.62	4.64
1	3.65	4.69
1.5	3.69	4.73
2	3.71	4.77
2.5	3.75	4.81
3	3.79	4.85
3.5	3.84	4.88
4	3.8	4.84
4.5	3.74	4.8
5	3.72	4.76
5.5	3.68	4.71
6	3.64	4.68

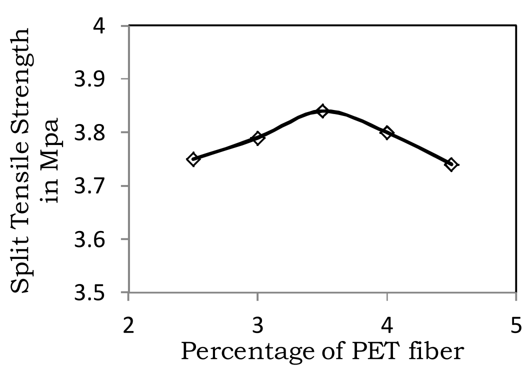

Fig. 6.14 Split Tensile Strength of M25 concrete with PET fibers

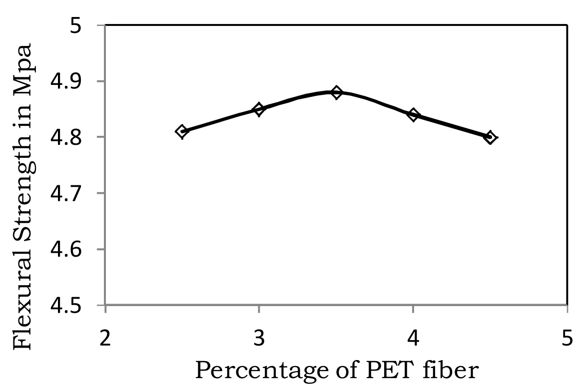

Fig. 6.15 Flexural Strength of M25 concrete with PET fibers

Table: 6.12 Split tensile and Flexural Strengths M30 concrete with PET fibers

% fiber	Split tensile strength at 28 Days	Flexural strength at 28 Days
0	3.75	4.77
0.5	3.79	4.8
1	3.81	4.86
1.5	3.86	4.89
2	3.91	4.91
2.5	3.93	4.95
3	3.98	5.01
3.5	4.03	5.07
4	3.95	5.02
4.5	3.85	4.94
5	3.8	4.9
5.5	3.78	4.88
6	3.71	4.84

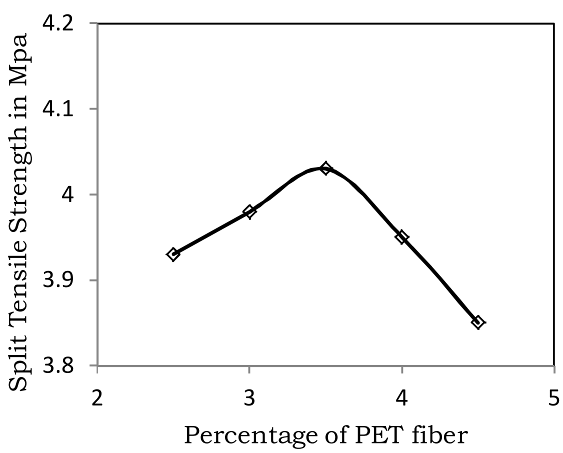

Fig. 6.16 Split Tensile Strength of M30 concrete with PET fibers

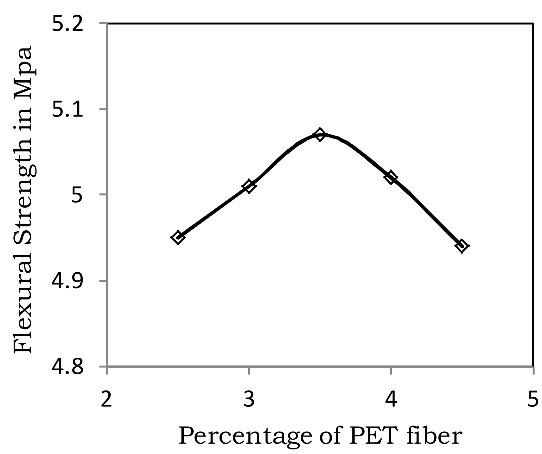

Fig. 6.17 Flexural Strength of M30 concrete with PET fibers

From the tables 6.9 to 6.12 and figures 6.12 to 6.17, the strengths of modified PET fiber reinforced concrete strengths are increases as percentage of fiber increases up to 3.5%, later the strength is decreasing. The increase in the compressive strength is 6.08%, 8.54% and 9.25% in M25 mix and 6.63%, 8.5% and 8.94% in M30 mix was observed at the age of 3days, 7days and 28 days. Similarly, the improvement in the split tensile and flexural strengths is 6.96% and 6.08% of M25 concrete and 7.46% and 6.28% for M30 concrete respectively.

6.6.1.3 Strength Properties of HDPP Concrete

The experimental results of HDPP fiber reinforced concrete i.e. compressive strength, split tensile strength and flexural strengths are given below tables and figures.

Table: 6.13 Compressive Strength of M25 Concrete with HDPP fibers

% fiber	3 Days	7 Days	28 Days
0	15.12	24.12	43.56
0.5	15.29	24.56	43.87
1	15.4	24.89	44.05
1.5	15.59	25.02	44.69
2	15.78	25.45	45.07
2.5	15.84	25.64	45.89
3	15.92	25.89	46.56
3.5	15.99	26.01	47.31
4	15.91	25.91	46.61
4.5	15.82	25.62	45.98
5	15.72	25.4	45.01
5.5	15.6	24.98	44.51
6	15.51	24.5	43.81

Table: 6.14 Compressive Strength M30 Concrete with HDPP fibers

% fiber	3 Days	7 Days	28 Days
0	17.02	26.45	45.71
0.5	17.29	26.59	46.08
1	17.4	26.85	46.69
1.5	17.52	26.98	46.94
2	17.68	27.56	47.68
2.5	17.81	27.89	48.24
3	17.95	28.01	48.98
3.5	18.02	28.6	49.6
4	17.9	27.96	48.9
4.5	17.78	27.12	48.12
5	17.65	26.85	47.56
5.5	17.48	26.42	46.78
6	17.2	26.02	46.01

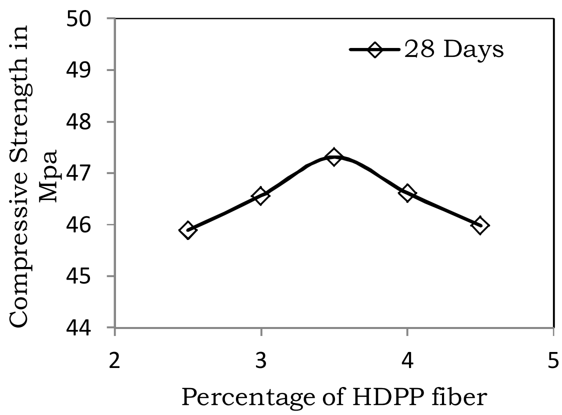

Fig. 6.18 Compressive Strength of M25 Concrete with HDPP fibers

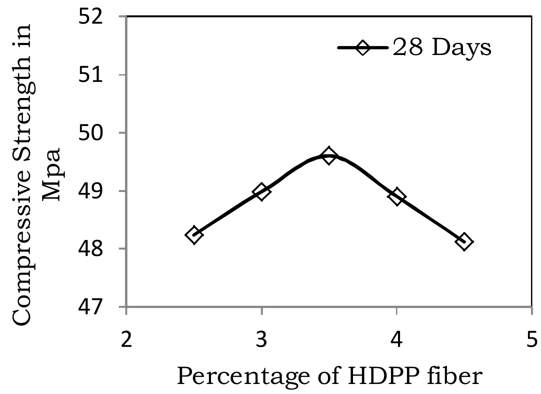

Fig. 6.19 Compressive Strength of M30 Concrete with HDPP fibers

Table: 6.15 Split tensile and Flexural Strengths M25 Concrete with HDPP fibers

% fiber	Split tensile strength at 28 Days	Flexural strength at 28 Days
0	3.59	4.6
0.5	3.63	4.64
1	3.66	4.66
1.5	3.69	4.69
2	3.71	4.73
2.5	3.73	4.78
3	3.76	4.81
3.5	3.79	4.84
4	3.75	4.8
4.5	3.72	4.77
5	3.68	4.72
5.5	3.65	4.67
6	3.61	4.65

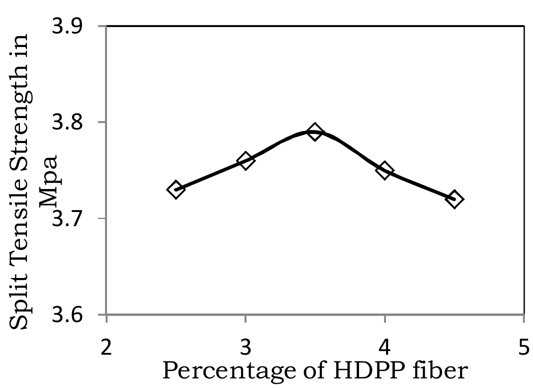

Fig. 6.20 Split Tensile Strength of M25 Concrete with HDPP fibers

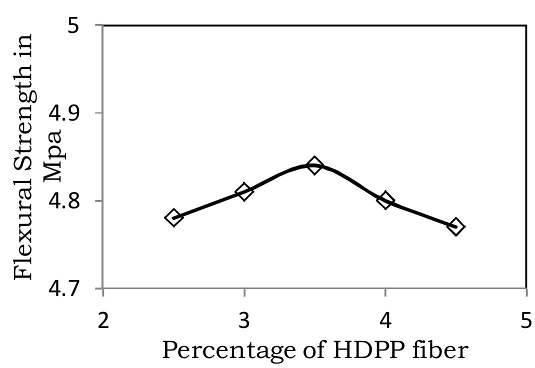

Fig. 6.21 Flexural Strength of M25 Concrete with HDPP fibers

Table: 6.16 Split tensile and Flexural Strengths
M30 Concrete with HDPP fibers

% fiber	Split tensile strength at 28 Days	Flexural strength at 28 Days
0	3.75	4.77
0.5	3.79	4.79
1	3.82	4.83
1.5	3.86	4.86
2	3.89	4.9
2.5	3.93	4.93
3	3.95	4.95
3.5	3.98	5.01
4	3.94	4.94
4.5	3.91	4.88
5	3.88	4.81
5.5	3.84	4.78
6	3.8	4.76

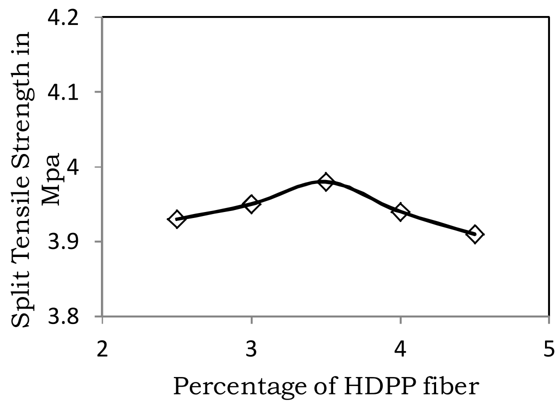

Fig. 6.22 Split Tensile Strength of M30 Concrete with HDPP fibers

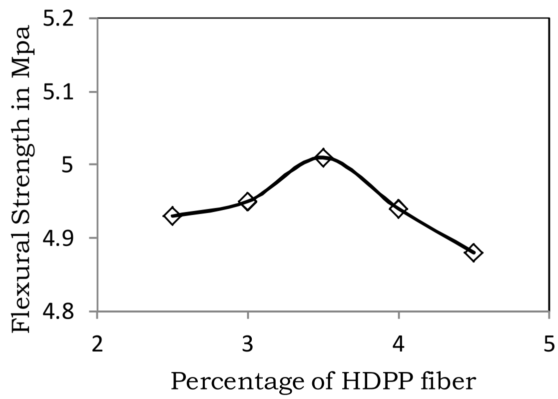

Fig. 6.23 Flexural Strength of M30 Concrete with HDPP fibers

The results of experiments are presented in the tables 6.13 to 6.16 and figures 6.18 to 6.23. From 0.5% to 3.5% of fiber in the concrete, the strength is increases as the percentage of fiber increases. The increase in the compressive strength is 5.75%, 7.83% and 8.6% in M25 mix and 5.87%, 8.13% and 8.51% in M30 mix was observed at the age of 3days, 7days and 28 days. Similarly, the improvement in the split tensile and flexural strengths is 5.57% and 5.21% of M25 concrete and 6.13% and 5.03% for M30 concrete respectively.

6.6.1.4 Strength Properties of POLYESTER Concrete

The experimental results of POLYESTER fiber reinforced concrete i.e. compressive strength, split tensile strength and flexural strengths are given below tables and figures.

Table: 6.17 Compressive Strength of M25 Concrete with POLYESTER fibers

% fiber	3 Days	7 Days	28 Days
0	15.12	24.12	43.56
0.5	15.23	24.29	44.01
1	15.35	24.56	44.86
1.5	15.41	24.71	45.12
2	15.59	24.89	45.89
2.5	15.68	25.21	46.56
3	15.79	25.71	46.89
3.5	15.91	25.94	47.25
4	15.75	25.65	46.92
4.5	15.62	25.13	46.45
5	15.55	24.88	45.94
5.5	15.34	24.65	45.02
6	15.2	24.5	44.8

Table: 6.18 Compressive Strength of M30 Concrete with POLYESTER fibers

% fiber	3 Days	7 Days	28 Days
0	17.02	26.45	45.71
0.5	17.19	26.89	46.05
1	17.31	27.05	46.85
1.5	17.4	27.32	47.25
2	17.52	27.54	47.92
2.5	17.65	27.86	48.15
3	17.82	28.06	48.96
3.5	17.98	28.51	49.5
4	17.8	28.1	48.9
4.5	17.61	27.8	48.05
5	17.45	27.51	47.8
5.5	17.36	27.29	47.12
6	17.2	27.01	46.75

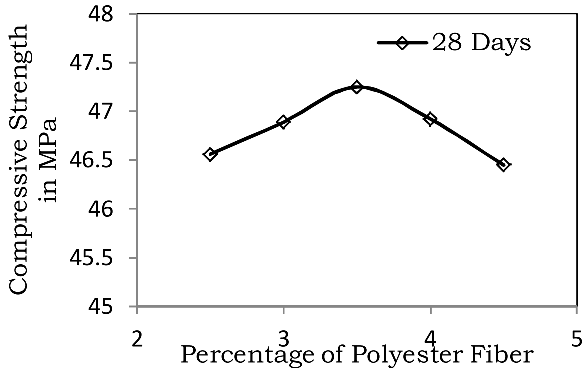

Fig. 6.24 Compressive Strength of M25 Concrete with POLYESTER fibers

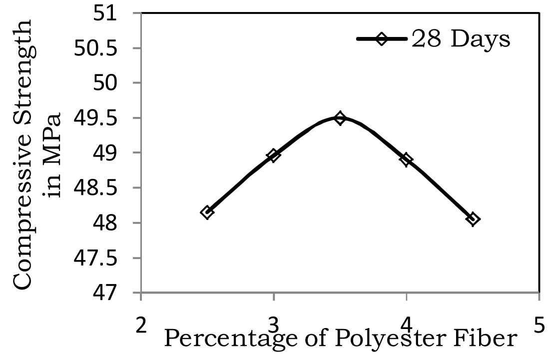

Fig. 6.25 Compressive Strength of M30 Concrete with POLYESTER fibers

Table: 6.19 Split tensile and Flexural Strengths M25 Concrete with POLYESTER fibers

% fiber	Split tensile strength at 28 Days	Flexural strength at 28 Days
0	3.59	4.6
0.5	3.62	4.63
1	3.65	4.68
1.5	3.68	4.71
2	3.69	4.73
2.5	3.71	4.75
3	3.74	4.78
3.5	3.77	4.81
4	3.73	4.77
4.5	3.7	4.74
5	3.67	4.7
5.5	3.65	4.66
6	3.61	4.61

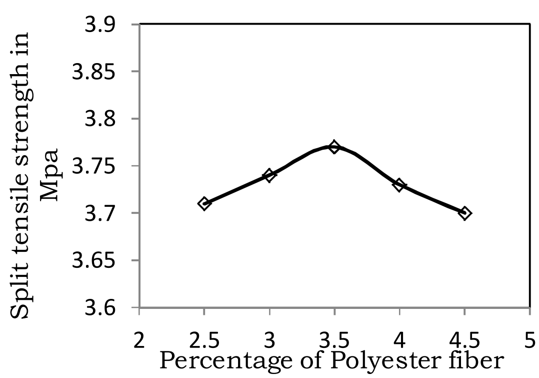

Fig. 6.26 Split Tensile Strength of M25 Concrete with POLYESTER fibers

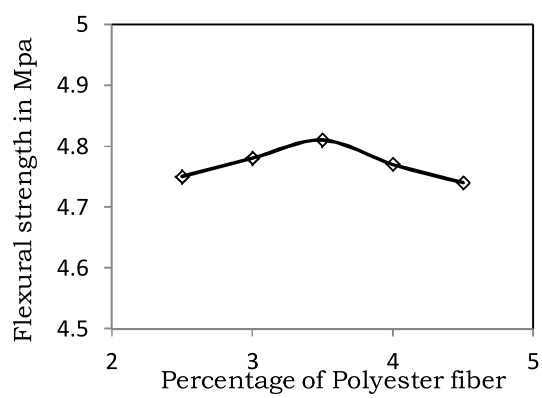

Fig. 6.27 Flexural Strength of M25 Concrete with POLYESTER fibers

Table: 6.20 Split tensile and Flexural Strengths M30 Concrete with POLYESTER fibers

% fiber	Split tensile strength at 28 Days	Flexural strength at 28 Days
0	3.75	4.77
0.5	3.78	4.81
1	3.82	4.84
1.5	3.86	4.86
2	3.89	4.89
2.5	3.91	4.91
3	3.93	4.94
3.5	3.95	4.98
4	3.92	4.93
4.5	3.88	4.9
5	3.85	4.87
5.5	3.81	4.82
6	3.77	4.8

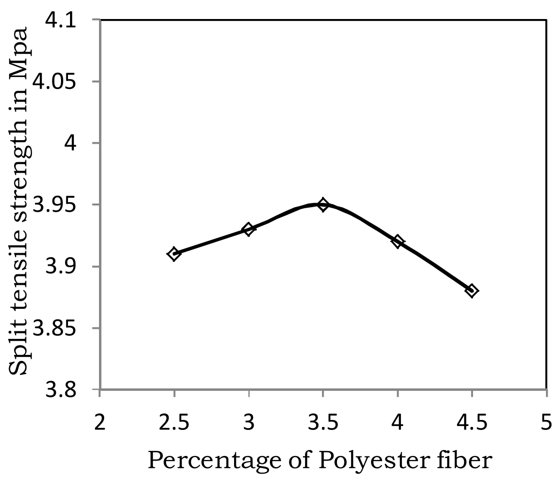

Fig. 6.28 Split Tensile Strength of M30 Concrete with POLYESTER fibers

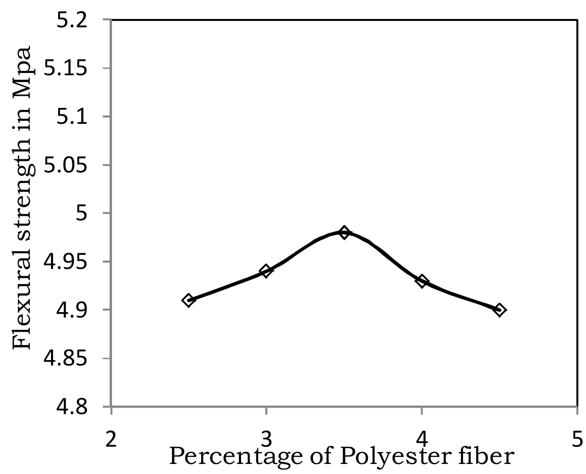

Fig. 6.29 Flexural Strength of M30 Concrete with POLYESTER fibers

The results of modified POLYESTER fiber reinforced concrete are shown in tables 6.17 to 6.20 and figures 6.24 to 6.29. Based on the results the strength properties of fiber reinforced concrete are increases as percentage of fiber increases up to 3.5%, later the strength is decreasing. An improvement of 5.22%, 7.54% and 8.47% in M25 mix and 5.64%, 7.78% and 8.29% in M30 mix was observed when 3days, 7days and 28 days compressive strengths of modified concrete at 3.5% POLYESTER fibers in the concrete. Similarly, the improvement in the split tensile and flexural strengths is 5.01% and 4.56% of M25 concrete and 5.33% and 4.4% for M30 concrete respectively.

