Application of Coir Fibres as Concrete Composites for Disaster prone Structures

R&D Project Report

Submitted by

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Submitted to

CENTRAL INSTITUTE OF COIR TECHNOLOGY

COIR BOARD
Peenya Industrial Area,
Bangalore – 560 058

MARCH 2010

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Abstract

Investigations to overcome the brittle response and limiting post – yield energy absorption of concrete led to the development of fibre reinforced concrete using discrete fibres within the concrete mass. A wide variety of fibres have been proposed by the researchers such as steel, glass, polypropylene, carbon, polyester, acrylic and aramid etc., Over half of the population around the world is living in slums and villages. The earthquake damages in rural areas get multiplied mainly due to the widely adopted non – engineered constructions. On the other hand, in many smaller towns and villages in southern part of India, materials such as nylon, plastic, tyre, coir, sugarcane bagasse and rice husk are available as waste. So, here an attempt has been made to investigate the possibility of reusing these locally available rural waste fibrous materials as concrete composites.

Since the materials used are locally available rural fibres, a detailed characterization is planned. A concrete mix has been designed to achieve the minimum grade of M_{20} as required by IS 456 – 2000. The investigation contains four phases. In the first phase, to identify the effects on workability and mechanical strength properties due to the addition of these rural fibres, workability tests such slump, vee – bee, air content tests and the mechanical strength tests on standard specimens such as compressive strength, split tensile strength, modulus of rupture, modulus of elasticity and shear strength were conducted on the different fibrous concrete specimens to obtain the optimum volume fraction and length of fibres.

Totally 174 cubes, 348 cylinders, 174 flexure specimens and 174 shear specimens were cast and tested. Based on the experimental results of workability and mechanical strength studies, a constant length of 50 mm and two volume fractions such as 0.5% and 1% are chosen for further studies. By analysing the results, empirical relations also have been proposed for mechanical strength properties and compared with the experimental results. From these results, it is concluded that even though addition of fibres reduces the workability of fresh concrete, marginal improvements in the mechanical strength properties are observed which ranges from 10% to 20%.

To asses the performance of different fibrous concrete beams under static loading condition, in the second phase of the investigation, 15 numbers of Reinforced Concrete Beam specimens were cast and tested. Load – deflection and Moment – rotation characteristics are studied. In addition to that, the critical seismic parameters such as ductility, energy absorption capacity and stiffness degradation are also evaluated and discussed. The experimental results are validated by conducting a finite element analysis using ANSYS 10. Concrete is modeled using 'SOLID 65' element and the reinforcements including fibres are modeled using 'LINK 8' element. ANSYS model predicts the experimental behaviour of beams very closely.

The third phase of the investigation aims to estimate the performance of fibrous concrete under seismic loading. For that, another set of 15 beams were cast and tested under simulated cyclic loading conditions. The middle third flexural load was applied on beams at an interval of 2.5 kN. Load – deflection hysteresis loops were drawn to obtain the cumulative values of ductility and energy absorption capacity. Damage indices were also obtained for each fibrous concrete beam specimen by analysing the experimental results, and compared with conventional concrete properties. To examine the effect of rural fibres in the most critical portions of a structure, the beam – column joints, in the fourth phase of this investigation, cyclic load testing was also conducted on three fibrous exterior joint specimens using nylon, plastic and coir fibres at a volume fraction of 1%. The seismic parameters are presented and discussed along with the conventional joint results.

From these elaborative experimental and analytical investigations, it is concluded that out of all the rural fibrous materials used, the contribution of steel binding wire, nylon and coir waste fibres are significant. The remaining are reasonably contribute in the performance enhancement under both static as well as cyclic loading conditions. To ascertain the durability of natural fibres in concrete, an accelerated test was conducted on natural fibres of two forms such as fibres in as received condition and fibres allowed to react with concrete for two years under alternate wetting and drying conditions. These fibre samples were subjected to SEM and EDX analysis, to obtain the changes happened in the microstructure of natural fibres. From these analyses, it is confirmed that the boundary of fibre-matrix transition zone has excellent adhesion. The impregnation of calcium content on the fibre walls showed the better strength enhancement.

Finally it is concluded that fibres recovered from various waste stream are suitable to use as secondary reinforcement in concrete. The advantage of using such rural fibres provides generally a low cost construction than using virgin fibres and the elimination of the need for waste disposal in landfills. Utilization of these fibres in concrete leads to an effective solid waste management technique.

CHAPTER 1

INTRODUCTION

1.1 GENERAL

This chapter deals with the develoment, applications, advantages and limitations of fibre reinforced concrete. It also analyses the necessity of local materials, objectives and scope of the present study with thesis organisation.

1.2 FIBRE REINFORCED CONCRETE

1.2.1 General

Concrete is acknowledged to be a relatively brittle material when subjected to normal stresses and impact loads, where tensile strength is approximately just one tenth of its compressive strength. As a result for these characteristics, concrete flexural members could not support such loads that usually take place during their service life. Historically, concrete member reinforced with continuous reinforcing bars to withstand tensile stresses and compensate for the lack of ductility and strength. Furthermore, steel reinforcement is adopted to overcome high potentially tensile stresses and shear stresses at critical location in concrete member. Even though the addition of steel reinforcement significantly increases the strength of concrete, the development of micro cracks must be controlled to produce concrete with homogenous tensile properties. The introduction of fibres is brought in as a solution to develop concrete with enhanced flexural and tensile strength, which is a new form of binder that could combine Portland cement in bonding with cement matrices. Fibres are most generally discontinuous, randomly distributed throughout the cement matrices. According to terminology adopted by the American Concrete Institute (ACI) Committee 544, in Fibre Reinforced Concrete, there are four catagories namely,

SFRC – Steel Fibre Reinforced Concrete

GFRC – GlassFibre Reinforced Concrete

SNFRC – Synthetic Fibre Reinforced Concrete and

NFRC – Natural Fibre Reinforced Concrete.