Table: 6.21 Nomenclature for modified concrete mixes

Name	**Final concrete mixes of M25 and M30**
C	Concrete without admixture
CA	Concrete with admixture
CAG	CA+ 50% GGBS
CAGR	CA + 50% GGBS + 25% ROBO Sand
CAGRHE	CA + 50% GGBS + 25% ROBO Sand + 3.5% HDPE
CAGRPE	CA + 50% GGBS + 25% ROBO Sand + 3.5% PET
CAGRHD	CA + 50% GGBS + 25% ROBO Sand + 3.5% HDPP
CAGRPO	CA + 50% GGBS + 25% ROBO Sand + 3.5% POLYESTER

A fiber reduces the crack spacing, thus indicating a more redistribution of stresses. As the first crack forms, the fibers bridge it, transmitting stresses across the crack surface. In order to enforce further crack opening the applied load has to be increased, which leads to the formation of another crack. This mechanism then repeats until failure.

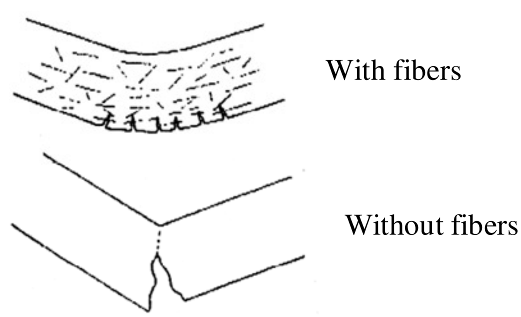

Fig.6.30 Mechanism of increase in flexure strength of concrete with fibers

The concrete strength is increases as the fiber content increases up to 3.5%, this may be due to the better dispersion of fibers in the concrete. Beyond this percentage, there might be fiber balling, meaning the fibers get hooked to each other causing poor dispersion. Because of this reason the strength of concrete is reducing.

Arkan Radi Ali 2013 conducted the experiments on the use of polypropylene fibers in the concrete. 2% of fibers in the concrete were used in this study. The compressive strength is increasing as the fiber content increases in the concrete. The present experimental investigation has been carried out the use of fibers in the concrete beyond 2%. It is found to be the optimum percentage fibers in the concrete are 3.5% (>2% hence in line with previous study).

6.7 STRENGTH CONTRIBUTION OF FIBERS

An approximate calculation of contribution of fibers to concrete strength for this case is presented in Appendix. The calculation is performed under the assumption that all fibers are oriented perpendicular to the crack propagation. The fiber size is 2.5mmX0.1mm approximately.

The maximum percentage of fibers is 3.5% volume of cement.

The volume of fibers in a 150mm cube is $0.012 \times 10^{-3} \mathrm{m}^3$.

Tensile load carrying capacity of HDPE fiber is 67.79kg or 677.9N.

The increase in the strength is $\frac{0.012 \times 10^{-3}}{0.15 \times 0.15 \times 0.15} \times \frac{1}{0.1 \times 2.5} \times 677.9 = 9.62\mathrm{MPa}$

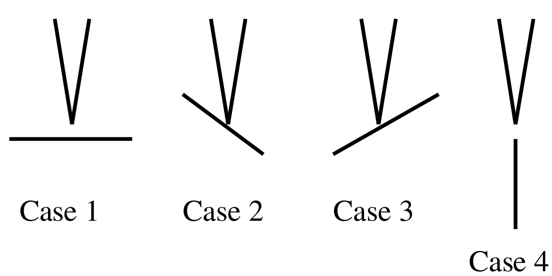

Fig. 6.31 fiber orientation with respect to crack tip

From the above figure, assume that there are four cases fiber orientation at the crack tip is in equal distribution (25%). The strength contribution for all four cases is given below table.

Table: 6.22 fiber strength contribution

Cases	Percentage of Strength contribution	Remarks
Case 1	100%	In direction of crack opening
Case 2	50%	Partially in direction of crack opening
Case 3	50%	
Case 4	0%	Not in direction of crack opening

From the above table clearly shows the 50% (25%X100% +25%X50% + 25%X50% + 25%X0%) of total fibers are only contributing towards the strength development. Conservatively it is assumed that only 40% fibers (all of which are assumed to be perpendicular to crack direction) contribute to strength development. So the strength improvement due to the fibers can be approximated to 40% of the above (9.62 MPa) value i.e. 3.85 MPa. From the experimental investigation, the strength improvement due to the HDPE fibers is 3.95 MPa. Below table shows the strength improvements of different fibers in the concrete.

Table: 6.23 Strength contribution due to fibers theoretical and experimental results

S. No.	Fiber type	Strength improvement (MPa) M25 concrete		Strength improvement (MPa) M30 concrete	
		Theoretical	Experimental	Theoretical	Experimental
1	HDPE	3.85	3.95	4.21	4.04
2	PET	4.088	4.03	4.48	4.09
3	HDPP	3.656	3.75	4.00	3.89
4	POLYESTER	3.42	3.69	3.74	3.79

From the above table it shown that there is good correlation between theoretical results and experimental results.

6.7 SUMMARY

Based on the present experimental study, the following points can be summarized.

1. The strength properties are increasing as percentage of fiber increases up to 3.5%. After this strength is reducing as the percentage of fiber increases.
2. The compressive strength, split tensile strength and flexural strength of concrete increased by 9.07, 6.12 and 5.65% respectively with 3.5% of HDPE fiber in the M25 concrete. Similarly in case of M30 concrete strengths are increased by 8.84, 6.93% and 5.87% at the age of 28 days respectively when compared to CAGR mix.
3. The increase in the 28 days compressive strength, split tensile and flexural strength of PET fiber reinforced M25 concrete is 9.25%, 6.96% and 6.09% respectively. For M30 Grade concrete 8.95%, 7.46% and 6.29% increased compressive strengths, split tensile strength and flexural strength respectively when compared to CAGR mix.
4. The modified concrete (with HDPP fibers) strengths i.e. compressive strength, split tensile and flexural strengths of concrete is increased by 8.61%, 5.57%, and 5.22% for M25 concrete and M30 concrete 8.51%, 6.13% and 5.03% respectively compared with concrete without fibers at the age of 28 days when compared to CAGR mix.
5. Similarly for Polyester fiber reinforced concrete, the increase in the 28 days compressive strength, split tensile strength and flexural strengths are 8.47%, 5.01% and 4.57% for M25 concrete and for M30 concrete 8.29%, 5.33% and 4.4% respectively when compared to CAGR mix.

Fig.6.32 HDPE Fibers

Fig.6.33 PET Fibers

Fig.6.34 HDPP Fibers

Fig.6.35 POLYESTER Fibers

Fig.6.36 Cubes for compressive strength

Fig.6.37 Cylinders for split tensile strength


Fig.6.38 Beams for flexural strength

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CHAPTER 7

STRESS-STRAIN BEHAVIOUR OF MODIFIED CONCRETE

7.1 INTRODUCTION

Graph obtained by drawing a curve for the values of stresses and strains obtained during testing a material specimen is called a stress - strain curve. By testing cylinders of standard size made with concrete, under uni-axial compression values of stresses and strains are obtained and the stress-strain curves are plotted. Even though the stress strain relation for cement paste and aggregate when tested individually is practically linear, it is observed from the stress-strain plots of concrete that, no portion of the curves is in the form of a straight line. In concrete the rate of increase of stress is less than that of increase in strain because of the formation of micro cracks, between the interfaces of the aggregate and the cement paste. Thus the stress strain curve is not linear. In conventional concrete the value of stress is maximum corresponding to a strain of about 0.002 and further goes on decreasing with the increasing strain, giving a dropping curve till it terminates at ultimate crushing strain.

7.2 EXPERIMENTAL INVESTIGATIONS

Cylinders made with different selected design concrete mix proportions with and without the addition of GGBS, ROBO Sand and fibers were tested for stress-strain behavior under uni-axial compression. Three cylinders for each mix were cast, tested under uni-axial compression and the average of three cylinders were taken for obtaining the stress-strain behavior of each design concrete mix. Thus stress-strain curves for all design concrete mixes with different percentage of fibers with GGBS and ROBO sand were plotted. The experimental values of stress and strain for

M25 and M30 design mix concrete with and without GGBS, ROBO sand and fibers given in tables 7.1 to 7.6.

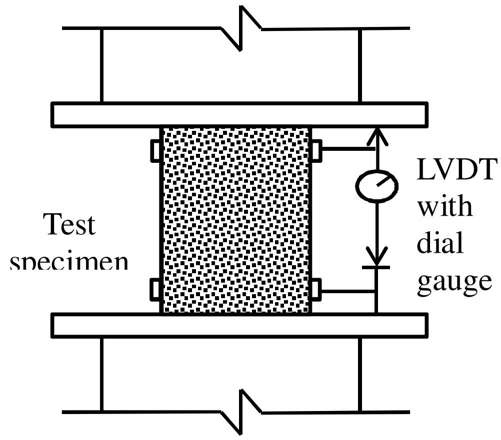

Fig. 7.1 Test setup

Table: 7.1 Experimental values of stress – strain for M25 design mix concrete with and without GGBS and ROBO sand

M25 CA concrete		
S. No	Strain	Stress N/mm2
1	0	0
2	0.00007	2.16
3	0.0002	4.41
4	0.00051	5.98
5	0.00075	8.25
6	0.00121	12.02
7	0.00149	13.99
8	0.00168	16.03
9	0.00192	17.29
10	0.00228	20.43
11	0.00263	21.83
12	0.00314	24.12
13	0.00351	25.42
14	0.00399	28.18
15	0.00441	28.74
16	0.00492	28.32
17	0.00518	27.42
18	0.00537	24.04
19	0.00549	21.25

M25 CAG concrete		
S. No	Strain	Stress N/mm2
1	0	0
2	0.00009	2.25
3	0.00031	4.39
4	0.00061	6.61
5	0.00088	8.51
6	0.00124	11.72
7	0.00152	13.89
8	0.00191	16.11
9	0.00216	17.75
10	0.00234	19.54
11	0.00274	21.78
12	0.00346	24.1
13	0.00389	25.98
14	0.00431	28.59
15	0.00471	29.86
16	0.005	30.58
17	0.00522	28.98
18	0.00537	27.12
19	0.00546	24.98

M25 CAGR concrete		
S. No	Strain	Stress N/mm2
1	0	0
2	0.00009	2.24
3	0.00024	4.42
4	0.00061	6.45
5	0.00099	8.79
6	0.00136	11.59
7	0.00179	14.79
8	0.00209	17.54
9	0.00229	18.98
10	0.00244	20.49
11	0.00286	22.32
12	0.00332	25.12
13	0.00406	28.15
14	0.00466	29.54
15	0.00496	30.59
16	0.00548	31.95
17	0.00574	32.87
18	0.00599	33.98
19	0.00626	33.45
20	0.00644	30.98

21	0.00664	26.92

Table: 7.2 Experimental values of stress – strain for M30 design mix concrete with and without GGBS and ROBO sand

M30 CA concrete		
S. No	Strain	Stress N/mm2
1	0	0
2	0.00014	2.41
3	0.00036	6.01
4	0.00055	8.89
5	0.00099	12.62
6	0.0012	14.02
7	0.00133	16.89
8	0.00148	19.68
9	0.00179	22.19
10	0.0021	24.98
11	0.0024	28.12
12	0.0029	30.25
13	0.0033	33.69
14	0.0038	35.98
15	0.0041	37.54
16	0.0043	39.59
17	0.0045	41.98
18	0.0048	43.11
19	0.0051	39.98
20	0.0055	35.87
21	0.0059	33.69

M30 CAG concrete		
S. No	Strain	Stress N/mm2
1	0	0
2	0.00026	2.54
3	0.00054	5.98
4	0.00098	8.88
5	0.0011	11.61
6	0.0013	13.45
7	0.0015	16.87
8	0.0017	19.82
9	0.00189	22.13
10	0.0022	24.36
11	0.0025	26.54
12	0.00261	29.01
13	0.003	32.01
14	0.0032	33.98
15	0.00338	35.45
16	0.00354	36.83
17	0.00382	37.56
18	0.0041	40.05
19	0.0044	42.94
20	0.0045	44.05
21	0.00471	45.64
22	0.0051	46.28
23	0.00541	44.12
24	0.00565	40.89

M30 CAGR concrete		
S. No	Strain	Stress N/mm2
1	0	0
2	0.0004	2.71
3	0.00072	5.91
4	0.00092	9.01
5	0.00118	11.59
6	0.0013	13.69
7	0.0016	16.38
8	0.0019	19.81
9	0.0021	22.51
10	0.0023	24.81
11	0.00264	26.94
12	0.00278	29.08
13	0.00312	32.09
14	0.00336	32.95
15	0.00355	34.58
16	0.00371	36.12
17	0.00401	38.84
18	0.00422	41.28
19	0.0045	44.68
20	0.00502	47.86
21	0.0054	48.26
22	0.0057	48.97
23	0.00608	47.01
24	0.00631	42.11

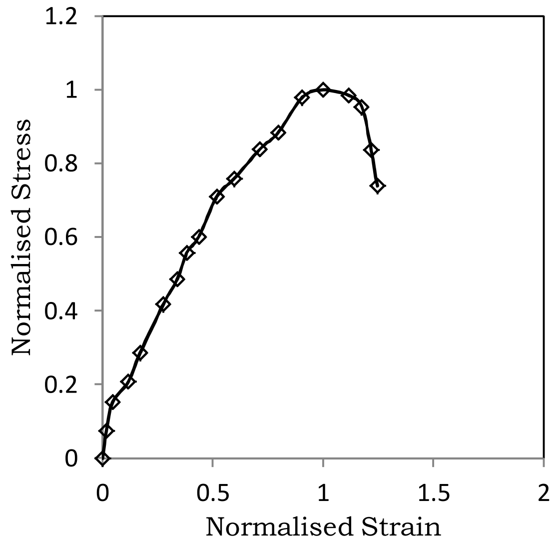

Fig. 7.2 Stress – strain behavior of M25 CA concrete

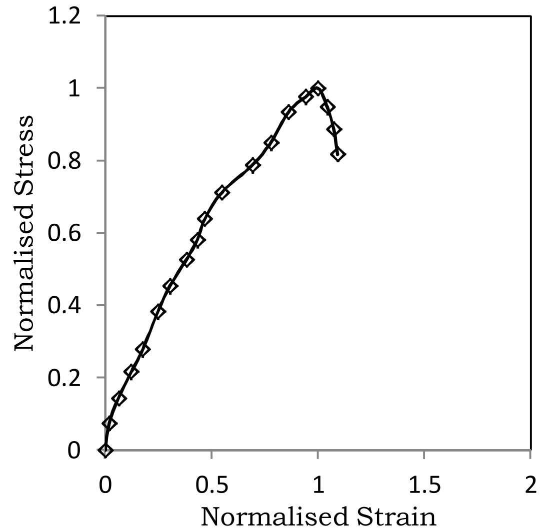

Fig. 7.3 Stress – strain behavior of M25 CAG concrete

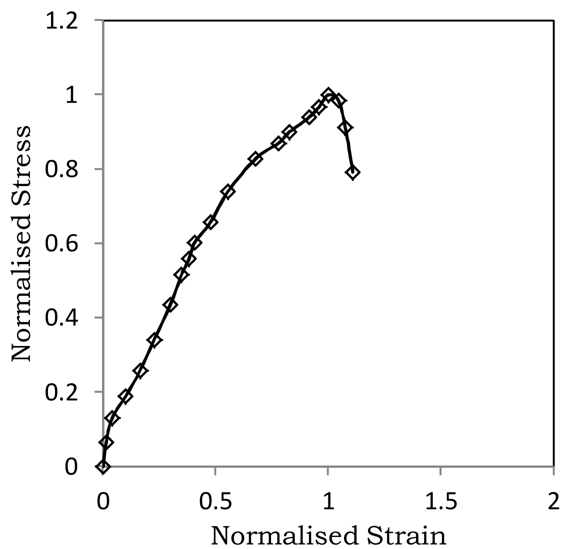

Fig. 7.4 Stress – strain behavior of M25 CAGR concrete

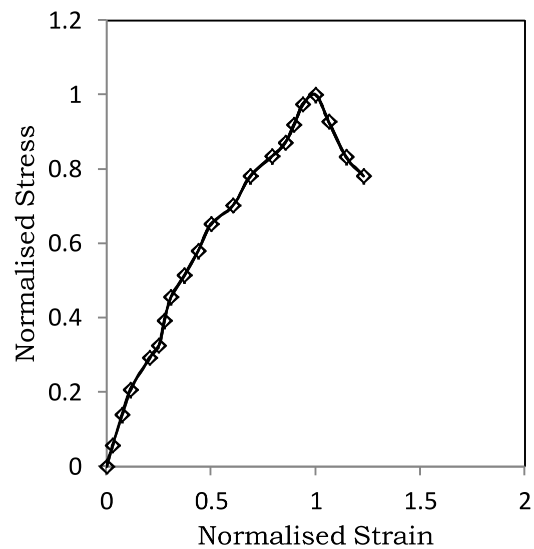

Fig. 7.5 Stress – strain behavior of M30 CA concrete

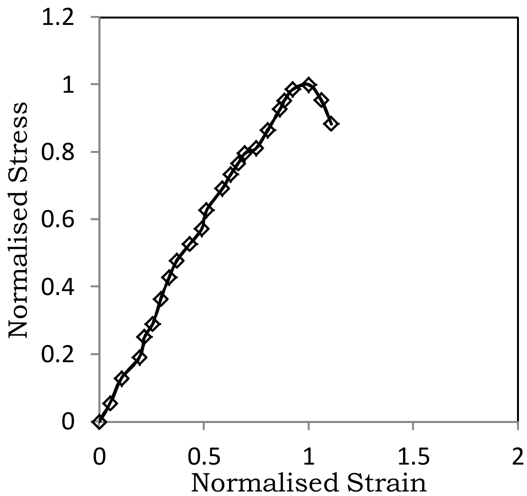

Fig. 7.6 Stress – strain behavior of M30 CAG concrete

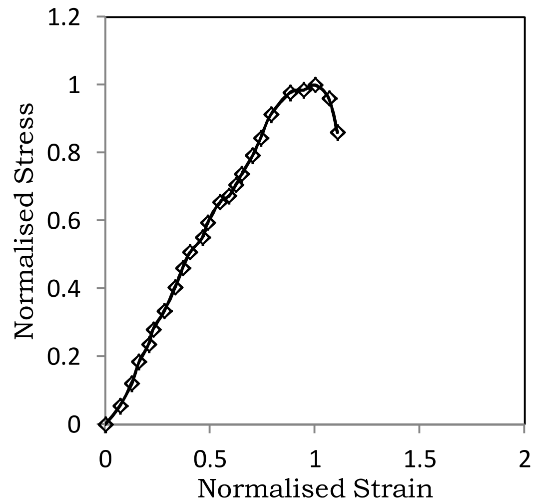

Fig. 7.7 Stress – strain behavior of M30 CAGR concrete

Figures 7.1 to 7.6 shows the normalized stress and normalized strain behavior of M25 and M30 design mixes with and without GGBS and ROBO sand.

Table: 7.3 Experimental values of stress – strain for M25 concrete with HDPE and PET Fibers

M25 CAGRHE concrete		
S. No	Strain	Stress N/mm2
1	0	0
2	0.00011	2.24
3	0.00028	4.42
4	0.00068	6.45
5	0.00119	8.79
6	0.00141	11.59
7	0.00185	14.79
8	0.00219	17.54
9	0.00249	19.09
10	0.00254	20.49
11	0.00291	22.41
12	0.00351	25.12
13	0.00416	28.15
14	0.00471	29.54

M25 CAGRPE concrete		
S. No	Strain	Stress N/mm2
1	0	0
2	0.00012	2.25
3	0.00031	4.42
4	0.00071	6.48
5	0.00122	8.81
6	0.00144	11.59
7	0.00187	14.89
8	0.00222	17.54
9	0.00252	19.19
10	0.00259	20.49
11	0.00298	22.41
12	0.0036	25.22
13	0.00421	28.15
14	0.00481	29.54

15	0.00506	30.59	15	0.00517	30.69
16	0.00558	32.11	16	0.00559	32.11
17	0.00589	33.05	17	0.00598	33.15
18	0.00645	34.49	18	0.00655	34.52
19	0.00661	33.41	19	0.00681	33.51
20	0.00684	31.89	20	0.00691	32.01
21	0.00719	27.45	21	0.00721	27.98

Table: 7.4 Experimental values of stress – strain for M25 concrete with HDPP and POLYESTER Fibers

M25 CAGRHD concrete			M25 CAGRPO concrete		
S. No	Strain	Stress N/mm2	S. No	Strain	Stress N/mm2
1	0	0	1	0	0
2	0.00015	2.25	2	0.00014	2.21
3	0.00032	4.41	3	0.0003	4.43
4	0.00081	6.51	4	0.00079	6.54
5	0.00132	8.88	5	0.00129	8.85
6	0.00154	11.69	6	0.00149	11.71
7	0.00199	14.98	7	0.00191	14.92
8	0.00242	17.61	8	0.00241	17.7
9	0.00262	19.39	9	0.00259	19.4
10	0.00279	20.58	10	0.00271	20.61
11	0.00298	22.54	11	0.00289	22.59
12	0.00381	25.32	12	0.00375	25.29
13	0.00441	28.61	13	0.00431	28.69
14	0.00491	29.54	14	0.00489	29.61
15	0.00537	30.81	15	0.00537	30.89
16	0.00572	32.11	16	0.00569	32.18
17	0.0063	33.25	17	0.00632	33.31
18	0.00669	34.62	18	0.00661	34.68
19	0.00696	33.51	19	0.00692	33.64
20	0.00719	32.11	20	0.00729	32.21
21	0.00741	27.81	21	0.00732	27.89

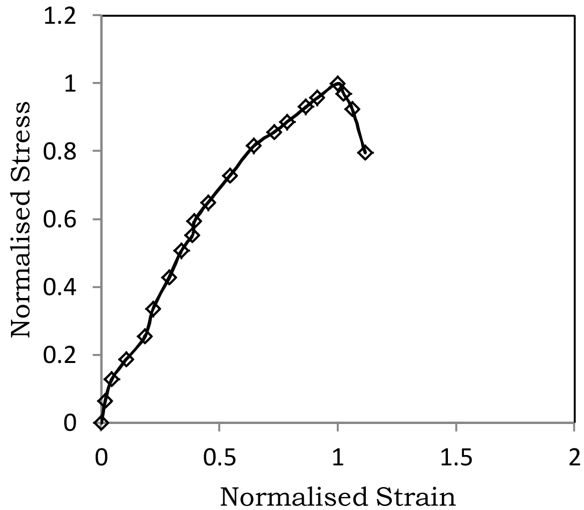

Fig. 7.8 Stress – strain behavior of M25 CAGRHE concrete

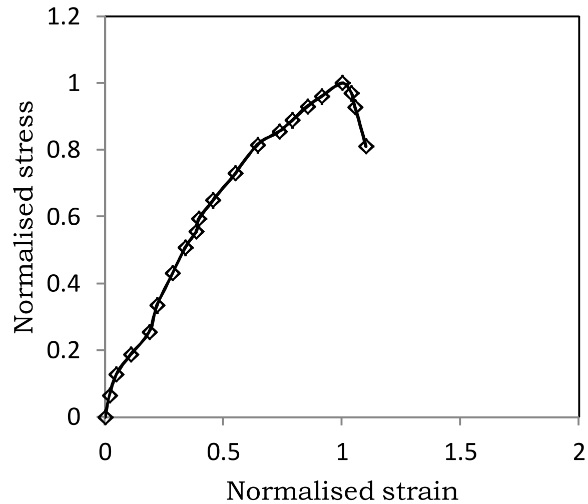

Fig. 7.9 Stress – strain behavior of M25 CAGRPE concrete

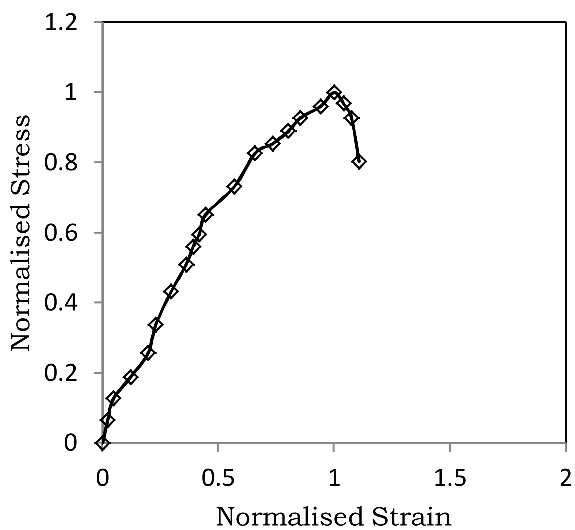

Fig. 7.10 Stress – strain behavior of M25 CAGRHD concrete

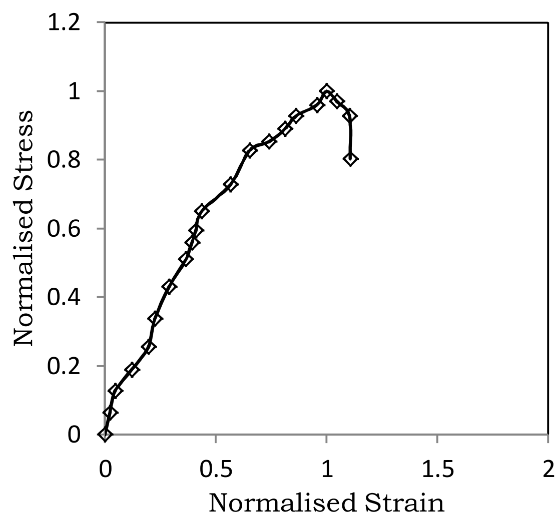

Fig. 7.11 Stress – strain behavior of M25 CAGRPO concrete

Table: 7.5 Experimental values of stress – strain for M30 concrete with HDPE and PET Fibers

M30 CAGRHE concrete		
S. No	Strain	Stress N/mm2
1	0	0
2	0.00042	2.78
3	0.00075	6.02
4	0.00094	9.09
5	0.00122	11.71
6	0.00136	13.91
7	0.00168	16.58
8	0.00199	19.95
9	0.00218	22.89
10	0.00241	25.09
11	0.00274	27.16
12	0.00289	29.86
13	0.00322	32.89
14	0.00343	33.98
15	0.00369	34.89
16	0.00384	36.59
17	0.00422	39.09
18	0.00442	41.39
19	0.00479	44.89
20	0.00516	48.69
21	0.00559	49.21
22	0.00595	49.98
23	0.00635	47.25
24	0.00662	43.89

M30 CAGRPE concrete		
S. No	Strain	Stress N/mm2
1	0	0
2	0.00044	2.82
3	0.00077	6.05
4	0.00098	9.09
5	0.00129	11.71
6	0.00141	14.12
7	0.00172	16.81
8	0.00208	20.15
9	0.00228	23.12
10	0.00249	25.81
11	0.00289	27.89
12	0.0031	30.29
13	0.00342	33.89
14	0.00363	34.99
15	0.00381	35.89
16	0.00409	37.59
17	0.00431	39.69
18	0.00453	42.59
19	0.00486	45.91
20	0.00526	49.79
21	0.00571	52.54
22	0.00612	53.18
23	0.00652	48.59
24	0.00682	44.91

Table: 7.6 Experimental values of stress – strain for M30 concrete with HDPP and POLYESTER Fibers

M30 CAGRHD concrete		
S. No	Strain	Stress N/mm2
1	0	0

M30 CAGRPO concrete		
S. No	Strain	Stress N/mm2
1	0	0

2	0.00043	2.83
3	0.00075	6.05
4	0.00095	9.15
5	0.00124	11.79
6	0.00138	14.22
7	0.00169	16.89
8	0.00205	20.21
9	0.00224	23.32
10	0.00249	25.88
11	0.0029	27.91
12	0.00312	30.39
13	0.00344	34.05
14	0.00369	35.08
15	0.00388	36.89
16	0.00416	37.96
17	0.00439	39.89
18	0.00458	42.78
19	0.00491	46.19
20	0.00529	49.98
21	0.00576	52.69
22	0.00622	53.69
23	0.00654	48.62
24	0.00689	44.21

2	0.00043	2.81
3	0.00076	6.03
4	0.00098	9.11
5	0.00124	11.71
6	0.00139	14.19
7	0.00171	16.91
8	0.00208	20.23
9	0.00228	23.34
10	0.00251	25.98
11	0.00296	27.96
12	0.00322	30.41
13	0.00348	34.06
14	0.00371	35.18
15	0.0039	36.9
16	0.00421	37.98
17	0.00441	39.91
18	0.00461	42.82
19	0.00496	46.29
20	0.00539	49.99
21	0.00586	52.79
22	0.00632	53.71
23	0.00659	48.59
24	0.00696	44.25

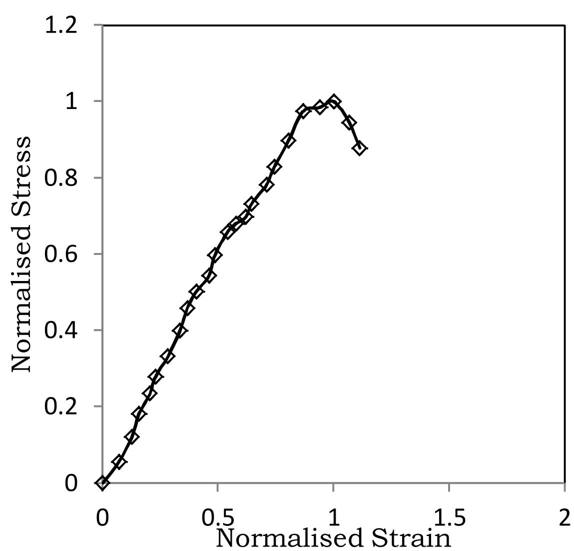

Fig. 7.12 Stress – strain behavior of M30 CAGRHE concrete

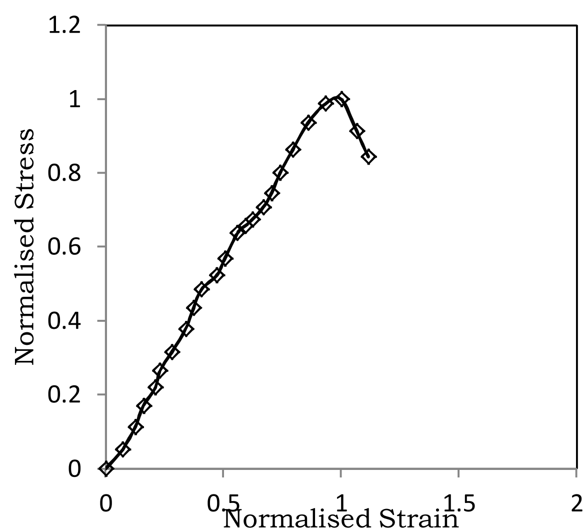

Fig. 7.13 Stress – strain behavior of M30 CAGRPE concrete

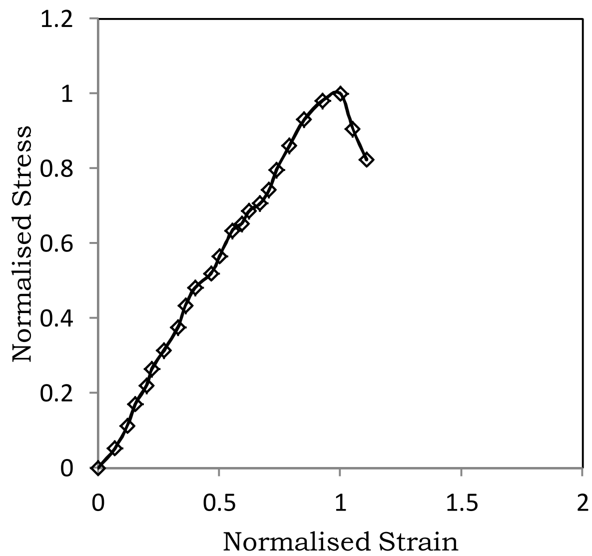

Fig. 7.14 Stress – strain behavior of M30 CAGRHD concrete

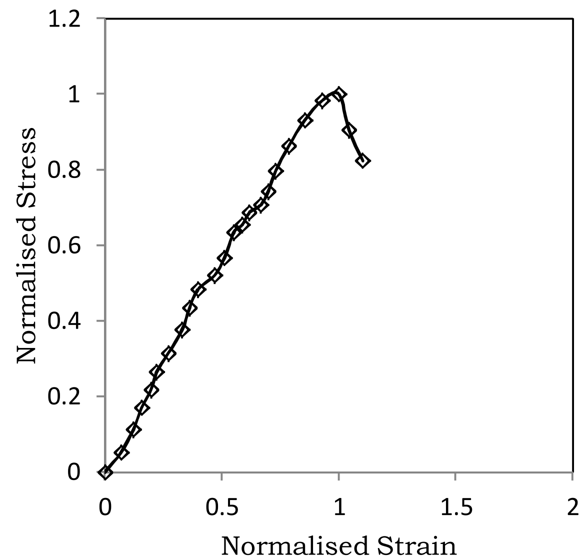

Fig. 7.15 Stress – strain behavior of M30 CAGRPO concrete

Table: 7.7 Peak stress and strain at peak stress of different mixes

MIX		Peak Stress	Strain at peak stress
concrete	**Type**		
M25	CA	28.74	0.0044
	CAG	30.58	0.005
	CAGR	33.98	0.006
	CAGRHE	34.49	0.0065
	CAGRPE	34.52	0.0066
	CAGRHD	34.62	0.0067
	CAGRPO	34.68	0.0066
M30	CA	43.11	0.0048
	CAG	46.28	0.0051
	CAGR	48.97	0.0057

M30	CAGRHE	49.98	0.006
	CAGRPE	53.18	0.0061
	CAGRHD	53.69	0.0062
	CAGRPO	53.71	0.0063

Table no. 7.7 shows the peak stresses and strains at peak stress. If observe the table, the peak stresses and strains at peak stress are increased up to 20% and 50% respectively for M25 concrete when compared to conventional design mix. Similarly for M30 concrete, the increase in the peak stress and strain at peak stress are 24.5% and 31% respectively.

7.3 MATHEMATICAL MODELING FOR STRESS-STRAIN BEHAVIOUR OF CONCRETE

After obtaining the stress-strain behavior of design concrete mixes with combination of GGBS and ROBO Sand experimentally, an attempt was made to get the analytical stress-strain curves for design concrete mixes.

To represent uni-axial stress-strain behavior of conventional concrete, number of empirical equations has been proposed but most of them can be used for only ascending portion of the curve. Carriera and Chu (1985) extended the empirical equation proposed by Popovics in 1973, which includes both ascending and descending portions of complete stress-strain curve. Most of the equations proposed were for conventional concrete.

Considering this gap in existing literature an attempt has been made to develop empirical equations for design concrete mix with and without GGBS, ROBO Sand and fibers.

7.3.1 Non-Dimensional (normalized) Stress-Strain Curves

The stress-strain curves indicate that, the behavior is similar for all the specimens. The similarity leads to the conclusion that there is only a unique shape of the stress-strain diagram, if expressed in a non-dimensional form, along both the axes. The said form can be obtained by dividing the stress at any level by peak stress and the strain at any level by peak strain. Thus all the stress-strain curves will have same point (1, 1) at peak stress. By non- dimensionalising the stresses and strains as above the behavior can be represented as a general behavior.

The stress – strain curves obtained experimentally for design concrete were normalized as specified above and normalized stress-strain values were calculated for all design concrete mixes.

Two design concrete mixes with GGBS, ROBO Sand and fibers, taken for investigation are of M25 and M30 grade mixes. A single normalized stress-strain curve is developed for the combination of three M25 design mixes and three M30 design mixes taking the average values of normalized stresses and strains. The normalized stress and strain curves presented from the fig.7.1 to 7.14.

7.3.2 Models Available for Stress-Strain Curves of Conventional Concrete

Many researchers developed various models for the prediction of stress-strain behavior of concrete. Some of the models are given below.

1) **Desay's and Krishnan's model (1964)**

For Normal strength concrete, stress-strain relationship is given by

$$Y = \frac{AX}{1 + BX^2}$$

Where Y = The Normalized stress

X = Normalized strain and

A, B are the constants and they can be find out by using boundary conditions. This model is valid only up to ascending branch of stress-strain curve.

2) **Saenz's Model (1964)**

With reference to Desay's model Saenz proposed a model by taking into account both the ascending and descending portions of the stress-strain curve. This model is in the form of

$$Y = \frac{AX}{1 + BX + CX^2}$$

Where Y= (σ / σ_u) and X = $(\varepsilon/\varepsilon_u)$

3) **Kent and Park Model (1971)**

For Normal strength concrete up to ascending portion:

The stress-strain model is $f_c = f_c'\{2(\varepsilon/\varepsilon_o) - (\varepsilon/\varepsilon_o)^2\}$

f_c' = compressive strength N/mm^2 ε_o = strain at peak stress

4) **Wang et.al. Model (1978)**

The model used by Wang et.al. is in the form of

$$f_c = f_c'\left\{\frac{A(\varepsilon/\varepsilon_o) + B(\varepsilon/\varepsilon_o)^2}{1 + C(\varepsilon/\varepsilon_o) + D(\varepsilon/\varepsilon_o)^2}\right\}$$

However instead of using one set of the coefficients A, B, C, and D to generate the complete curve, Wang et.al, used two sets of coefficients – one for the ascending branch and the other for the descending branch. The respective coefficients being obtained from the relevant boundary conditions assigned to each part of the curve.

5) **Carreria and Chu's Model (1985)**

This model is in the form of

$$f_c = f_c' \left\{ \frac{\beta(\varepsilon/\varepsilon_o)}{\beta - 1 + D(\varepsilon/\varepsilon_o)^{\beta}} \right\}$$

$$\text{In which } \beta = 1 - \left(f_c'/\varepsilon_o\, E_{it}\right)$$

Where f_c' = cylinder ultimate compressive strength

ε_o = strain at ultimate stress

E_{it} = initial tangent modulus

7.3.3 Proposed Model for Stress-Strain Behavior

Equations in different forms were tried to get the complete stress-strain behavior of design concrete mixes. From the results obtained, the second order regression equation was postulated in order to derive the relation between the stress and strain. The proposed equation is transformed form of wang's model is in the form of

$$Y = \frac{AX + BX^2}{1 + CX + DX^2}$$

X – Normalized strain; Y – Normalized stress; A, B, C and D – Constants

Using non-dimensional stress-strain curves, constants for different design concrete mixes are determined and from that the equations are developed. Ultimately analytical equations giving the complete stress-strain behavior are developed for M25 and M35 grade concrete with GGBS, ROBO Sand and fibers.

The constants for design concrete mixes are given in the table 7.8 and the equations for the design concrete mixes are given in the table 7.9.