1.2.2 Applications of Fibre Reinforced Concrete

The inclusion of fibres in concrete is to delay and control the tensile cracking of composite material. Fibres thus transform an inherent unstable tensile crack propagation to a slow controlled crack growth. This crack controlling property of fibre reinforcement delays the initiation of flexural and shears cracking. It imparts extensive post cracking behaviour and significantly enhances the ductility and the energy absorption capacity of the composite. Earlier fibre-reinforced concrete was used in pavements and industrial floors. But subsequently, Fibre Reinforced Concrete have wide variety of usages in structures such as heavy-duty pavements, Airfields, industrial floor, water retaining and hydraulic structures, parking structure decks, water and waste water treatment plants, pipes, precast roof and wall panels, and the techniques of shotcrete application.

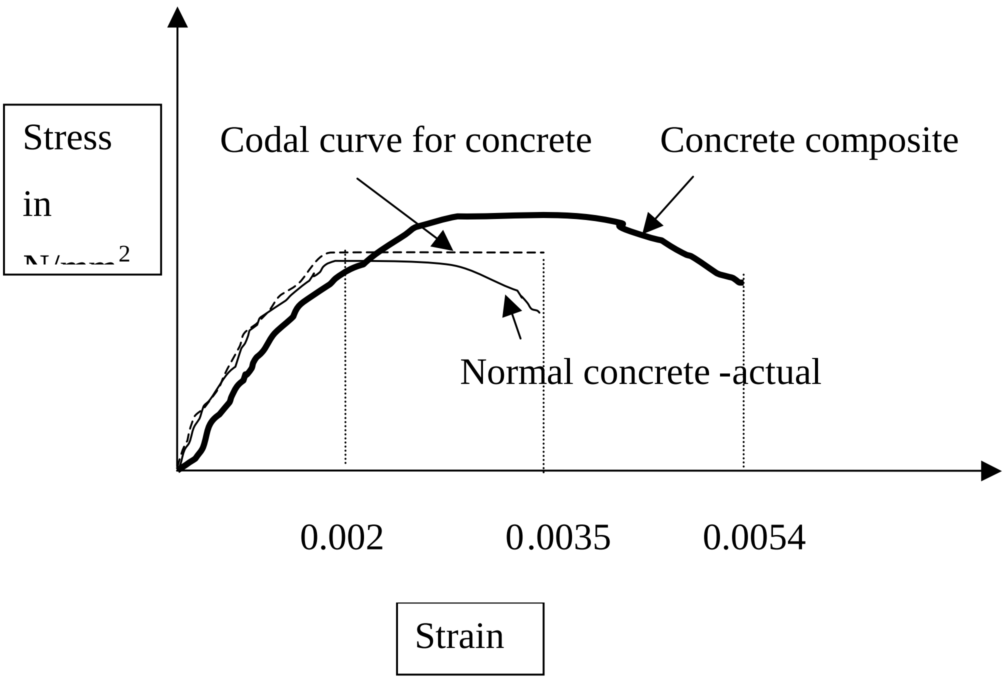

Fig.1 Concept of Ductility Enhancement

Figure 1 indicates the enhancement of ductility in the case of Fibre Reinforced Concrete Composites. Hence the FRC has the potential application in Earthquake resistant structures.

1.2.3 Limitations of Fibre Reinforced Concrete

Fibres, which are randomly distributed throughout the concrete, can overcome cracks and control shrinkage more effectively. These materials have outstanding combinations of strength and energy absorption capacity. In general, the fibre reinforcement is not a substitution to conventional steel reinforcement. The fibres and steel reinforcement have their own role in concrete technology. Therefore, many applications in which both fibres and continuous reinforcing steel bars can be used together. However, fibres are not efficient in withstanding the tensile stresses compared to conventional steel reinforcements. But, fibres are more closely spaced than steel reinforcements, which are better in controlling crack and shrinkage. Consequently, conventional steel reinforcements are used to increase the load bearing capacity of concrete member; fibres are more effective in crack control. The lack of corrosion resistance of normal steel fibres could be a disadvantage in exposed concrete situations. The synthetic fibres are uneconomical to medium level people. Fire resistance of synthetic fibres is also needed to be evaluated.

For example, 1 m^3 of concrete will cost about Rs. 5,000/-. If 1% of steel fibre is put to 1m^3 of concrete, the cost of steel fibres would come around Rs.5,000/-. Hence people living in rural areas that always prefer the non-engineered constructions can not use these fibres. So for medium level constructions, particularly located in medium to high seismic prone areas locally available new contruction materials which would cost are required to be cost effective.

1.3 WASTE FIBROUS RURAL MATERIALS

1.3.1 General

Concrete made with Ordinary Portland Cement has certain characteristics: it is strong in compression and tends to brittle. The weakness in tension can be overcome by the use of primary reinforcement rods and to some extend by the inclusion of a sufficient volume of certain fibres. Moreover the use of fibres alters the behaviour of fibre-matrix composite after concrete has cracked, there by improving its Ductility Since the conventional fibres like steel, polypropylene and glass fibres have some limitations, focus on some other alternative materials which are easy to find in the locality is important.

1.3.2 Local Materials

In India a great amount of Municipal Solid wastes and Agricultural wastes is produced everyday. Reuse of such waste materials in concrete construction is happening nowdays. But they are in the form of Aggregates, Cement (for example fly ash, brick wastes, crusher powder etc,). Similarly only small quantity of work is concentrated on Composites, particularly on natural waste materials. In many smaller towns and villages in the southern parts of India, materials such as Rice Husk, Coir, Nylon Fibre and Sugarcane stems result in the form of fibres and granular materials as waste. Such materials were chosen and properly treated and shaped in the form of fibres or granules and introduced in concrete beams in critical zones for accessing the properties by testing under middle third loading.