Table: 7.8 constants of A, B, C and D for the different design mix design with and without GGBS, ROBO sand and fibers

MIX		Constants for design concrete mixes			
Concrete	**Type**	**A**	**B**	**C**	**D**
M25	CA	1.904	-1.474	1.667	-0.583
	CAG	1.841	-1.650	-0.106	-0.697
	CAGR	1.933	-1.706	-0.017	-0.750
	CAGRHE	1.864	-1.606	-0.131	-0.603
	CAGRPE	1.869	-1.641	-0.130	-0.637
	CAGRHD	1.779	-1.556	-0.209	-0.564
	CAGRPO	1.944	-1.748	0.035	-0.839
M30	CA	1.615	-1.186	-0.253	-0.289
	CAG	1.266	-1.061	-0.726	-0.067
	CAGR	1.211	-1.022	-0.818	0.006
	CAGRHE	1.207	-0.996	-0.805	0.018
	CAGRPE	1.091	-0.914	-0.899	0.080
	CAGRHD	1.134	-0.966	-0.856	0.029
	CAGRPO	1.133	-0.969	-0.870	0.039

Table: 7.9 Proposed equations for different design mix design with and without GGBS, ROBO sand and fibers

MIX		Proposed Equations
Concrete	**Type**	
M25	CA	$Y = \frac{1.094X - 1.474X^2}{1 + 1.667X - 0.583X^2}$
	CAG	$Y = \frac{1.841X - 1.65X^2}{1 - 0.106X - 0.697X^2}$
	CAGR	$Y = \frac{1.933X - 1.706X^2}{1 - 0.017X - 0.75X^2}$

M25	CAGRHE	$Y = \dfrac{1.864X - 1.606X^2}{1 - 0.131X - 0.603X^2}$
	CAGRPE	$Y = \dfrac{1.869X - 1.641X^2}{1 - 0.13X - 0.637X^2}$
	CAGRHD	$Y = \dfrac{1.779X - 1.556X^2}{1 - 0.209X - 0.564X^2}$
	CAGRPO	$Y = \dfrac{1.944X - 1.748X^2}{1 + 0.035X - 0.839X^2}$
M30	CA	$Y = \dfrac{1.615X - 1.186X^2}{1 - 0.253X - 0.289X^2}$
	CAG	$Y = \dfrac{1.266X - 1.061X^2}{1 - 0.726X - 0.067X^2}$
	CAGR	$Y = \dfrac{1.211X - 1.022X^2}{1 - 0.818X + 0.006X^2}$
	CAGRHE	$Y = \dfrac{1.207X - 0.996X^2}{1 - 0.805X + 0.018X^2}$
	CAGRPE	$Y = \dfrac{1.091X - 0.914X^2}{1 - 0.899X + 0.08X^2}$
	CAGRHD	$Y = \dfrac{1.134X - 0.966X^2}{1 - 0.856X + 0.029X^2}$
	CAGRPO	$Y = \dfrac{1.133X - 0.969X^2}{1 - 0.87X + 0.039X^2}$

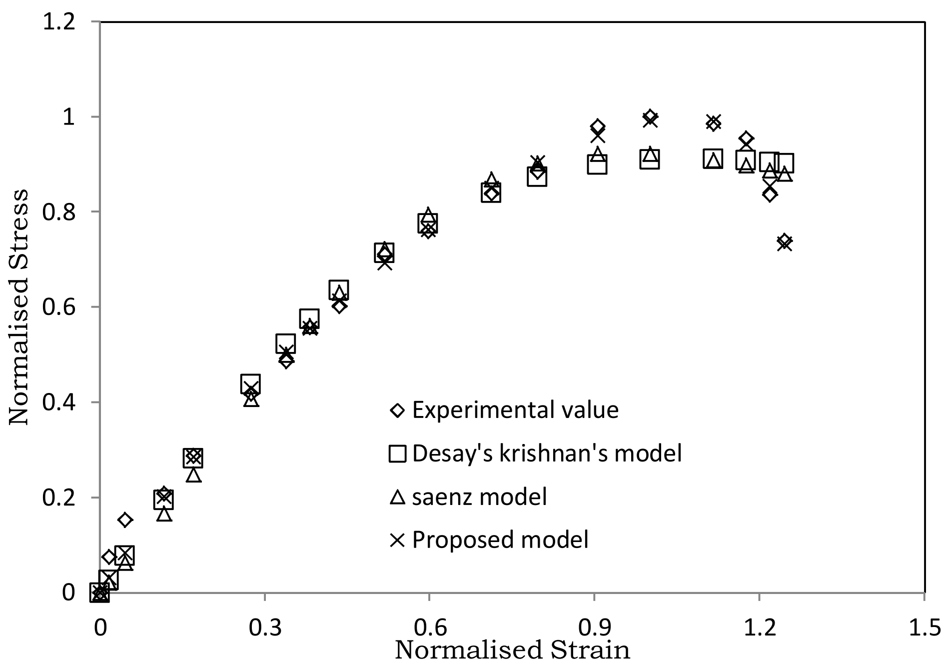

Fig. 7.16 Comparison of stress – strain behavior of M25 CA concrete

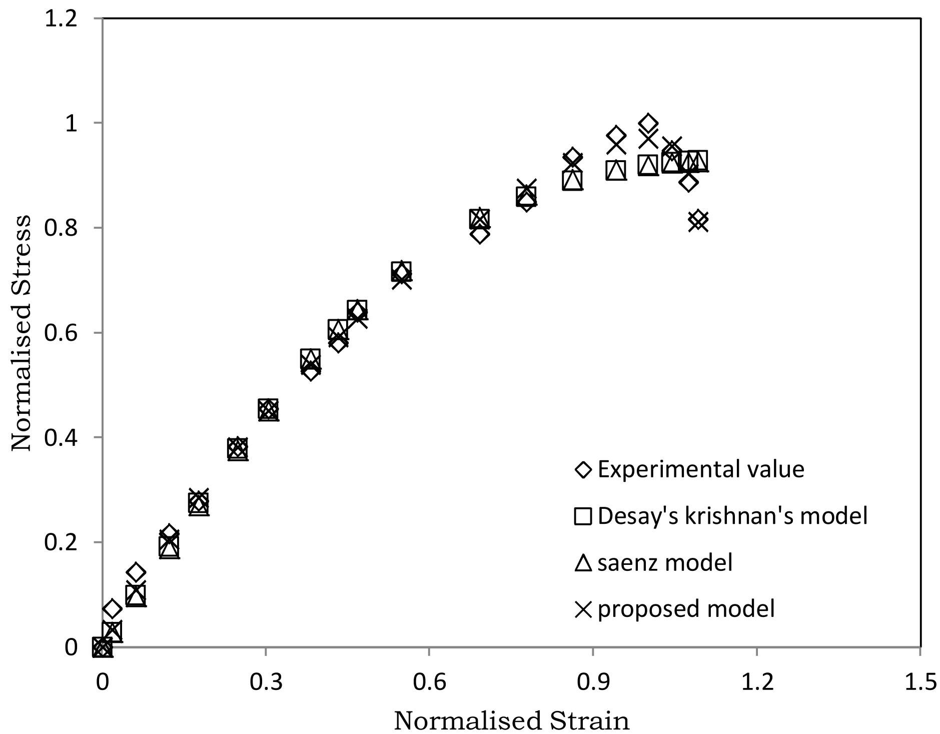

Fig. 7.17 Comparison of stress – strain behavior of M25 CAG concrete

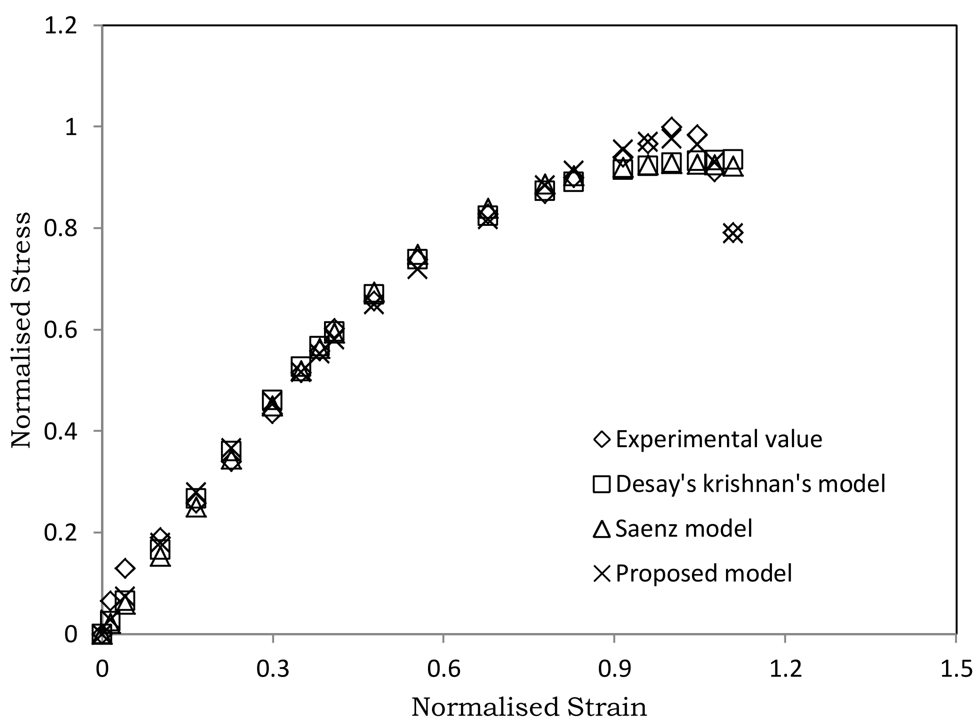

Fig. 7.18 Comparison of stress – strain behavior of M25 CAGR concrete

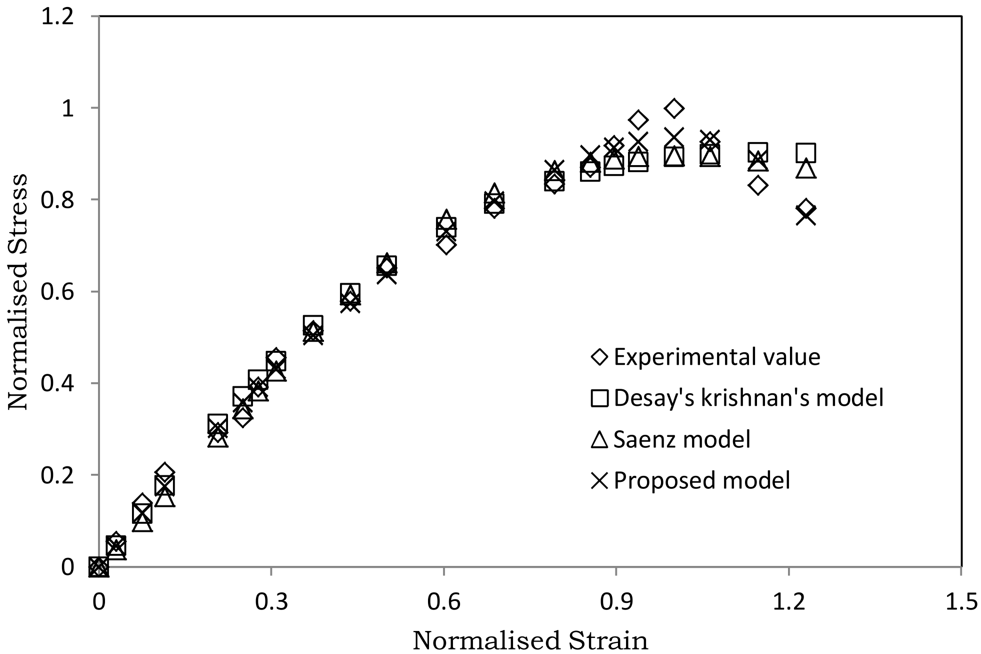

Fig. 7.19 Comparison of stress – strain behavior of M30 CA concrete

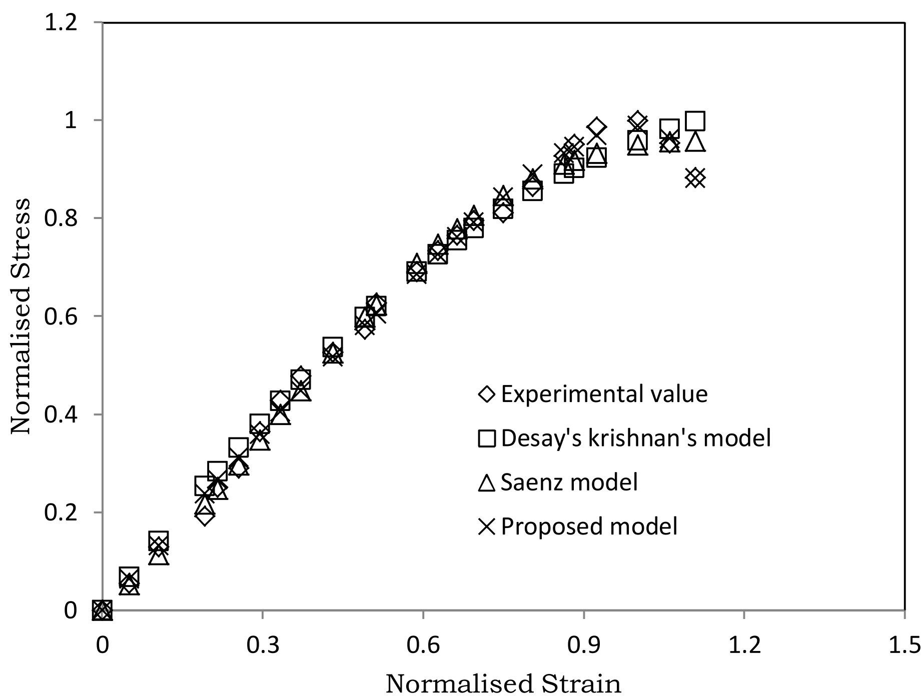

Fig. 7.20 Comparison of stress – strain behavior of M30 CAG concrete

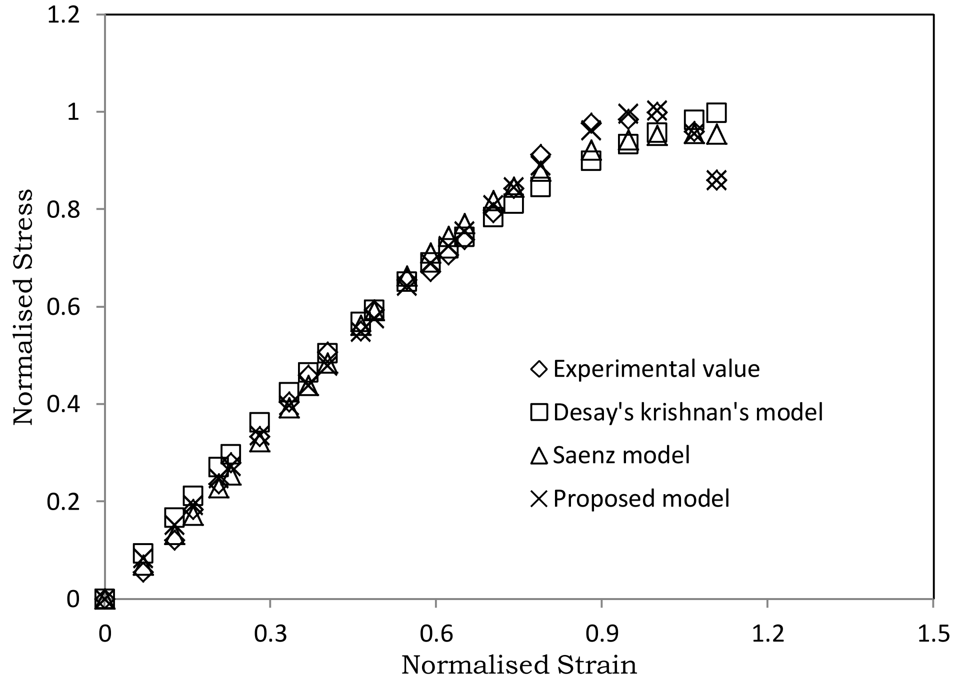

Fig. 7.21 Comparison of stress – strain behavior of M30 CAGR concrete

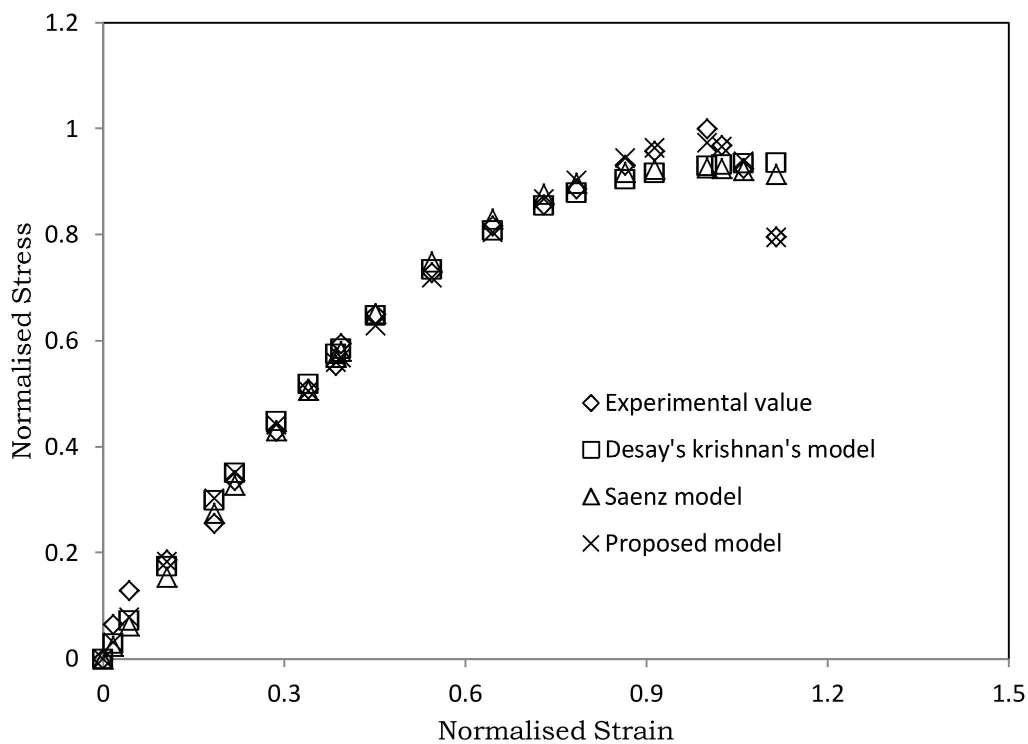

Fig. 7.22 Comparison of stress – strain behavior of M25 CAGRHE concrete

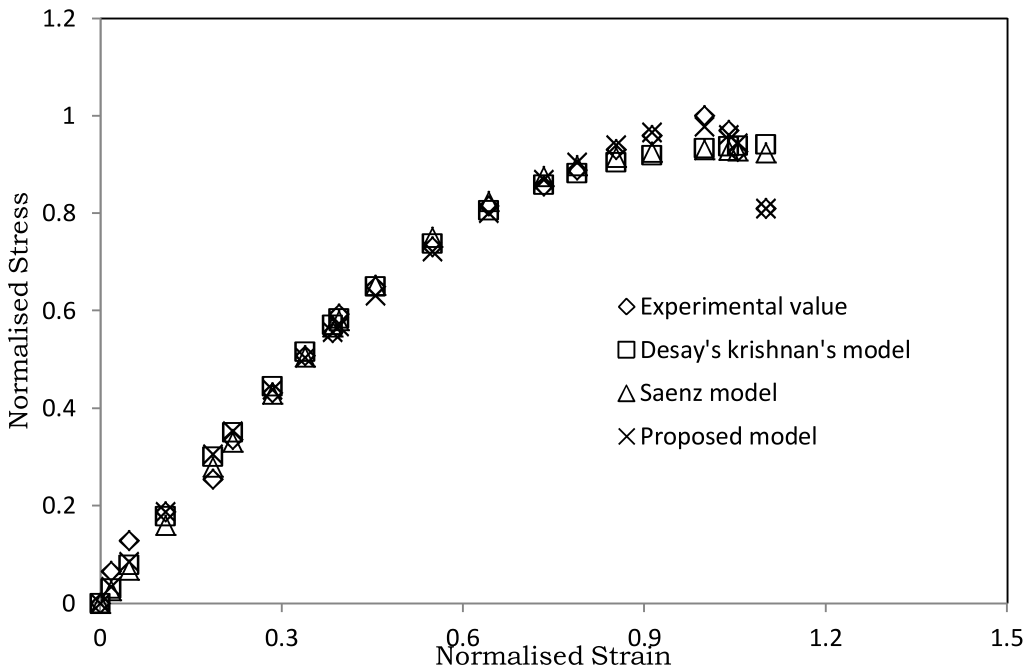

Fig. 7.23 Comparison of stress – strain behavior of M25 CAGRPE concrete

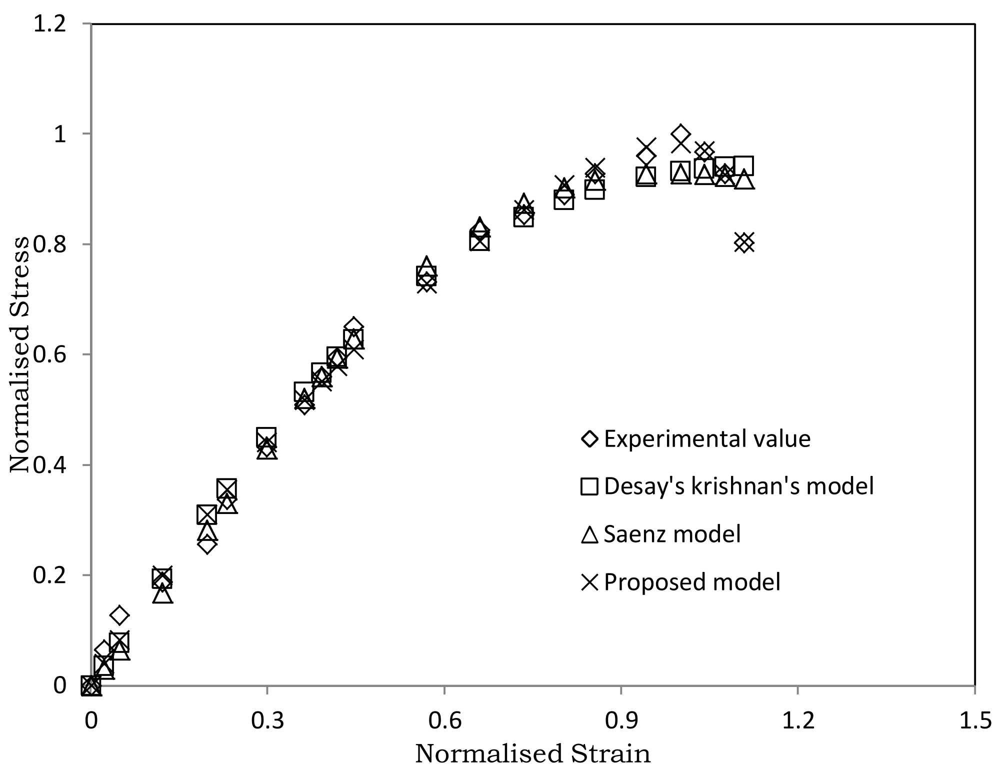

Fig. 7.24 Comparison of stress – strain behavior of M25 CAGRHD concrete

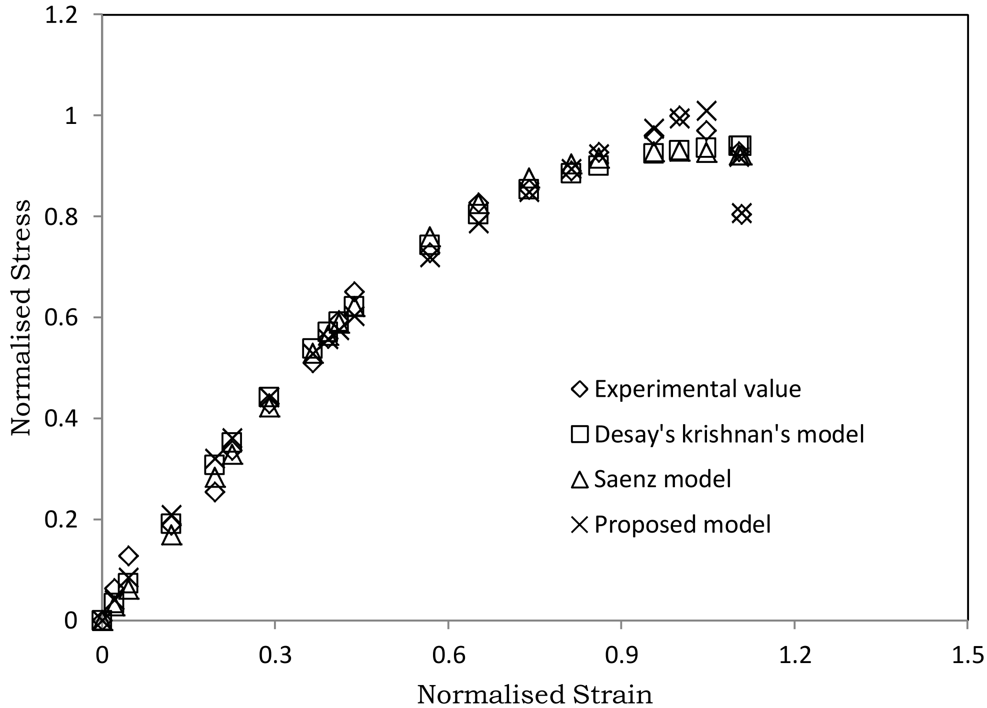

Fig. 7.25 Comparison of stress – strain behavior of M25 CAGRPO concrete

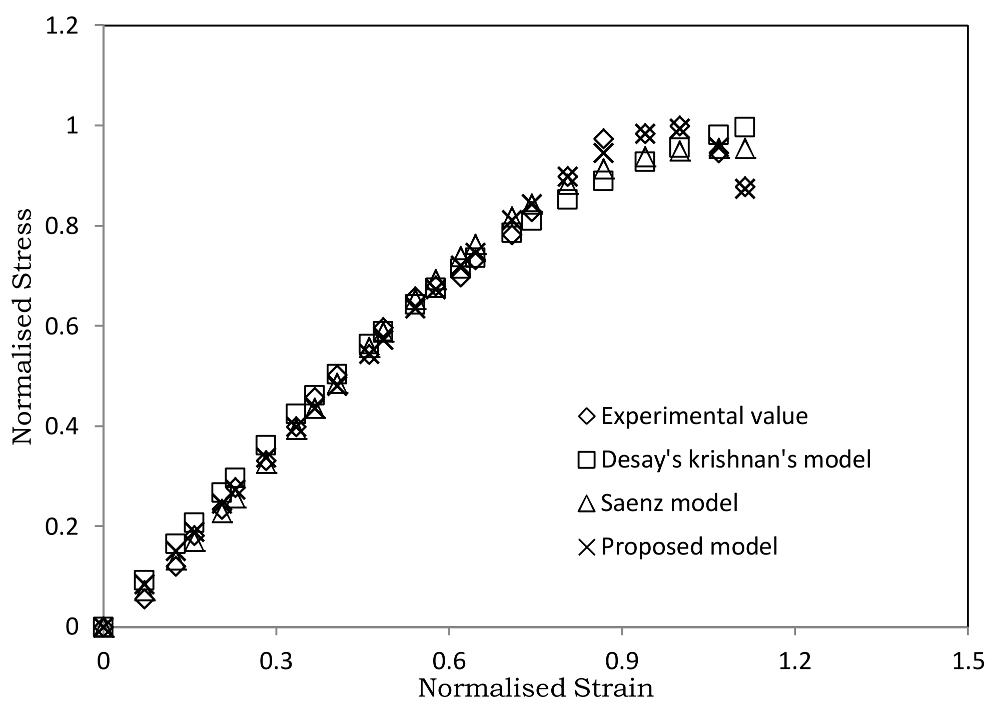

Fig. 7.26 Comparison of stress – strain behavior of M30 CAGRHE concrete

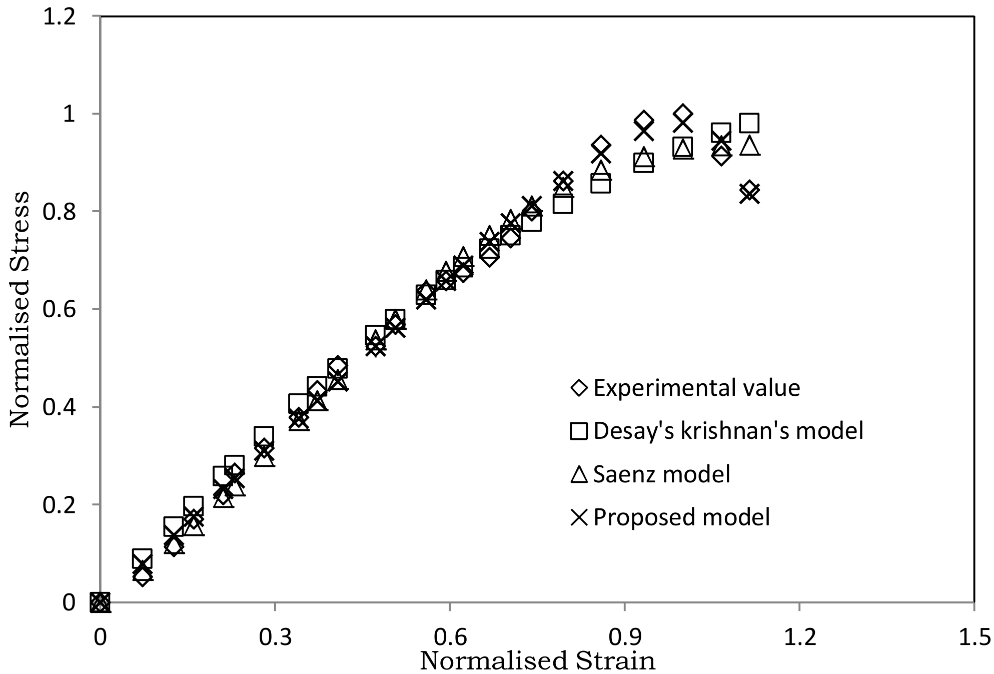

Fig. 7.27 Comparison of stress – strain behavior of M30 CAGRPE concrete

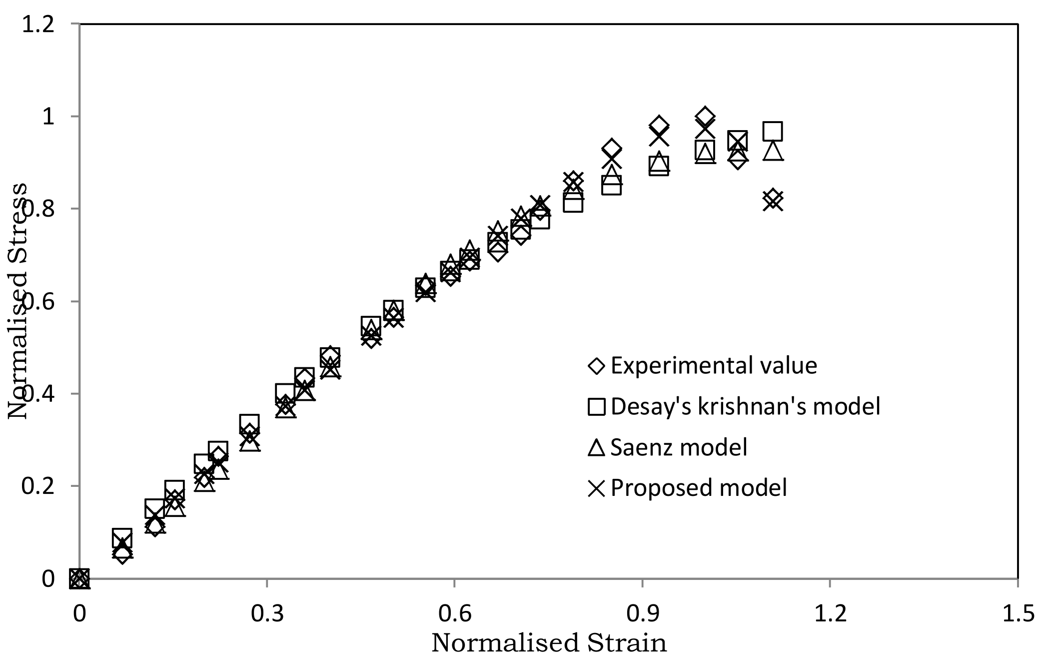

Fig. 7.28 Comparison of stress – strain behavior of M30 CAGRHD concrete

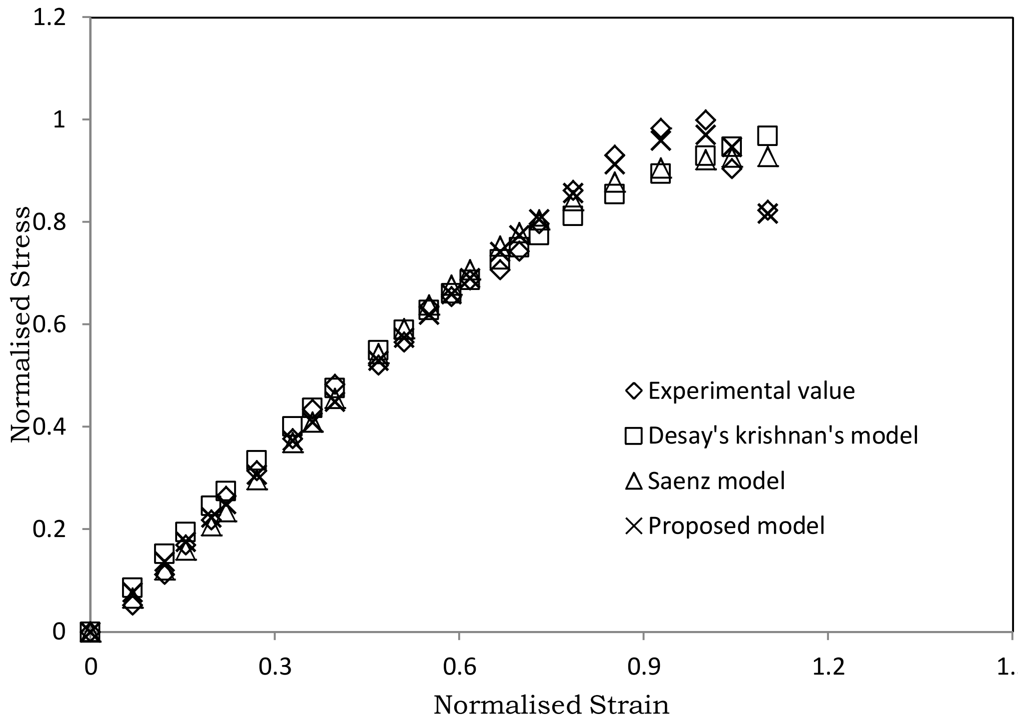

Fig. 7.29 Comparison of stress – strain behavior of M30 CAGRPO concrete

Comparison of existing stress strain models i.e. Desay's Krishnan's models and Saenz models to the proposed model and experimental values given in the figures 7.15 to 7.28. There is a good agreement between the proposed and existing models.

Table: 7.10 R^2 and Ra^2 values of different mixes with different models and proposed model

Material Combinations	Desay's krishnan's		Saenz model		Proposed model	
	R^2	**Ra^2**	**R^2**	**Ra^2**	**R^2**	**Ra^2**
M25 Design mix concrete						
CA	0.9667	0.9647	0.9699	0.9662	0.9952	0.9942
CAG	0.9823	0.9812	0.9823	0.9801	0.9965	0.9958
CAGR	0.9819	0.9809	0.9827	0.9808	0.9958	0.995
CAGRHE	0.983	0.982	0.9851	0.9834	0.9957	0.995
CAGRPE	0.9853	0.9845	0.9867	0.9853	0.9961	0.9954
CAGRHD	0.9833	0.9824	0.9861	0.9846	0.9954	0.9947

CAGRPO	0.9833	0.9824	0.9855	0.9839	0.9932	0.992
M30 Design mix concrete						
CA	0.9761	0.9748	0.9796	0.9773	0.992	0.9906
CAG	0.9863	0.9856	0.9922	0.9914	0.9968	0.9964
CAGR	0.9811	0.9803	0.9893	0.9883	0.9974	0.997
CAGRHE	0.9832	0.9824	0.9908	0.9899	0.9969	0.9964
CAGRPE	0.9772	0.9761	0.9862	0.9848	0.9964	0.9958
CAGRHD	0.9761	0.9751	0.9834	0.9818	0.9955	0.9949
CAGRPO	0.9764	0.9753	0.9837	0.9821	0.9956	0.9949

Table 7.10 shows the values of Coefficient of Multiple Determination (R^2) and Adjusted Coefficient of Multiple Determination (Ra^2) of different models and proposed model. From this it may be concluded that the proposed model is suitable for this design concrete mixes. The suitability of the regression model was decided based on the value of R^2. As the value of R^2 approaches 1 it is the best suitable regression model and if it moves towards 0 then the value of the residuals increase. With increase in the number of variables, the value of residuals decrease and the coefficient of determination R^2 increases. To achieve a more precise comparison Ra^2 is used, which is adjusted for the degrees of freedom. Ra^2 is used for comparing the range of predicted values at the design points to the average prediction error. It is like signal to noise ratio used in Taguchi method. All the mixes had R^2 and Ra^2 values greater than 99%. In each of the cases, the predicted R^2 value was in reasonable agreement with the Ra^2 value. The value of R^2 was closer to 1, which was desirable.

7.4 SUMMARY

1. Based on the stress-strain curves of all design mixes it is observed that the stress-strain pattern is to be almost similar. The only difference is that compared to that of other mixes, the GGBS – ROBO sand and fiber mixes have shown improved stress values. It is observed that for higher grade of concrete with increase in stress there was decrease in strain.
2. Empirical equations for the stress-strain response of all the mixes have been proposed in the form of $y = ((Ax + Bx^2)/(1 + Cx + Dx^2))$ where 'x' is normalized strain and 'y' is normalized stress. The same empirical formula is valid for both ascending and descending portions.
3. The equations for mixes are mentioned table 7.8. These proposed empirical equations can be used as stress block in analyzing the flexural behavior of sections of structural elements. The proposed equations have shown good correlation with experimental values.
4. It is observed that there is an increase in the peak compressive strength for different mixes made with GGBS, ROBO sand and fibers. The increase is due to reactivity of GGBS.

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CHAPTER 8

DURABILITY STUDIES ON MODIFIED CONCRETE

8.1 INTRODUCTION

Durability of concrete is defined as its ability to resist weathering action, chemical attack, abrasion, or any other process of deterioration. Durable concrete will retain its original form, quality, and serviceability when exposed to its environment. Conventional concrete is not recommended for in groundwater conditions contaminated with sulphate. As their applications often require contact with soil, sulphate and hydrochloric attack would represent a significant durability problem. These chemicals can be found in a variety of situations, such as in the manufacture of textiles, food-processing factories, oil refineries, sewage pipes, and fertilizer factories.

The objectives of the work described in this chapter were to compare the relative performance of modified concrete to that of conventional designed concrete exposed to sulfuric and hydrochloric acid solutions.

Design mix concrete cubes of 150 x 150 x 150 mm size are cast with all design mixes of concrete and are tested for compressive strength at the age of 28 days are taken for durability studies. The durability studies are done on cubes immersed in Sulfuric Acid (H_2SO_4), Hydrochloric acids (HCl) and Sodium Sulphate (Na_2SO_4) for 30days, 45days and 60days. The obtained results are presented in tables and the test results are also presented in the form of graphs.