1.4 OBJECTIVES AND SCOPE OF THE STUDY

In the recent times, seismic effects have become a major governing factor in analysis, design and construction in India. This is mainly due to the occurrence of medium to severe earthquakes in regions which were not prone to earthquake earlier. Bhuj in the western India and many cities in South India had experienced earthquakes after 1990 and earlier these parts were considered as non-seismic zone. Due to this unexpected attack, losses in lives, property and infrastructural resources are on the rise particularly in residential areas with majority of non-engineered systems.

So, besides improving codal provisions and construction practices, it is necessary to adapt local materials in construction with down-to-earth technology. So far the work on composites is confined to application of aircraft & automobile industries with very little work being imparted in civil infra-structural activities. The application of composites in civil infra-structural activities mostly concentrated on the mechanical strength on composites and not on its usage in structural system. On the other hand solid waste disposal has become one of the major problems in modern cities. At present there are two major methods in practice to dispose wastes. One is land filling and theother is burning. First one requires more valuable land and second one pollutes the environment. So, alternate methods to dispose solid waste should be found. By considering these requirements, here an attempt is made to study the possibilities of reusing the coir fibre materials as fibre composites in concrete which not only tries to solve the ductility problem but also the problem of waste disposal atleast to a small extent.

Hence the focus of the study is to characterize the mechanical, structureal and microstructural properties of local and waste materials as composites in terms of flexibility, ductility & energy absorption to improve seismic resistance. Since the materials chosen in this research work are locally available materials, a detailed characterisation through various testing and analytical mehtods are essential. Linear regresion analyses have been carried out for mechanical strength properties and equations are proposed. The results of flexural testing have been validated by doing the finite element analyses with commercially available software 'ANSYS'.

The current approah to the design of earthquake resistant structures is based on damage prevention during low magnitude earthquakes allowing some damage during moderate and intermediate tremors and on prevention of collapse during severe earthqukes.A numerical damage index has to be defined to facilitate the quantification of damage in analysis and design of engineering systems. Damage indices are potentially valuable design tools, since they provide a means by which different design or retrofi options can be compared objectively. So here damge index based on ductiltiy and energy absorption are also eveluated according to the previous proposed equations.

CHAPTER 2
LITERATURE REVIEW

2.1 GENERAL

In this chapter, an eloborative discussion is made regarding works done so far in this area as literature review. Fibre reinforced concrete with different fibres and their behaviour studies are discussed at the initial subheadings. Works on waste materials are discussed in the subsequent headings comprehensively.

2.2 REVIEW ON FIBRE REINFORCED CONCRETE

2.2.1 General

Ronald F. Zollo (1997) presented an overview regarding history and development of Fibre Reinforced Concrete in last 30 years. According to the literature, in the early 1960s, the works on fibre reinforced concrete have been started. A lot of research works has been conducted by many researchers on different fashions. Since local material is the focus of the project, there is no much literature available in both national and international level. Even majority of these projects have studied about steel fibres alone. So far there were only a few works which have studied about the other fibres like nylon, plastic, rubber and natural fibres. But those researches are completely different from the current study, since they have concentrated along the material strength properties not on strucutral behaviour.

2.2.2 History and Development

The concept of using fibres in a brittle matrix was first recorded with the ancient Egyptians who used the hair of animals and straw as reinforcement for mud bricks and walls in housing.This dates back to 1500 B.C. (Balaguru et. al, 1992). During the same period, straws were used to reinforce sun-baked bricks for a 57m high hill of ‘Aqar Quf’, near Baghdad.

During 1900's that asbestos fibres were developed, manufactured and widely used to augment mechanical properties of cement matrix as described by Bentur and Mindess (1990).

According to terminology adopted by American Concrete Institute (ACI) Committee 544, there are four catagories of Fibre Reinforced Concrete namely 1) SFRC (Steel Fibre Reinforced Concrete), 2) GFRC (Glass Fibre Reinforced concrete), 3) SNFRC (Synthetic Fibre Reinforced Concrete) and 4) NFRC (Natural Fibre Reinforced Concrete). It also provides the information about various mechanical properties and design applications. Cement and Concrete Institute also published the clasification of FRC in their website. Based on their classification, Fibres are classified into Glass, Steel, Sythetic (includes Acrylic, Aramid, Carbon, Nylon, Polyster, Polyethylene, Polypropylene) and Natural Fibres.

2.2.3 Mechanical Properties

C.B.Kukreja, S.K.Kaushik, M.B.Kanchi and O.P.Jain (1980) conducted some experiments and reported that based on the results of three methods such as split tensile test, direct tensile test and flexural test, split tensile strength test was recommended for fibrous concrete. Here also increase in tensile strength and post cracking strength , toughnes were reported.

Tensile strength of SFRC was also studied by researchers namely S.Goash, C.Bhattacharya and S.P.Roy (1989) and reported as inclusion of suitable short steel fibres increases the tensile strength of concrete even in low volume fractions. Optimum aspect ratio was found as 80 and the maximum increase in tensile strength was obtained as 33.14% at a fibre content of 0.7% by volume. Also it was reported that cylinder split tensile strength gave more uniform and consistent results than the modulus of rupture test and direct tension test.

Stress – Strain characteristics of steel fibre reinforced concrete under compression was studied by P.Sabapathi and H.Achyutha (1989). Cube compressive strength and Initial Tangent Modulus of Elasticity were obtained and equation for stress-strain relation was also proposed.