8.2 ACID ATTACK

After 28 days of curing, each cube is tested for weight and dimensions. The cubes are subjected to 5% solutions of H_2SO_4, HCl and Na_2SO_4 individually. Cubes are continuously immersed in solution for 30 days, 45 days and 60 days.

The specimens are arranged in the plastic trays in such a way that the clearance around and above the specimen is not less than 30mm.The response of the specimens to the solution is evaluated through change in appearance, weight, compressive strength, thickness and solid diagonals. Two specimens from each group are used for testing after 30days, 45days and 60days of immersion. Before testing, each specimen is removed from the tray, brushed with a soft nylon brush and rinsed in tap water. This process removes loose surface material from the specimens.

8.2.1 Visual Assessment

There are no standard criteria for evaluating the resistance of concrete exposed to chemical acids. For visual assessment the following scale is used: (Al-temimi and Sonebi 2003)

- 0: no attack
- 1: very slight attack
- 2: slight attack
- 3: moderate attack
- 4: severe attack
- 5: very severe attack
- 6: partial disintegration

Table: 8.1 Visual assessment of concrete deterioration level

Visual assessments	H_2SO_4			HCl			Na_2SO_4		
	30	45	60	30	45	60	30	45	60
M25 Design concrete mix									
CA	3	3-4	4-5	2-3	3-4	3-4	2-3	3	3-4
CAG	3-4	3-4	4-5	2	3	3	2	3	3
CAGR	3-4	3-4	4-5	2	3	3	2	3	3
CAGRHE	2-3	3-4	4-5	2	3	3	2	3	3
CAGRPE	2-3	3-4	4-5	2	3	3	2	3	3
CAGRHD	2-3	3-4	4-5	2	3	3	2	3	3
CAGRPO	2-3	3-4	4-5	2	3	3	2	3	3
M30 Design concrete mix									
CA	2-3	3-4	4-5	2-3	3-4	3-4	2-3	3	3-4
CAG	3-4	3-4	4-5	2	3	3	2	3	3
CAGR	3-4	3-4	4-5	2	3	3	2	3	3
CAGRHE	3	3-4	4-5	2	3	3	2	3	3
CAGRPE	3	3-4	4-5	2	3	3	2	3	3
CAGRHD	3	3-4	4-5	2	3	3	2	3	3
CAGRPO	3	3-4	4-5	2	3	3	2	3	3

The visual assessment of the concrete specimens (cubes) after 30 days, 45 days and 60 days of immersion in sulfuric acid, hydrochloric acid and Sodium Sulphate are summarized in Table 8.1. The concrete specimens showed very severe attack at 60 days in sulfuric acid, moderate attack in HCl and Na_2SO_4. The specimens turned into a white pulpy mass in addition to peeling. These reactions resulted from expansive reactions in the concrete binder (Barker et al. 1999; Hartshorn et al. 1999). In addition, sulfates react with the hydrated calcium-silicate phase present in all Portland cements, thereby forming gypsum (Ca_2SO_4), which reacts with C_3A to form ettringite and monosulphoaluminate (Hartshorn et al. 1999; Older and Colan-Subauste 1999). These reactions result in a substantial expansion and peeling and lead to an increase in mass loss each day after cleaning and removing the deteriorated layers with a steel-wire brush. In hydrochloric acid and Sodium Sulphate solution, the degree of deterioration appeared slightly greater. The chlorides react with the hydrated calcium silicate phase present in all port land cements, thereby forming $CaCl_2$, which reacts with C_3A to form chloroaluminate and ettringite. The concrete specimens are showed moderate attack, but the overall degree of attack tended to be severe in sulfuric solution.

8.2.2 Weight Loss and Compressive Strength Loss

The weight and compressive strength of cubes immersed for 30 days, 45 days and 60 days in acids are noted down. The Weight Loss and Strength loss at 30 days, 45 days and 60 days for all grades of design mixes are calculated. The Weight and Compressive strength loss at 30 days, 45 days and 60 days results of M30 concrete mix are shown in Table 8.2. The figures 8.1 to 8.7 show the graphs of weight loss, compressive strength loss M25 concrete mix.

Table: 8.2 weight loss and compressive strength loss of
M30 design concrete mix immersed in H_2SO_4, HCl and Na_2SO_4

M30 Mix design		**H_2SO_4**			**HCl**			**Na_2SO_4**		
		Days of immersion			Days of immersion			Days of immersion		
		30	**45**	**60**	**30**	**45**	**60**	**30**	**45**	**60**
CA	w before	8.20	8.20	8.20	8.20	8.20	8.20	8.20	8.20	8.20
	w after	7.85	7.57	7.27	7.86	7.59	7.30	7.88	7.62	7.34
	% w loss	4.27	7.68	11.34	4.15	7.44	10.98	3.90	7.07	10.49
	St. before	38.15	38.15	38.15	38.15	38.15	38.15	38.15	38.15	38.15
	St. after	37.12	35.65	34.31	37.22	35.69	34.34	37.26	35.72	34.36
	%loss	2.70	6.55	10.07	2.44	6.45	9.99	2.33	6.37	9.93
CAG	w before	8.17	8.17	8.17	8.17	8.17	8.17	8.17	8.17	8.17
	w after	7.84	7.65	7.33	7.86	7.68	7.35	7.89	7.71	7.39
	% w loss	4.04	6.36	10.28	3.79	6.00	10.04	3.43	5.63	9.55
	St. before	43.21	43.21	43.21	43.21	43.21	43.21	43.21	43.21	43.21
	St. after	42.11	40.09	38.88	42.15	40.12	38.91	42.18	40.16	38.95
	%loss	2.55	7.22	10.02	2.45	7.15	9.95	2.38	7.06	9.86

CAGR	w before	8.21	8.21	8.21	8.21	8.21	8.21	8.21	8.21	8.21
	w after	7.89	7.65	7.39	7.91	7.66	7.41	7.91	7.68	7.45
	% w loss	3.90	6.82	9.99	3.65	6.70	9.74	3.65	6.46	9.26
	St. before	45.71	45.71	45.71	45.71	45.71	45.71	45.71	45.71	45.71
	St. after	44.58	42.64	41.15	44.60	42.67	41.18	44.62	42.70	41.21
	%loss	2.47	6.72	9.98	2.43	6.65	9.91	2.38	6.58	9.84
CAGRHE	w before	8.36	8.36	8.36	8.36	8.36	8.36	8.36	8.36	8.36
	w after	8.03	7.80	7.54	8.04	7.82	7.56	8.04	7.83	7.59
	% w loss	3.95	6.70	9.81	3.83	6.46	9.57	3.83	6.34	9.21
	St. before	49.75	49.75	49.75	49.75	49.75	49.75	49.75	49.75	49.75
	St. after	48.39	46.59	45.15	48.40	46.61	45.18	48.41	46.65	45.19
	%loss	2.73	6.35	9.25	2.71	6.31	9.19	2.69	6.23	9.17
CAGRPE	w before	8.36	8.36	8.36	8.36	8.36	8.36	8.36	8.36	8.36
	w after	8.01	7.75	7.54	8.02	7.78	7.56	8.03	7.80	7.59
	% w loss	4.19	7.30	9.81	4.07	6.94	9.57	3.95	6.70	9.21
	St. before	49.80	49.80	49.80	49.80	49.80	49.80	49.80	49.80	49.80
	St. after	48.45	46.69	45.26	48.48	46.71	45.29	48.50	46.75	45.32
	%loss	2.71	6.24	9.12	2.65	6.20	9.06	2.61	6.12	9.00
CAGRHD	w before	8.37	8.37	8.37	8.37	8.37	8.37	8.37	8.37	8.37
	w after	8.06	7.72	7.55	8.08	7.73	7.59	8.09	7.75	7.61
	% w loss	3.70	7.77	9.80	3.46	7.65	9.32	3.35	7.41	9.08
	St. before	49.60	49.60	49.60	49.60	49.60	49.60	49.60	49.60	49.60
	St. after	48.44	46.51	45.01	48.46	46.53	45.04	48.47	46.55	45.06
	%loss	2.34	6.23	9.25	2.30	6.19	9.19	2.28	6.15	9.15
CAGRPO	w before	8.38	8.38	8.38	8.38	8.38	8.38	8.38	8.38	8.38
	w after	8.07	7.72	7.56	8.08	7.74	7.59	8.09	7.76	7.60
	% w loss	3.70	7.88	9.79	3.58	7.64	9.43	3.46	7.40	9.31
	St. before	49.50	49.50	49.50	49.50	49.50	49.50	49.50	49.50	49.50
	St. after	48.33	46.41	44.91	48.36	46.42	44.93	48.34	46.43	44.94
	%loss	2.36	6.24	9.27	2.30	6.22	9.23	2.34	6.20	9.21

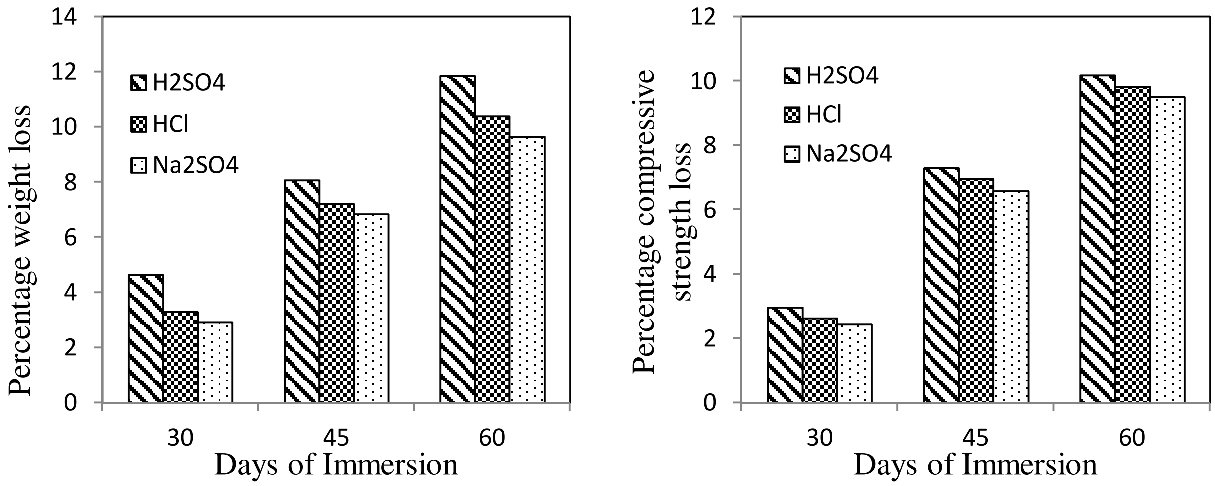

Fig. 8.1 weight loss and compressive strength loss of M25 CA concrete

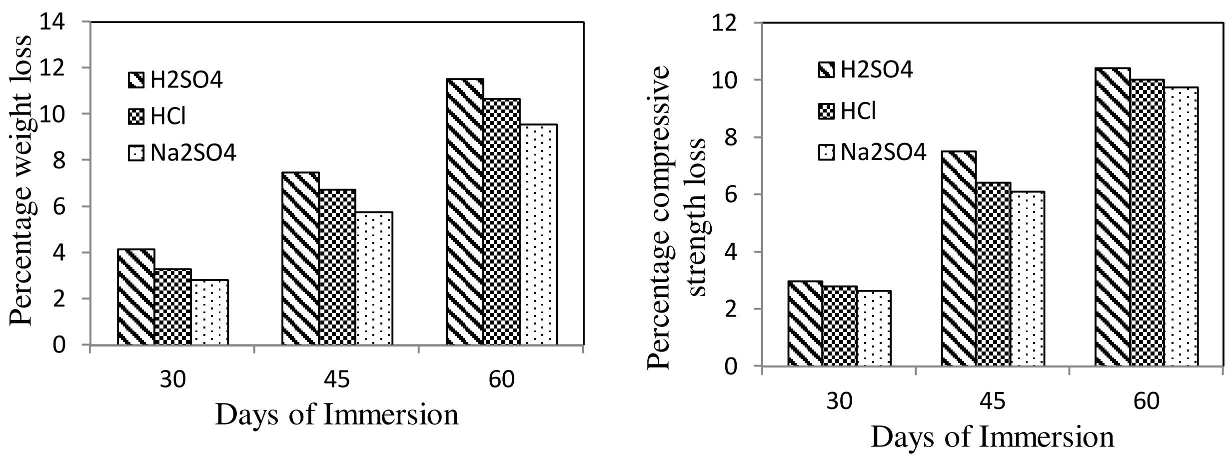

Fig. 8.2 weight loss and compressive strength loss of M25 CAG concrete

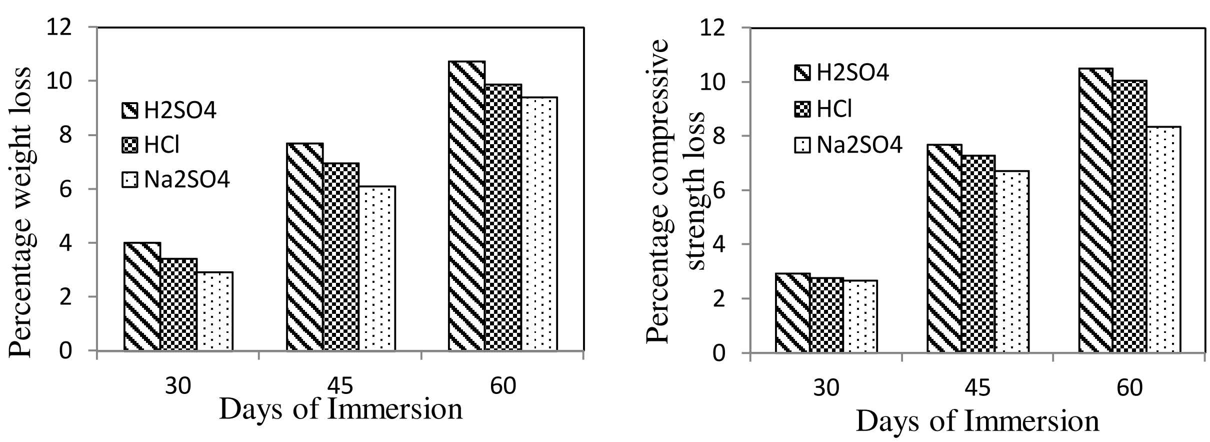

Fig. 8.3 weight loss and compressive strength loss of M25 CAGR concrete

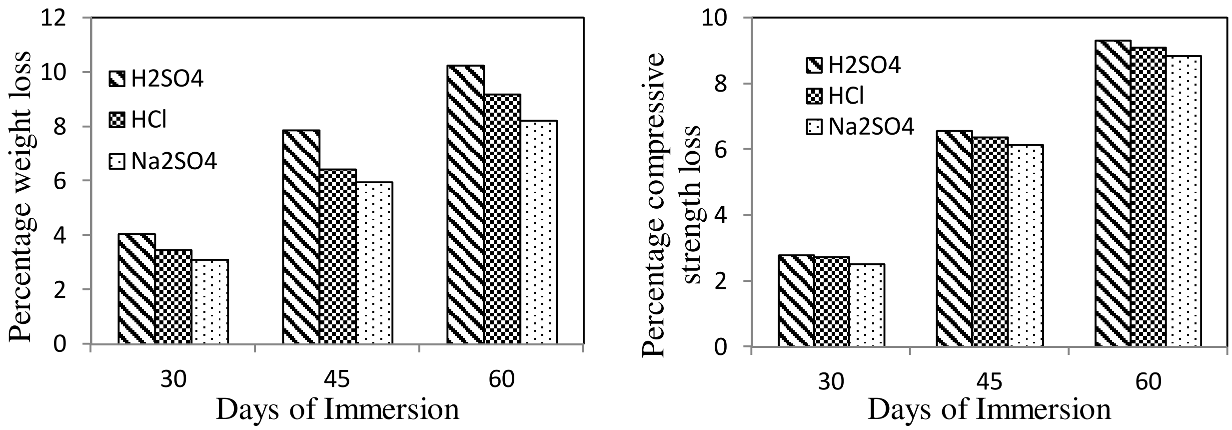

Fig. 8.4 weight loss and compressive strength loss of M25 CAGRHE concrete

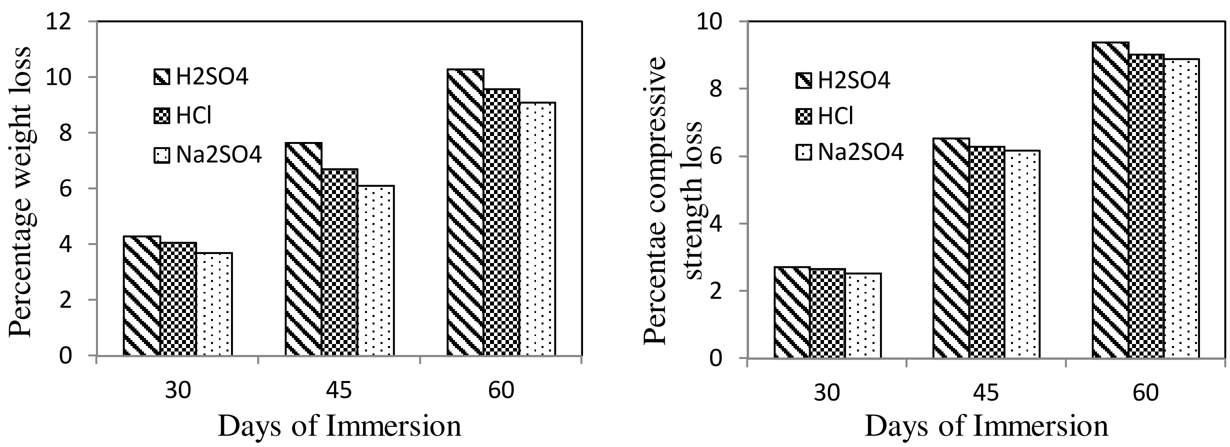

Fig. 8.5 weight loss and compressive strength loss of M25 CAGRPE concrete

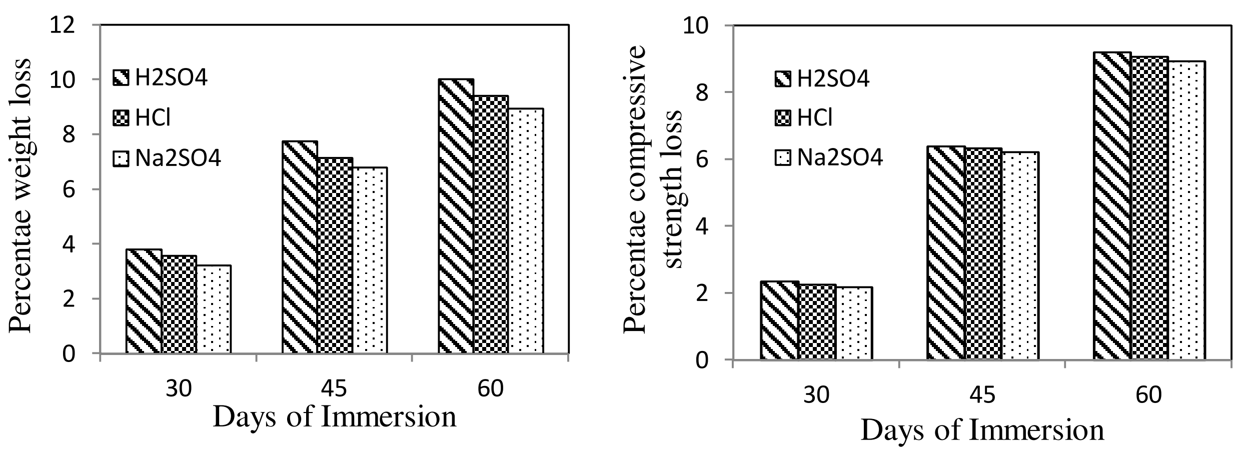

Fig. 8.6 weight loss and compressive strength loss of M25 CAGRHD concrete

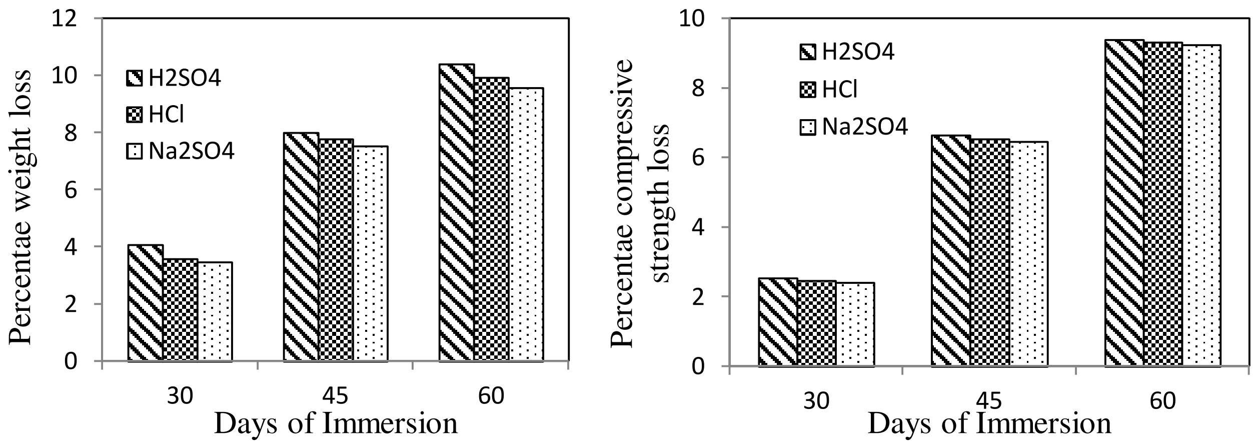

Fig. 8.7 weight loss and compressive strength loss of M25 CAGRPO concrete

8.2.3 Acid Durability Factor and Acid Attack Factor

For determining the resistance of concrete specimens to aggressive environment such as acid attack, the durability factors as proposed in the ASTM 666-1997. The standard test method for resistance of concrete to rapid freezing and thawing and the durability factors are defined in terms of relative dynamic modulus of elasticity. In the present investigation, the “Acid Durability Factors” are derived directly in terms of relative strengths. The relative strengths are always compared with respect to the 28 days value.