Distribution and orientation of fibres in FRC significantly affects the properties of FRC. Based on this concept Paviz Soroushian and Cha-Don Lee (1990) have carried out some investigation by counting the number of fibres per unit cross sectional area of SFRC specimen incorporationg various volume fractions of different fibres. Theoretical expressions were derived for the number of fibres per cross sectional area in fibre reinforced concrete as a function of volume fraction and length, assuming the cross sectional boundaries are the only factors distributing the 3-D random oriention of fibres. Comparisions were also made between number of fibres per cross sectional area and the reorientation fibres in concrete due to vibration.

To ascertain the tensile strength of fibre reinforced concrete, a simple testing set up was introduced to replace the costly direct tensile strength test apparatus by Youjiang Wang, Victor C. Li and Stanley Backer (1990). Methodology and testing procedure were also given. But still it requires servo controlled testing machine.

Stress – Strain behaviour of short, confined, reinforced concrete column with and without steel fibres was ascertained by N.Ganesan and V.Ramana Murthy (1990). The volume fraction of 1.5% with aspect ratio of 70 of steel fibres was used. The variable of the study was percentage reinforcement of lateral reinforcement. The strain at peak loads was increased to certain extent.

M. Ziad Bayasi and Paviz Soroushian (1992) reported that the rhelogical properties of SFRC are significant. The large surface area and interlocking property of fibres leads formation of balls among the concrete during mixing which can create damage to the hardened material properties. An experimental investigation was conducted by them to study the fresh concrete properties of concrete with different types of steel fibres. It was concluded that the fresh concrete workability preoperties of FRC were significantly affected by fibre reinforcing index. At a specific fibre reinforcing index, crimpled fibres seem to give slightly higher value than plain fibres.

Balaguru and Shah (1992) have reported that the fibres that are long and at higher volume fractions were found to ball up during the mixing process. The process called ‘balling’ occurs and causes the concrete to become stiff and a reduction in workability with increase volume dosage of fibres. This has a tendency to influence the quality of concrete and strength.

Mechanical properties of high strength fibre reinforced concrete were also studied by Faisal F Wafa and Samir A. Ashour (1992). A total of 504 test specimens were tested for different mechanical properties such as compressive strength, split tensile strength, flexural toughness and modulus of rupture. The mix was designed to achieve compressive strength of 94 N/mm^2. Three volume fractions of steel fibres such as 0.5%, 1.0% and 1.5% were selected. It was concluded that no real workability problem was encountered upto the addition of 1.5% volume fraction of fibres in concrete. Steel fibres enhanced the ductility and post cracking load carrying capacity of high strength concrete. 4.6%, 67%, 159.8% increase in compressive strength, modulus of rupture and split tensile strength were achieved by introducing hooked steel fibres as reinforcement in high strength concrete. Here also some emprical formulae were proposed in terms of volume fraction of fibres and compressive strength of conventional concrete.

$$f_{cf} = f_c + 3.53\ V_f \qquad (2.7)$$

$$f_r = 0.99\ f_c^{0.5} \qquad (2.8)$$

$$f_{sp} = 0.58\ f_c^{0.5} \qquad (2.9)$$

where,

V_f = Volume fraction of fibres

Consolidation of concrete on split tensile strength was studied by A.M.Shaaban and H.Gesund (1993). 0 to 8% of different mixes were selcted for the study. It is concluded that split tensile strength is increasd by 20% due to external vibration than standard rodded compaction.

Similar to the studies on steel fibre reinforced concrete, some works have also carried out by the researchers on synthetic fibre reinforced concrete. Ziad Bayasi and Jack Zeng (1993) have conducted some experimental works on workability and mechanical strength properties. Fibrillated polypropylene fibres of length ½ inch and ¼ inch at three volume fractions 0.1, 0.3 and 0.5% were used in concrete and workability properties such as slump, inverted slum cone, air content and mechanical strength properties such as compressive, impact and flexural behaviour were studied. In addition to them rapid chloride permeability is also conducted.

Impact test was conducted according to ACI Committee 544 method. Number of blows required to fail the cylindrical specimen is placed below a height of 457 mm. The hammer weight was about 45.5 kg. From this extensive research work, it was concluded that polypropylene fibres have no detectable effect on workablity upto 0.3%. Permeabilty of concrete was increased due to the addition of polypropylene fibres. Similarly post – peak flexural strength and impact resistance also increased due to the fibre addition.

Statistical prediction of compressive strength of steel fibre reinforced concrete has been carried out by V.Kumar, S.D.Suman and Mohammad Shamim (1997) and reported that the compressive strength of SFRC increases steeply with the increase of fibre content upto 1% (by volume) beyond which the rate of increase in strength reduces. It was also reported that the compressive strength of SFRC increases with the increase in the aspect ratio upto 60 and beyond this the rate of increase in strength reduces. It was furhter concluded that Fibre Reinforcing Index (FRI) significantly influences the compressive strength. the strength increased upto FRI = 90 for stright fibres and FRI = 60 for crimpled fibres. Beyond these values the rate of increase in strength started to decrease. They also proposed some statistical emprical relationships between compressive strength and FRI.

$$f_{cf} = af_c + bx^2 + cx^2 + dx^3 \qquad (2.4)$$

where,

x = Fibre Reinforcing Index (RI)

The values of co-efficients were determined using non-linear regression analysis as for straight fibres, a = 1.003, b = 0.214653MPa, c = -0.001136MPa, d = -1.22657×10^{-6} Mpa

for crimped fibres, a = 0.904, b = 0.1075MPa, c = 0.0006928MPa, d = -8.841×10^{-6} Mpa

M.C.Nataraja et.al (1998) have conducted for a study on steel fibre reinforced concrete under compression. Here the behaviour of steel fibre reinforced concrete under compression for cylinder compressive strength ranged from 30 to 50 N/mm^2. Round crimpled fibres with three volume fracions of 0.5 percent, 0.75 percent and 1.0 percent and for two aspect ratios of 55 and 82 are considered. The effect of fibre addition to concrete on compressive strength was studied. It was concluded that the addition of fibres increases the compressive strength and toughness. Some square fitting line analyses were also done and some equations were proposed for compressive strength in terms of fibre reinforcing index (length of fibre x volume fraction).

$$f_{cf} = f_c + 2.1756\ (RI) \qquad (2.1)$$

(for strength upto 30 N/mm^2)

$$f_{cf} = f_c + 2.145\ (RI) \qquad (2.2)$$

(for strength between 30 to 50 N/mm^2)

$$f_{cf} = f_c + 2.2604\ (RI) \qquad (2.1)$$

(for strength upto 50 N/mm^2)

An intresting study was carried out by Rami H. Haddad and Ahmed M. Asteyate (2001) to predict the role of synthetic fibres such as polypropylene and nylon fibres in delaying steel corrosion cracks and improving the bond with concrete. Different length of polypropylene and nylon fibres with various volumes were mixed with concrete.Pullout tests and corrosion study were conducted and concluded that both the fibres contributed more in delaying the corrosion and improving the bond strength. Moreover it is pointed out that polypropylene firbes play a major role than nylon fibre in the improvement of bond.

Another intresting investigation was carried out by Yaghoub Mohammadi ans S.K.Kaushik (2003) about the effect of mixed aspect ratio of fibres on mechanical strength properties of concrete. 25 mm – 50 mm long crimped type flat steel fibres were mixed in different proportions with concrete and tested for split tensile, compressive, static flexural strength. Compressive toughness and flexural toughness were obtained from the test results. It is found that use of 65% long fibres and 35% short fibres gives optimum composite properties when compared with other mixes. An important note also was given in that literature that use of mixed aspect ratio of fibres does not have a significant effect on the static modulus of elasticity.

Wu Yao, Jie Li and Keru Wu (2003) have examined the mechanical behaviour of hybrid fibre reinforced concrete at low fibre volume fraction. Three hybrid composites such as polypropylene and carbon, carbon and steel and steel and polypropylene fibres were chosen and the mechanical strength properties such as Compressive strength, split tensile strength, modulus of rupture and flexural toughness were ascertained. A statistical response surface method and three level full factorial experimental design were used to study the effects of volume fraction and aspect ratio of fibre on fractional energy, compressive strengh, splitting tensile strength, flexural strength and characteristic length of steel fibre reinforced concrete. When the mechanical properties are alone considered, the optimal values of design variables such as

0.64% for volume fraction and 76.44 for aspect ratio were obtained. If both mechanical properties and cost optimisation are considered, volume fraction of 0.558% and aspect ratio of 75.87 were obtained.

Job thomas and Ananth Ramasamy (2007) carried some experimental investigation on mechanical properties of steel fibre reinforced concrete. Three different strengths such as normal strength (35 MPa), moderately high strength (65 Mpa) and high strength (85 Mpa) concrete mixes were selected for this study. 30 mm long steel fibres (Aspect ratio of 55) with three different volume fractions as 0.5%, 1.0% and 1.5% were selected and uniformly distributed in the mix. The mechanical strength properties such as 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 were studied. Based on 60 test data regression analysis is done and emprical relations were provided.

Based on the test results of 320 numbers of specimens on mechanical strength properties of high strength fibrous concrete, J.Premalatha and R.Sundararajan (2007) suggested no significant improvement in compressive strength was obtained beyond 1.5% volume fraction of steel fibre content. The high strength concrete mix was designed to achieve 60 N/mm^2 and their strength properties like compresive strength, modulus of rupture, split tensile strength were studied and emprical relations were also proposed in terms of Fibre Reinforcing Index (FRI).

$$f_{rf} = f_r + 5.85F \qquad (2.5)$$

$$f_{spf} = f_{sp} + 3.42F \qquad (2.6)$$

where,

F = Fibre Reinforcing Index (RI)

Effects of aspect ratio and volume fractions of steel fibre on the mechanical properties of SFRC were studied by Semsi Yazici et.al (2007). Three aspect ratios (l/d) of 45, 65, 80 and three volume fractions of 0.5%, 1.0%, 1.5% hooked end bundled fibres were taken in that study. Increase in the aspect ratio decreases the workability. In compressive strength 4 – 19%

incraese, in split tensile strength 11 – 54% increase and in flexural strength 3 – 18% increase were obtained from the experimental investigation. A multilinear regression analysis was used to obtain the following relationships.

$$f_c \text{ (Mpa)} = 50.4869 + 0.0434 \times l/d + 1.9667 \times V_f (\%) \quad (2.10)$$

$$f_{sp} \text{ (Mpa)} = 2.2121 + 0.0077 \times l/d + 1.4233 \times V_f (\%) \quad (2.11)$$

$$f_r \text{ (Mpa)} = 0.8261 + 0.0638 \times l/d + 3.000 \times V_f (\%) \quad (2.12)$$

2.2.4 Fibre Reinforced Concrete beams under static loading

M.Lakshmipathy and A.R.Santhakumar (1987) have conducted an experimental analytical investigation on two span continuous beams with steel fibres. The important characteristics such as cracking behaviour, ductiltiy and energy absorption were ascertained from experimental investigation and compared with analytical results. The fibrous concrete beams served superior than conventioanl concrete.

An experimental investigation was carried out by D.L.N Rao, M.R.A Kadir and Kawa Taha Abu Al Awffa (1987) on deformation characteristics and strength of reinforced concrete beams made with steel fibres in pure bending. 1.85m span beams were casted and tested under static flexural loading. The increase in depth of neutral axis and hence flexural stiffness of fibre reinforced concrete beams at all stages of loading reflected the ability of fibres in arresting the crack growth. The inclusion of steel fibres in the concrete significantly increased the post cracking stiffness at all the stages upto failure. Lim.T.Y, Paramasivam.P and Lee.S.L (1998) carried out some experimental and analytical study on bending behaviour of steel fibre reinforced concrete beams. A simplified Moment – Curvature (M–Ø) relationship for beam with rectangular was proposed. The proposed model was verified with experimental results carried out 2.2m span reinforced concrete beams. The enhancement in ductility and energy absorption under static loading was delivered through the study.