The “Acid Durability Factors” (ADF) (Sunil Pratap Reddy et. al. 2010) can be calculated as follows

$$ADF = (Sr \times N) / M$$

Where, Sr - Relative Strength at N days, (%) N - Number of days at which the durability factor is needed and M - Number of days at which the exposure is to be terminated.

The extent of deterioration at each corner of the struck face and the opposite face is measured in terms of the acid diagonals (fig. 8.8) for each of two cubes and the “Acid Attack Factor” (AAF) per face is calculated as follows (Sunil Pratap Reddy et. al. 2010).

AAF = (Loss in mm on eight corners of each of 2 cubes)/4

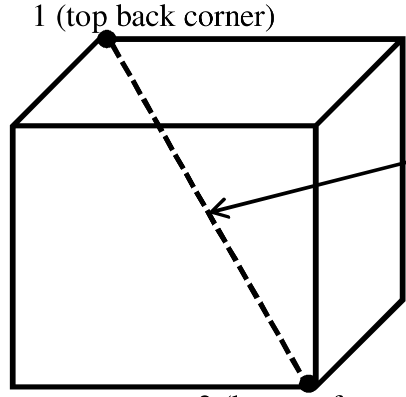

Fig. 8.8 Measurement of Solid diagonal

The results of the tested Acid Resistance behavior at 30 days, 45 days and 60 days for M25 design concrete mixes are shown in Table 8.3 and figures 8.9 to 8.15 shows the ADF and AAF for M30 design concrete mixes.

Table: 8.3 ADF and AAF of M25 design concrete mix immersed in H_2SO_4, HCl and Na_2SO_4

M25 Mix design		**H2SO4**			**HCl**			**Na2SO4**		
		Days of Immersion			**Days of Immersion**			**Days of Immersion**		
		30	**45**	**60**	**30**	**45**	**60**	**30**	**45**	**60**
0%ggbs	**Sr**	97.04	92.69	89.81	97.36	93.05	90.17	97.56	93.43	90.49
	ADF	48.52	69.52	89.81	48.68	69.78	90.17	48.78	70.07	90.49
	AAF	0.41	0.72	1.16	0.26	0.56	0.70	0.25	0.55	0.69
50%ggbs	**Sr**	97.02	92.48	89.57	97.19	93.57	89.98	97.36	93.88	90.25
	ADF	48.51	69.36	89.57	48.59	70.18	89.98	48.68	70.41	90.25
	AAF	0.39	0.71	1.15	0.25	0.55	0.68	0.25	0.54	0.66
50%ggbs + 25%robo sand	**Sr**	97.06	92.31	89.49	97.22	92.70	89.94	97.31	93.27	91.64
	ADF	48.53	69.23	89.49	48.61	69.52	89.94	48.66	69.96	91.64
	AAF	0.39	0.70	1.17	0.26	0.54	0.69	0.25	0.53	0.68
50%ggbs + 25%robo sand + HDPE fibers	**Sr**	97.20	93.43	90.68	97.26	93.62	90.91	97.47	93.85	91.16
	ADF	48.60	70.07	90.68	48.63	70.22	90.91	48.74	70.39	91.16
	AAF	0.36	0.68	1.16	0.27	0.57	0.68	0.25	0.55	0.65
50%ggbs + 25%robo sand +	**Sr**	97.27	93.47	90.61	97.33	93.70	90.96	97.46	93.82	91.11
	ADF	48.63	70.10	90.61	48.67	70.27	90.96	48.73	70.37	91.11

PET fibers	**AAF**	0.35	0.67	1.18	0.27	0.56	0.67	0.26	0.54	0.65
50%ggbs + 25%robo sand + HDPP fibers	**Sr**	97.63	93.60	90.78	97.74	93.66	90.91	97.82	93.76	91.06
	ADF	48.82	70.20	90.78	48.87	70.24	90.91	48.91	70.32	91.06
	AAF	0.36	0.68	1.16	0.28	0.55	0.69	0.27	0.54	0.68
50%ggbs + 25%robo sand + POLYESTER fibers	**Sr**	97.46	93.35	90.60	97.52	93.46	90.69	97.59	93.54	90.75
	ADF	48.73	70.02	90.60	48.76	70.10	90.69	48.79	70.16	90.75
	AAF	0.37	0.67	1.17	0.27	0.56	0.68	0.25	0.55	0.66

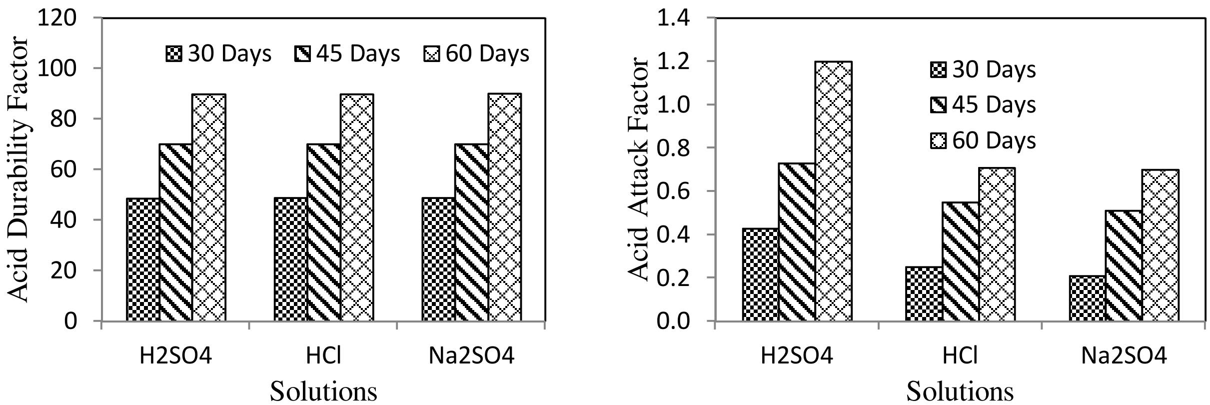

Fig. 8.9 ADF and AAF of M30 CA concrete

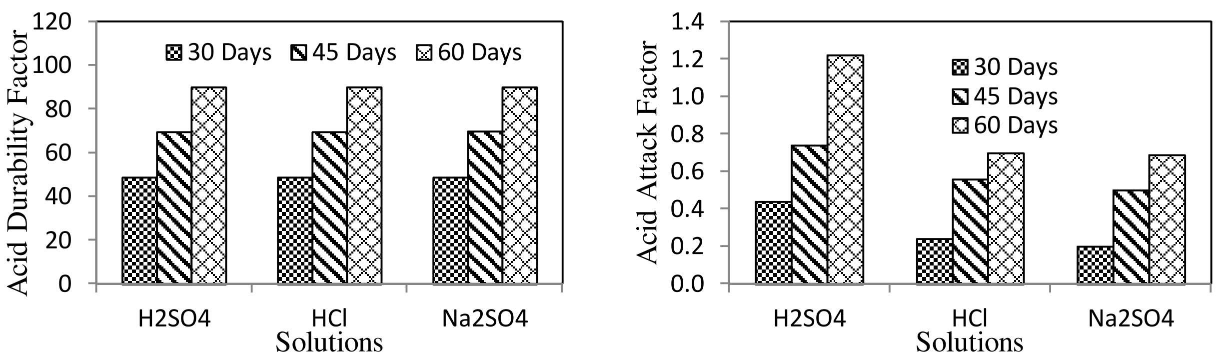

Fig. 8.10 ADF and AAF of M30 CAG concrete

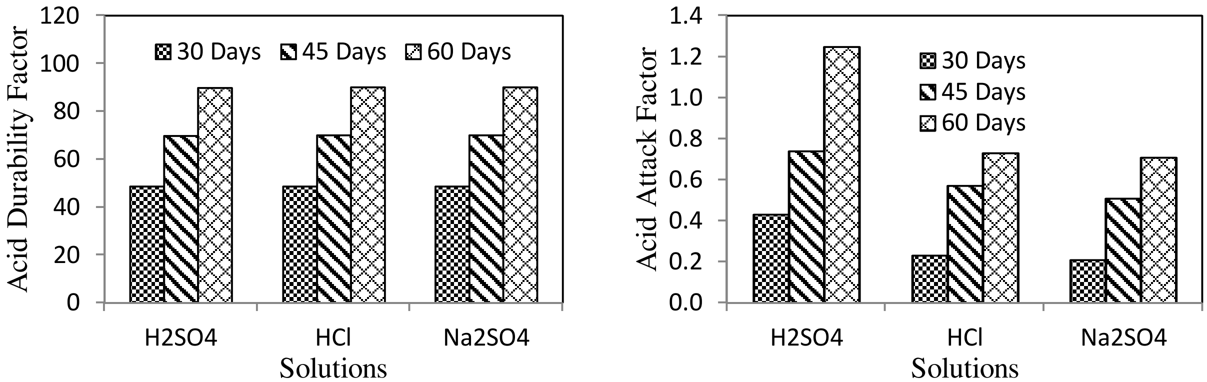

Fig. 8.11 ADF and AAF of M30 CAGR concrete

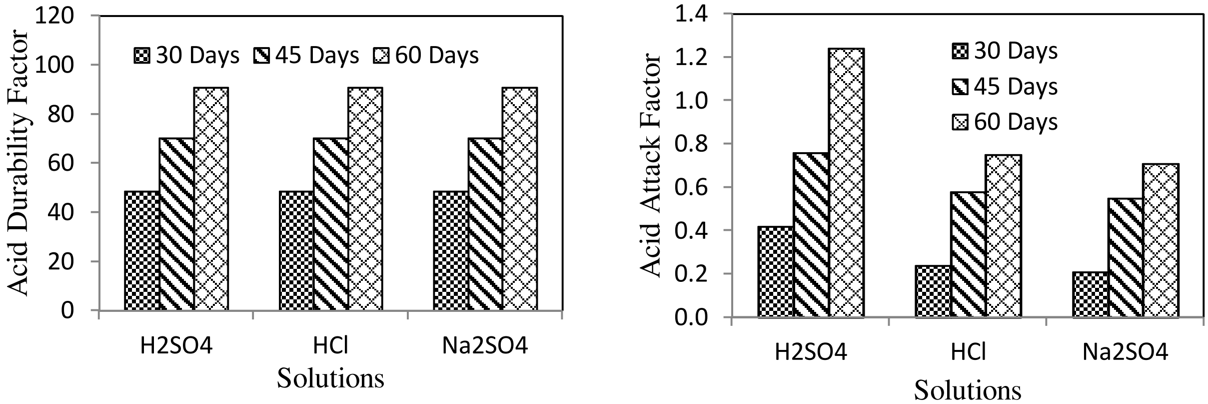

Fig. 8.12 ADF and AAF of M30 CAGRHE concrete

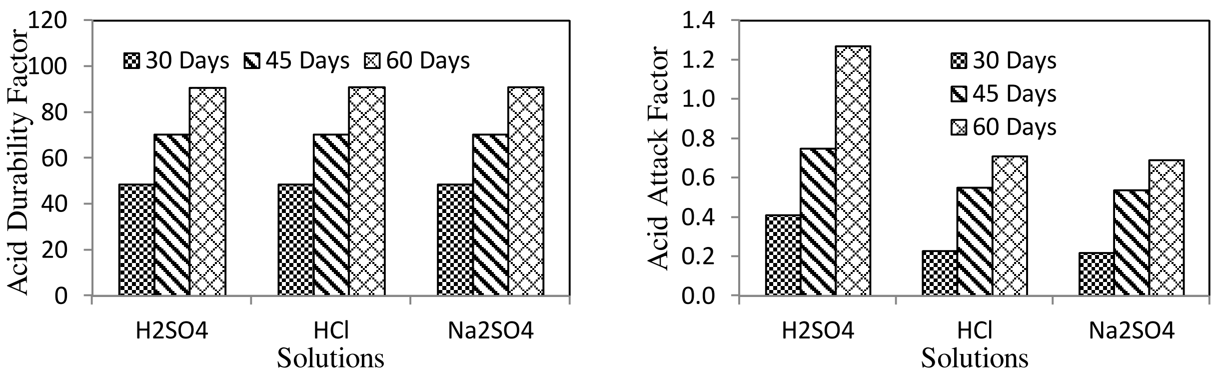

Fig. 8.13 ADF and AAF of M30 CAGRPE concrete

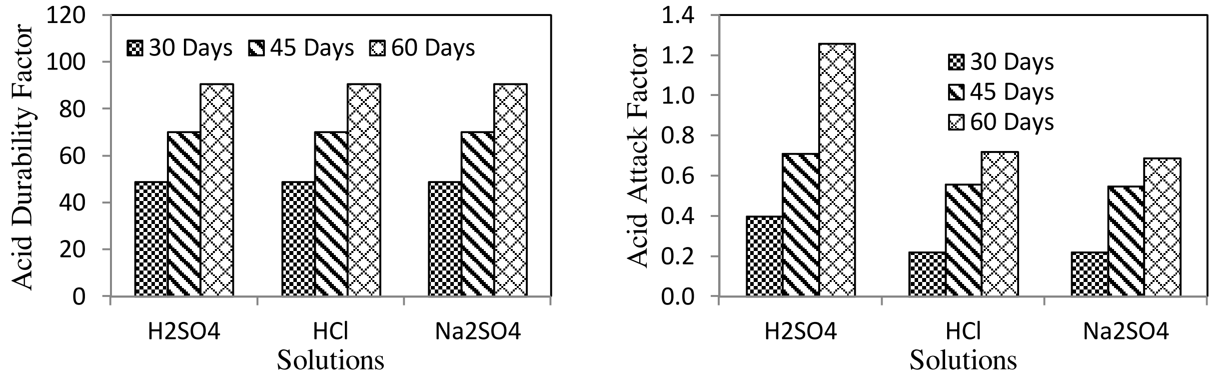

Fig. 8.14 ADF and AAF of M30 CAGRHD concrete

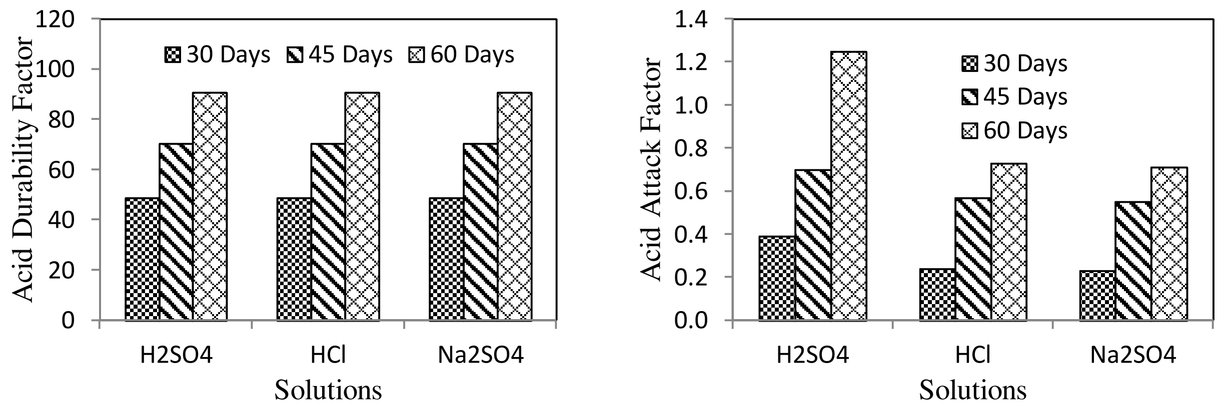

Fig. 8.15 ADF and AAF of M30 CAGRPO concrete

From the figures 8.1 to 8.7 and Table 8.2 shows variation in percentage weight loss and compressive strength loss with different percentages of GGBS, ROBO Sand and fibers for M25 and M30 grades. It is observed that the percentage of weight loss is more for the cubes immersed in 5% H_2SO_4, than the design concrete mix.

8.2.4 Chloride and Sulphate attack: When the concrete is exposed to sulfuric acid solution, hydrogen ions and sulfate ions deteriorate the concrete properties by reacting with hydration products to make cement matrix more porous and/or expansive (Joong-Kyu Jeon 2006).

Concrete is chemically stable in an alkaline environment, but unstable in neutral or acidic environment. Hydrogen ions in sulfuric acid usually react with calcium ions in cement matrix then to decompose the hydration products as seen in the following chemical equations.

$Ca\ (OH)_2 + 2H^+ \longrightarrow Ca^{2+} + 2H_2O$

CSH gel: $3CaO \cdot 3H_2O + 6H^+ \longrightarrow 3Ca^{2+} + 2SiO_2 + 6H_2O$

Ettringite: $3CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 32H_2O + 6H^+ \longrightarrow 3Ca^{2+} + Al_2O_3 \cdot 3H_2O + 3CaSO_4 + 32H_2O$

Sulfate ions penetrated into concrete in general react with calcium hydroxide of cement matric to form gypsum, which softens the inner concrete structure and decreases the concrete properties. The gypsum in cement softens concrete and decomposes hydration products and thus the weight and strength of the concrete specimens was reduced. It was observed in the present study that the degree of deterioration was dependent on binder and it is likely attributed to different hydration products and the rate of hydration. The higher the calcium hydroxide may imply the higher probability of formation of gypsum in the cement matrix. In case of GGBS specimens was less damaged probably due to its latent hydraulic characteristics. Calcium oxide (CaO) and silicon di oxide (SiO_2) in GGBS reacts with water to form CSH hydrate in an alkaline environment and alumina (Al_2O_3) in GGBS also latently forms CAH hydrate. This hydration process consumes calcium hydroxide, there by less probability of reacting between sulfuric acid. Hence 50% GGBS concrete in the present study was the most beneficent in delaying the acid corrosion of specimen. The hydrates generated from GGBS reduced the porosity of concrete, which allowed sulfuric acid to penetrate the specimens at a lower rate.