Balaguru and Shah (1992) reported that the modern developments of using only straight steel fibres began in the early 1960's. Till now, a wide range of other type of fibres was used in cement matrices. Construction industries have led the development of conventional fibres such as steel, stainless steel and glass.

Cheng-Tzu Thomas Hsu, Rujun Linda He and Samer Ezeldin (1992) were presented a computer algorithm to analyse the load – deflection and moment – curvature behaviour of steel fibre reinforced concrete beams. A total of 11 SFRC simple beams were casted and tested under two point loading. The experimental results obtained were compared with computer program results.

Flexural behaviour of fibre reinforced concrete beams were experimented by H.V.Dwarakanath and T.S.Nagaraj (1998). In this study fibres were put in two types of locations such as over the entire depth and oner half the depth of the beam on the tension side. 20 numbers of 1.8m long reinforced concrete beams with steel fibres were tested under flexural static loading. Midspan deflections and curvatures at salient points such as cracking, and ultimate points were compared. It is found that half depth mode of inclusion of fibres for under reinforced concrete beams and full depth mode of inclusion of fibres for over reinforced concrete beams.

Antonia F. Barbosa and Gabriel O. Ribeiro (1998) worked on ANSYS for finite element analysis of reinfroced concrete structures. A simply supported reinfroced concrete beam subjected to uniformly distributed load was taken as a simple example in that study. Two different models were considered for steel reinforcement such as discrete and smeared. Load – deflection curves obtaind through ANSYS have been compared with experimental results and found satisfactory.

Piti Sukontasukkul (2004) conducted an experiemntal investigation on toughness of steel and polypropylene fibre reinforced concrete beams under bending using two different methods such as ASTM C1018 and JSCE SF-4. The behaviour of steel fibre reinforced concrete was single peak response whereas polypropylene fibre reinforced concrete was double peak response. The deformations under two methods were compared.

Earlier a similar study has been conducted by S.K.Padmarajaiah and Ananth Ramaswamy (2004) on flexural strength of steel fibre reinforced high strength concrete in fully and partially prestressed beam specimens. It is found that the toughness and ductility of prestressed high strength concrete beams have been increased with the increase in fibre content. The maximum increase in ductility was 18%, 45% and 68% and percentage increase in energy absorption 25%, 78% and 88% for fully prestressed beams with full depth of steel fibre content of 0.5%, 1.0% and 1.5% volume fractions.

Experimental investigation and Analytical modeling for flexural behaviour of reinforced fibrous concrete beams using synthetic fibres were performed by D.Suji, S.C.Natesan and R.Murugesan (2006). Graded fibrillated polypropylene fibres were used in this study. 1.8 m long rectangular reinforced concrete beams were casted with and without fibre at different volume fractions of 0.1%, 0.2% and 0.3%. Moment carrying capacity of beams were arrived and compared with theoretical equations. Also it was concluded that the crack pattern was remains same for all beams, but the rack with and length were reduced for fibre reinforced concrete beams.

Ultimate strength of steel fibre reinforced self compacting concrete beams were tested by Ganesan.N, Indira.P.V and Santhosh kumar.P.T (2006). 1.2m long reinforced concrete beams are prepared using self compacting concrete with steel fibres of three volume fractions of 0.25%, 0.5% and 0.75%. It is foiund that strength and ductility of fibre reinforced self compacting concrete specimens have increased substantially over conventional concrete.

Load – Deflection performance of partialy prestressed concrete T-beams with steel fibres in partial and full depth was assessed by J.Thomas and A.Ramasamy (2006). Three mixes M35, M65 and M85 were used with steel fibres at a volume fraction of 1.5%. Totally 6 numbers of 3.85 m long T-beams with simply supported span of 3.6m prestressed with 7 mm wires were casted and tested under four point bending. Analytical models were proposed for computation of load – deflection and moment – curvature. Comparisions were made between the experimental and analytical results.

The same authors Job thomas and Ananth Ramasamy (2006) reported the details of finite element modeling and analysis of some shear critical prestressed steel fibre reinforced concrete beams. A commercially available finite element software 'ANSYS' was used to analyse the beams. The concrete was modelled using 'SOLID 65'- eight node brick model, which is capable of simulating the cracking crushing behaviour of brittle materials. The reinforcements were modelled using 'LINK 8' – 3D spar eement. The slip between reinforcement and the concrete has been modeled using 'COMBIN 39' – non linear spring element. The ANSYS correctly predicted the diagnal tension and shear failure of prestressed csoncrete beams observed in the experiement.

Finite element analysis using ANSYS was done by Greeshma.S and Jaya.K.P (2007) to analyse a shear wall under seismic loading. Modelling of shear wall was done using SOLID 65 model and reinforcements were modeled using LINK 8 element. The analyses were carried out for the shear wall subjected to both static and dynamic loading.

2.2.4 Fibre Reinforced Concrete beams under cyclic loading

One of the improtant applications of fibre reinforced concrete is earthquke resistant structures. Not only earthquakes most of the unanticipated loadings are cyclic in nature. So it is mandatory to study the beahaviour of fibre reinforced concrete beams under cyclic loading which simulates the seismic motion.