Chloride reacts with the hydrates of cement and form Freidel's salt that does not have any harmful effects on concrete, but when chloride content in concrete reaches more than the threshold value, the protective alkaline layer of steel reinforcement is broken and in the presence of oxygen and humidity, steel reinforcement gets corroded (Prasad 2006).

Freidel's salt: $3CaO \cdot Al_2O_3 \cdot 3CaCl_2 \cdot 10H_2O + 6H^+ \longrightarrow 3Ca^{2+} + Al_2O_3 \cdot 3H_2O + CaCl_2 + 6H_2O$

From the studies of compressive strengths of all grades of concrete, before and after immersion in acids. It is observed that when immersed in Na_2SO_4 the all grades of concrete are showing lesser compressive strength loss than immersed in H_2SO_4 and HCl. The percentage compressive strength loss of M25 mix, when immersed in 5% H_2SO_4 is 9.22 to 10.51. For M30 the percentage compressive strength loss values are 9.12 to 10.07. From the studies of compressive strength, it is observed that the compressive strength loss is less for cubes immersed in Na_2SO_4 than the cubes immersed in $H_2SO_4$and HCl.

Durability studies carried out in the investigation through acid attack test with 5% H_2SO_4, 5% HCl and 5% Na_2SO_4 revealed that GGBS, ROBO Sand with fibers are more durable in terms of "Acid Durability Factors" than reference concrete. The investigation through acid attack test with 5% H_2SO_4 revealed that modified fiber reinforced concrete is 1.08% and 0.91% more durable in terms of "Acid Durability Factors" than the reference M25 and M30 concretes respectively.

The investigation through acid attack test with 5% Na_2SO_4 revealed that modified fiber reinforced concrete is 0.68% and 1.03% more durable in terms of "Acid Durability Factors" than the reference M25 and M30 concretes respectively. Durability studies carried out in the investigation through acid attack test with 5% HCl revealed that modified fiber reinforced concrete is 0.87% and 0.88% more durable in terms of “Acid Durability Factors” than the reference M25 and M30 concretes respectively.

For the Acid Attack Factors similar studies has been carried out through acid attack test with 5% H_2SO_4, 5% HCl, and 5% Na_2SO_4. Table 8.5 and figures 8.9 to 8.15, Shows the “Acid Attack Factors” for the different concrete mixes of M25 and M30.

8.3 SUMMARY

1. The strength and durability can be improved by the addition of a GGBS, ROBO Sand and fibers to design concrete.
2. The Acid durability factors (ADF) were found to be 1.03% more in concrete made with GGBS, ROBO Sand and fibers in all grades.
3. The Acid Attack Factors (AAF) has shown that the GGBS, ROBO Sand and fibers mixes are 1.7% more resistant for acid attack.
4. The strength loss and weight loss observed to be less in mixes with GGBS, ROBO Sand and fibers.

Fig. 8.16 Cube Samples for durability test

Fig. 8.17 Cube Samples immersed in the acidic solutions

Fig. 8.18 Cube Samples after the test

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CHAPTER 9

SUMMARY AND CONCLUSIONS

From the experimental studies, conducted on various combinations of supplementary materials like GGBS for cement and ROBO sand for fine aggregate along with non – bio degradable waste products as fibers in the concrete, the conclusions are summarized as follows:

(i) The average 28 days compressive strength of M25 and M30 concrete by using SNP 3 admixture has increased by 30 and 25.8% respectively compared to concrete without admixture.

(ii) The experimental results show that 50% of cement can be replaced with GGBS and 25% of fine aggregate can be replaced with ROBO sand simultaneously.

(iii) The compressive strength of CAG concrete has increased by 47.34% and 39.7% of M25 and M30 concretes respectively compared with concrete type C.

(iv) Split tensile strength and flexural strength of CAG concrete has increased by 8.10% and 5.91% for M25 concrete while for M30 concrete the increments are found to be 8.68% and 7.39% respectively compared to concrete type CA.

(v) The percentage of increase in the compressive strength is 55.74% and 47.79% at the age of 28 days of both M25 and M30 CAGR concrete compared to type C concrete. The percentage of increase in the split tensile and flexural strengths are 9.03% and 8.75% at the age of 28 days for M25 CAGR concrete and 9.45% and 10.16% for M30 CAGR concrete compared to concrete type CA.

(vi) Compressive strength of CAGRHE, CAGRPE, CAGRHD and CAGRPO M25 concretes are 69.86, 70.15, 69.15 and 68.93 % more than the M25 concrete type C respectively.

Similarly, the percentage increase for M30 CAGRHE, CAGRPE, CAGRHD and CAGRPO concretes are 60.85, 61.01, 60.36 and 60.04 % respectively compared to M30 C concrete.

(vii) Similarly, the percentage increase in the split tensile strengths of CAGRHE, CAGRPE, CAGRHD and CAGRPO M25 concretes are 18.69, 19.63, 18.07 and 17.45 % respectively and the split tensile strengths of M30 concretes of CAGRHE, CAGRPE, CAGRHD and CAGRPO types are 20.06, 20.66, 19.16 and 18.26 % more than the M30 CA concrete.

(viii) The flexural strength 14.89, 15.37, 14.42 and 13.71 % are more of CAGRHE, CAGRPE, CAGRHD and CAGRPO M25 concretes respectively and the increase in the flexural strengths of CAGRHE, CAGRPE, CAGRHD and CAGRPO M30 concretes are 16.63, 17.09, 15.7 and 15.01 % respectively compared with the concrete type CA concrete.

(ix) Empirical equations for the stress-strain response of all the mixes have been proposed in the form of $y = ((Ax + Bx^2)/(1 + Cx + Dx^2))$ where 'x' is normalized strain and 'y' is normalized stress.

(x) The stress – strain equations for all mixes i.e. CA, CAG, CAGR, CAGRHE, CAGRPE, CAGRHD and CAGRPO of both M25 and M30 concrete are given in table 7.8. The proposed equations have shown good correlation with experimental values.

(xi) It is observed that there is an increase in the peak stress about 20% and strain at peak stress approximately 50% for different mixes i.e. CAGRHE, CAGRPE, CAGRHD and CAGRPO compared to CA concrete.

(xii) The strength and durability can be improved with the addition of GGBS, ROBO Sand and fibers to the concrete.

(xiii) The Acid durability factors (ADF) were found to be 1.08% more in concrete made with GGBS, ROBO Sand and fibers in all grades. It shows that durability can be improved by using supplementary materials and fibers.

(xiv) The Acid Attack Factors (AAF) has shown that the GGBS, ROBO Sand and fibers mixes are 1.7% more resistant for acid attack. The durability of concrete having supplementary materials and fibers is better than the normal concrete.

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CONTRIBUTIONS FROM THE STUDY

1. Locally available industrial waste materials are used in the development of cement concrete mix.
2. Cement content in the concrete can be reduced by 50% from the present study. This will help the environment from the greenhouse gases which are released during the production of cement.
3. Also new concrete mix has been developed with non-bio degradable waste plastic materials as fibers in the concrete. This will also help the disposal of non-bio degradable waste plastics.
4. Proposed equations of stress strain can be used directly to know the behavior of modified concrete without conducting experiments.
5. The new developed cement concrete mix has more durable than the normal concrete. It can be used in the acidic environments also.

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SCOPE FOR FURTHER WORK

Based on the experimental studies carried, the following areas have been identified for future research.

(i) The current research work has concentrated primarily on the strength properties of concrete and can be extended to study other properties like creep, shrinkage, fatigue, permeability, at elevated temperatures, chloride penetration and micro structural investigation, etc.

(ii) This study can be extended further to develop green concrete and/or sustainable concrete with different supplementary materials, industrial waste products and polymers with chemicals.

(iii) Also it can be further extended for self-compacting concrete, high strength and high performance concretes with different supplementary materials.

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APPENDIX

Mix Design

A1. M25 Concrete

i. Grade of Concrete: M25
ii. Type of cement: OPC 43 grade
iii. Maximum nominal size of the aggregate: 20mm
iv. Minimum cement content: 320 kg/m3
v. Water cement ratio: 0.45
vi. Workability: 100mm
vii. Specific gravity of cement: 3.15
viii. Specific gravity of Coarse aggregate: 2.65
ix. Fine aggregate: confirming to Zone II
x. Specific gravity of fine aggregate: 2.62

$$Target\ Strength\ f'_{ck} = f_{ck} + 1.65\ s$$

$$where\ f_{ck} - charcteristic\ compressive\ strength\ at\ 28\ days$$

$$s - standard\ deviation$$

Standard deviation for M25 concrete is 4 (IS 456 – 2000 table no. 8)

Target strength = 31.6 N/mm^2

Maximum water content for 20mm aggregate – 186lts

For 100 mm slump = 186 + 186(6/100) = 197 lts

Super plasticizer used in the mix design, water content can be reduced by 25.5%

Water = 197 X 0.745 = 147 lts

Cement = 147/045 = 327 kg.

Volume of concrete = 1m3

Volume of cement $= \frac{327}{3.15} \times \frac{1}{1000} = 0.1038\ m^3$

Volume of water $= \frac{147}{1} \times \frac{1}{1000} = 0.147\ m^3$

$$\text{Volume of admixture } = \frac{3.27}{1.179} \times \frac{1}{1000} = 0.00277\ m^3$$

$$\text{Volume of aggregates } = 1 - (0.1038 + 0.147 + 0.00277) = 0.74643\ m^3$$

From table no.3 (IS 10262) volume of coarse aggregate corresponding to 20 mm aggregate and fine aggregate (zone II) for water cement ration 0.5 is 0.62

For water cement ratio: 0.45 volume of coarse aggregate is 0.63 and volume of fine aggregate is 0.37.

$$\text{Mass of the coarse aggregate} = 0.74643 \times 0.63 \times 2.65 \times 1000 = 1246 \text{ kg}$$

$$\text{Mass of the fine aggregate} = 0.74643 \times 0.37 \times 2.62 \times 1000 = 723 \text{ kg}$$

$$\mathbf{Mix\ design\ M25\ concrete = 327 : 723 : 1246\ \ with\ 147\ lts\ of\ water}$$

A2. M30 Concrete

i. Grade of Concrete: M30
ii. Type of cement: OPC 43 grade
iii. Maximum nominal size of the aggregate: 20mm
iv. Minimum cement content: 320 kg/m3
v. Water cement ratio: 0.42
vi. Workability: 100mm
vii. Specific gravity of cement: 3.15
viii. Specific gravity of Coarse aggregate: 2.65
ix. Fine aggregate: confirming to Zone II
x. Specific gravity of fine aggregate: 2.62

$$Target\ Strength\ f'_{ck} = f_{ck} + 1.65\ s$$

$$where\ f_{ck} - charcteristic\ compressive\ strength\ at\ 28\ days$$

$$s - standard\ deviation$$

Standard deviation for M30 concrete is 5 (IS 456 – 2000 table no. 8)

Target strength = 38.25 N/mm^2

Maximum water content for 20mm aggregate – 186 lts

For 100 mm slump = 186 + 186(6/100) = 197 lts

Super plasticizer used in the mix design, water content can be reduced by 25.5%

Water = 197 X 0.745 = 147 lts

Cement = 147/0.42 = 350 kg.

Volume of concrete = 1m3

Volume of cement $= \frac{350}{3.15} \times \frac{1}{1000} = 0.1111\ m^3$

Volume of water $= \frac{147}{1} \times \frac{1}{1000} = 0.147\ m^3$

Volume of admixture $= \frac{3.5}{1.179} \times \frac{1}{1000} = 0.00296\ m^3$

Volume of aggregates $= 1 - (0.1111 + 0.147 + 0.00296) = 0.73894\ m^3$

From table no.3 (IS 10262) volume of coarse aggregate corresponding to 20 mm aggregate and fine aggregate (zone II) for water cement ration 0.5 is 0.62

For water cement ratio: 0.42 volume of coarse aggregate is 0.636 and volume of fine aggregate is 0.364

Mass of the coarse aggregate $= 0.73894 \times 0.636 \times 2.65 \times 1000 = 1245$ kg

Mass of the fine aggregate $= 0.73894 \times 0.364 \times 2.62 \times 1000 = 704$ kg

Mix design M30 concrete = 350 : 704 : 1245 with 147 lts of water

A3. Strength contribution of fibers

M25 concrete mix per cubic meter

Mass of the cement = 327 kg

Mass of the fine aggregate = 723 kg

Mass of the coarse aggregate = 1246 kg

Mass of the water = 147 kg

Concrete cube volume = $0.15 \times 0.15 \times 0.15 = 3.375 \times 10^{-3} m^3$

Density of cement = 3150 kg/m^3

Density of the fine aggregate = 2620 kg/m^3

Density of the coarse aggregate = 2650 kg/m^3

Density of the water = 1000 kg/m^3

Total volume of cement in one cube of size 150mm = $\dfrac{3.375 \times 10^{-3} \times 327}{3150}$

$= 3.503 \times 10^{-4} m^3$

The maximum percentage of fibers in the concrete is 3.5% of volume of cement

Volume of fibers = $3.503 \times 10^{-4} \times \dfrac{3.5}{100} = 1.22 \times 10^{-5} m^3$

The volume of fibers in a 150mm cube is $1.22 \times 10^{-5}\mathrm{m}^3$.

Assume fiber size is 2.5mm $\times$ 0.1mm

Tensile load carrying capacity of HDPE fiber is 67.79kg or 677.9N.

The increase in the strength is $\dfrac{1.22 \times 10^{-5}}{0.15 \times 0.15 \times 0.15} \times \dfrac{1}{0.1 \times 2.5} \times 677.9 = 9.62\mathrm{MPa}$

Assume only 40% fibers are contributing to the strength development.

Strength due to HDPE fibers = $9.62 \times 0.4 = 3.85$ MPa

M30 concrete mix per cubic meter

Mass of the cement = 350 kg

Mass of the fine aggregate = 704 kg

Mass of the coarse aggregate = 1245 kg

Mass of the water = 147 kg

Concrete cube volume = $0.15 \times 0.15 \times 0.15 = 3.375 \times 10^{-3} m^3$

Density of cement = 3150 kg/m^3

Density of the fine aggregate = 2620 kg/m^3

Density of the coarse aggregate = 2650 kg/m^3

Density of the water = 1000 kg/m^3

Total volume of cement in one cube of size 150mm = $\dfrac{3.375 \times 10^{-3} \times 350}{3150}$

$= 3.75 \times 10^{-4} m^3$

The maximum percentage of fibers in the concrete is 3.5% of volume of cement

Volume of fibers = $3.75 \times 10^{-4} \times \dfrac{3.5}{100} = 1.3125 \times 10^{-5} m^3$

The volume of fibers in a 150mm cube is $1.3125 \times 10^{-5}\mathrm{m}^3$.

Assume fiber size is 2.5mm $\times$ 0.1mm

Tensile load carrying capacity of HDPE fiber is 67.79kg or 677.9N.

The increase in the strength is $\dfrac{1.3125 \times 10^{-5}}{0.15 \times 0.15 \times 0.15} \times \dfrac{1}{0.1 \times 2.5} \times 677.9 = 10.54\mathrm{MPa}$

Assume only 40% fibers are contributing to the strength development.

Strength due to HDPE fibers = $10.54 \times 0.4 = 4.21$ MPa

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PUBLICATIONS FROM THE STUDY

International Journals:

1. **M. Venu** and P. N. Rao, *"Study of Rubber Aggregates in Concrete: An Experimental Investigation"*, International Journal of Civil Engineering and Technology, Vol. 1(1), pp 15-26, 2010.
2. **Venu Malagavelli** and Rao, P.N., "*High Performance Concrete with GGBS and ROBO sand*", International Journal of Engineering Science and Technology, Vol. 2(10), pp 5107-5113, 2010.
3. **Venu Malagavelli** and Rao, P.N., ***"****Effect of non-bio degradable waste in Concrete slabs"*, International Journal of Civil and Structural Engineering, Vol. 1(3), pp 449-457, 2010. ***DOI:10.6088/ijcser.00202010036***.
4. **Venu Malagavelli** and Neelakanteswara Rao Paturu, "*Polyester Fibers in concrete – An experimental Investigation*", Journal of Advanced materials research, Vol. 261-263, pp 125-129, 2011. ***DOI:10.4028/www.scientific.net/AMR.261-263.125***
5. **Venu Malagavelli** and Neelakanteswara Rao Paturu, "*Strength Characteristics of Concrete using Solid Waste – An experimental Investigation*", International Journal of Earth Sciences and Engineering, Vol. 4(6), pp 937-940, 2011.
6. **Venu Malagavelli** and Neelakanteswara Rao Paturu, "*Strength and Workability Characteristics of Concrete By Using Different Super Plasticizers*", International Journal of Materials Engineering, Vol. 2(1), pp 7-11, 2012. ***DOI: 10.5923/j.ijme.20120201.02***

International Conference Proceedings:

1. **Venu, M**., Padmavathi, V. and Rao, P. N. **"***Effects of admixtures in concrete mix design*"**,** Proceedings of International Conference on Advances in Concrete, Structural and Geotechnical Engineering (ACSGE-2009), October 25-27,Birla Institute of Technology & Science, Pilani, Rajasthan, India.
2. **M. Venu**, P.N.Rao and V. Padmavathi, *"High Performance Concrete - Role of Admixtures"*, Proceedings of International Conference on Materials, Mechanics and

Management (IMMM-2010), January 14-16, College of Engineering Trivandrum, Thiruvananthapuram, Kerala.

3. **Venu Malagavelli** and P. N. Rao, *"Behavior of HDPE Fiber Reinforced Concrete"*, Proceedings of International Conference on Current Trends in Technology (NUiCONE – 2010), December 09 – 11, Nirma University, Ahmedabad, Gujarat.
4. **Venu Malagavelli** and Neelakanteswara Rao Paturu, "*Durability and Strength Characteristics of Fiber Reinforced Concrete*", Proceedings of 11th International Conference on Concrete Engineering and Technology 2012 (CONCET 2012), June 12 – 13, University of Malaya, Malaysia, pp. 188 – 193 (ISBN 978-967-0380-12–4).

Papers Communicated:

1. "Effect of strength properties of concrete by using supplementary materials as partial replacement of cement and fine aggregate", International journal of sustainable cement based materials, ELSEVIER.
2. "Stress - strain behavior of concrete made with industrial waste", International Journal of concrete structures and materials, SPRINGER
3. "Durability characteristics of concrete made with GGBS, ROBO sand and plastic fibers", International journal of Sustainable and Built Environment, ELSEVIER.

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Biography of the Candidate

Malagavelli Venu at present is a Lecturer in the Department of Civil Engineering at Birla Institute of Technology and Science (BITS), Pilani, Hyderabad campus, Andhra Pradesh, India. He has obtained his Bachelor's Degree in Civil Engineering in 2002 from JNTU college of Engineering, Hyderabad, Andhra Pradesh. In 2004, he obtained his Master's degree in Civil Engineering with specialization in Structural Engineering from BITS – Pilani, Rajasthan, India. He has been actively involved in teaching and research work. He has published total ten papers in various journals and conferences based on the present work. He is a life member of Institution of Engineers (India), Indian Concrete Institute, Indian Society for Technical Education and Indian Institute of Bridge Engineers.

Biography of the Supervisor

Dr. P. N. Rao is working as a Professor, in the Department of Civil Engineering at Birla Institute of Technology and Science (BITS), Pilani, Hyderabad campus, Andhra Pradesh, India. He completed Ph.D. in Civil Engineering from the Birla Institute of Technology and Science (BITS), Pilani. He graduated in Civil Engineering from Thapar University, Punjab and Master's degree from BITS – Pilani, Rajasthan, India. He is involved in teaching and research for more than 25 years. His research interests include low cost building materials, High Strength Concrete, High Performance Concrete, Finite Element Analysis, Sustainable Concrete etc. He has published more than twenty papers in peer reviewed international and national journals and conferences.

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