The critical seismic design parameter called cumulative ductility indicator was proposed by Banon H, Biggs J and Irvine H (1981).

$$D = \Sigma (\delta_{m,i} – \delta_y) / \delta_y = \Sigma (\mu_i - 1)$$

Where,

$\delta_{m,i}$ – maximum displacement in i^{th} cycle

μ_i - ductility in i^{th} cycle

Roufail and Meyer (1987) proposed some analytial modelling of hysteretic behaviour of reinforced concrete strucutres. Measures of stiffness degradation have been considered as damage indicators.

$$D = \frac{K_f \; (k_m - k_0)}{K_m \; (k_f - k_0)}$$

Where,

k_f – Secant stiffness at failure

k_m– Secant stiffness at the maximum deformation caused by the applied loads

k_o – Initial tangent stiffness

But in the above equation effect of repeted cycclic loading was not considered.

Kratzig W.B, Meyer I.F and Meskouris K (1989) propsed a model to evaluate the damage index in reinfroced concrete under cyclic loading. The proposed damage index was based on the hysteric energy absorbed by a member. The first loading cycle at given amplitude is termed as primary half cycle, with subsequent cycle at the same or smaller amplitudes termed as follows. Then the damage index for the positive half cycle was defined as

$$D^+ = \frac{\Sigma E^+_{p,i} + \Sigma E^+_i}{\Sigma E^+_f + \Sigma E^+_i}$$

Where,

$E^+_{p,i}$ – energy absorped in a primary half cycle

E^+_I - energy absorped of a follower half cycle

E^+_f – energy absorped in monotonic test to failure

With a similar index defined for a negative cycles, overall damage index was given by,

$$D = (D^+ + D^-) – (D^+D^-)$$

Wang M.L and Shah S.P have proposed a reinfroced concrete hysteritic model on the damage concept. The proposed damage was a simple one in which the rate of accumulation of damage is assumed proportional to the damage already incurred. The proposed equation was

$$D = \frac{\exp(s\alpha) - 1}{\exp(s) - 1}$$

Where,

$$\alpha = c\Sigma \frac{\delta_{m,i}}{\delta_f}$$

c, s are constants, the value of constants c = 0.1 and s = 1.0 for well reinforced member.

Simulation of highly ductile fibre reinforced cement based composite componenets under cyclic loading was carried out by Tong-Seok Han, Peter h. feenstra and Sarah L. Billington (2003). Experiments on cantilever beams and corresponding finite element models were proposed in the literature.

Similarly cyclic response of highly ductile fibre reinforced cement based composites were ascertained by Keith E. Kesner, Sarah L.Billington and Kyle S. Douglas (2003) and reported that improvement in performance was in DFRCC members.

U.B. Choubey, Uttamasha and B.K.Prasad (2006) have reported some studies on cyclic behaviour of latex modeified reinforced concrete beams. Tests on 14 numbers of different reinforced concrete beams with and without latex were casted and studied. In that study forward cyclic loading was applied and load-deflection curves were obtained.

2.2.5 Fibre Reinforced Beam Column Joint under cyclic loading

An experimental investigation was conducted on the behaviour of beam-column joints with inclined reinforcing bars under seismic condition and reported by A. G. Tsonos, I. A. Tegos and G. Gr. Penelis (1992). By conducting experiments on twenty joints, it was concluded that, improvement in the ductile behaviour of exterior beam-column joints resulting in the presence of inclined reinforcing bars in their core region.

A MS thesis was presented by Michael Gebman (2001) on the application of SFRC in building frames to help provide a basis for possible modifications to the building code. The thesis was focused on frame joints, particularly on the improvement in joint seismic performance by using steel fibre reinforced concrete. By adding hooked steel fibres to reinforced concrete, the joint was toughened which enables the structure to survive strong earthqukes. Simulated quasi-static earthquke loading was applied on the joints and hysteresis loops were drawn. Based on the curves, ductility, beam cracking and column cracking were observed carefuly and found that stel fibres enhances all the structural behavior of joints.

SIFCON is a special type of fibre reinforced concrete at high volume fraction (4 to 20%) in which the formwork is filled to capacity with steel fibres and the resulting fibre network is then infiltrated by a cement based slurry. Ductile behaviour of SIFCON structural members were studied by G.S.Thirugnanam, P.Govindan and A.Sethurathnam (2001). The investigation was carried out in two stages. Initially single span beams were tested to quantify the structural behaviour in the hinging zones of flexural members. In the second stage, multi bay,multi-storey RC frames were tested with SIFCON beam-column joints to study the structural behaviour under cyclic loading. The seismic parameters such as cumulative ductiltiy, cumulative energy absorption, stiffness degradation and failure mode were obtained. It was concluded that Ductiltiy was increased to 100% and energy absorption was increased about 50% in hinging zones of structural joints because of SIFCON.

An experimental investigation was carried out by Mustafa GENCOGLU, Iihan EREN (2002) to study the effect of steel fibre reinforced concrete on the behaviour of the exterior beam-column joints subjected to reverse cyclic loading. To achieve the ductility in joints for earthquake resistant structures, closely spaced reinforcements are recommended by the code. To avoid the congestion of reinforcement, alternate solution was given in the literature. Steel fibres of aspect ratio 75 were added at a 1% volume fraction. Tests were conducted by applying reverse cyclic loading and mode of failure and energy dissipation capacity was observed. It was concluded that steel fibres can be used as alternative for the increase in confining reinforcement so as to minimise the congestion of reinforcement at beam-column joint and hence reduce the problem of consolidation of concrete.

Intrinsic response control of moment resisting frames using advance composite materials and structural elements was tested by Gregor Fischer and Victor C. Li (2003). In this research work, the load-deformation response of a composite frame system under reverse cyclic loading condition was investigated. From this work it was concluded that engineered cementatious composites and fibre reinforced polymer reinforcements can be used as alternative approach for earthquke resistant structures with improved performance in terms of dynamic response, residual displacement, damage tolerance and rehabilitation requirements.

Seismic performance of confined high strength concrete square columns with carbon fibre reinforced polymers were experimented by A.Hosseini, Ali R. Khaloo and S.Fadaee (2005). To evaluate the performance of high strength, reinforced concrete columns confined with carbon fibre reinforced polymers, six square concrete columns were tested under constant axial load and cyclic lateral load. The energy damage indicator and ductiltiy parameters of strengthened columns were improved. Both system and section ductility and energy dessipation were improved considerably in strengthened joints.

As seismic design of structures moves towards performance based design, there is need for a few strcutural members and systems that possess enhanced deformation capacity and damage tolerance, while requiring simple reinforcements.One option for achieveing thiss goal was given and experimented by Gustavo J. Parra-Montesinos, Sean W. Peterfreund and Shih-HO Chao (2005). Using High-performeance fibre-reinforced cement composites, high damage tolerant beam-column joints were achieved. It is concluded that beam-column joints constructed with an HPFRCC material containing a 1.5% volume fraction of ultra-high molecular weight polyethylene fibres exhibited excellent strength, deformation capacity and damage tolerance. It is also reported that use of HPFRCC materials in beam plastic regions allowed an increase in transverse reinforcement spacing to half the effective beam depth.

P.Asha et.al (2006) carried out an experimental investigation on cyclic response of reinforced concrete exterior beam – column joints. Two types of reinforcement were considered namely circular hoop and conventional reinforcement. Load-displacement hysteresis loops were drawn and hence cumulative enerfgy absorption and ductility were ascertained. Similar study was carried out by Valeria Corinadesi and Giacomo Moriconi (2006) on behaviour of beam-column joints made of sustainable concrete. In this two different types of aggregares namely natural aggregates and recycled aggregates were used. In order to ascertain the real scale behaviour, several beam-column joints were casted and tested under low cycle loading. There are two options in seismic design. One is design of structures with full strength so it will respond elastically while ensuring adequate ductility and energy absorption. Second one permits the structure to be designed for considerably lower forces than those required for first one. Here also ductility and energy dessipation capability were ascertained based on the load-displacement hysteristic loops.

Strength and ductility of partially confined bridge column under seismic loading were studied by D.Prapakaran and R.Sundararajan (2007). The main objective of the study was to investigate the influence of partial confinement at plastic hinge region on the structural characteristics of circular bridge column. Based on the lateral load - tip displacement hysteristic loops, cumulative energy absorption were calculated and compared.

2.2 REVIEWS ON WASTE FIBROUS MATERIALS

2.2.1 General

India owns huge amount of waste materials in the form of organic and inorganic state. Nowadays inorganic waste materials such as plastic, nylon, rubber are produced in massive volumes because of increase in use of inorganic materials for various purposes such as automobile parts, households, industrial wastes etc., One of the leading problem of generating these inorganic waste materials is disposal without environmental pollution. Common methods of solid waste disposal are land filling and incineration. But these methods lead uneconomic. So, attempts to reuse the waste materials for construction purposes have been made by many researchers in many forms. Similarly organic natural fibres are abundantly available in many parts of the world. For different reasons, developing countries recognise the importance of the use of ecologically friendly and cost effective materials in urban and rural buildings. This section will elaborate the studies carried by the researchers on inorganic fibrous materials.

2.2.2 Review on inorganic fibres

Gonzalo Martinez-Barrera et.al (2006) reported about concrete reinforced with irradiated nylon fibres. Modified nylon fibres were mixed with cement mortar at 1.5%, 2.0% and 2.5% volume fractions. The compressive strength of the fibre reinforced concrete was evaluated.

Some studies on properties of concretes containing reengineered plastic shred fibre were studied by K.Anbuvelan, M.M.Khadar, M.H.Lakshmipathy and K.S.Sathyanarayanan (2007). Reengineered plastic shred fibres are made by re-processing the plastic waste and then rolling it into plastic sheets which were subsequently shredded into fibres of required dimensions. Compressive, split tensile, flexural, abrasion, impact strength and plastic shrinkage studies were studied by the authors. It was concluded that the engineering properties were improved by the addition of plastic shred fibres.

2.2.3 Review on Natural Fibres

Natural fibres are prospective reinforcing materials and their use until now has been more traditional than technical. They have long served many useful purposes but the application of materials technology for the utilization of natural fibres as the reinforcement in concrete has only taken place in comparatively in recent years. The distinctive properties of natural fibre reinforced concretes are improved tensile and bending strength, greater ductiltiy, greater resistance to cracking and hence improved impact strength and toughness. Besides its ability to sustain loads, natural fibre reinforced concrete is also required to be durable. Durability relates to its resistance to deterioration resulting from external causes as well as internal causes (M.A.Aziz, P.Paramasivam and S.L.Lee 1984).

Mechanical characterization and impact behaviour of concrete reinforced with natural fibres were studied by S.K. Al-Oraimi and A.C.Seibi (1995). Here an experimental study was conducted using glass and palm tree fibres on high strength concrete. Mechanical strength properties such as compressive, split tensile, flexural strengths and post cracking toughness were studied. It was concluded that natural fibres are comparable with glass fibres. A finite element analysis was also done using ANSYS software. Both analytical and experimental results were compared and acceptable.

Rheological properties of coir fibre reinforced cement mortar were carried out by G.Ramakrishna and T.Sundararajan (2002). Flow value, cohesion and angle of internal friction were determined for three diffrenet mix ratios and four different aspect ratios and fibre contents. Based on the rheological properties of fresh mortar, it was recommended to use shorter fibres with low fibre-content for achieving workability and higher fibre content for better cohesiveness in wet state.

G.Ramakrishna, T.Sundararajan and Usha Nandhini (2002) compared the theoretical and experimental investigations on the compressive strength and elastic modulus of coir and sisal fibre reinforced concretes for various volume fractions. It was observed that both the experimental and analytical values of elastic modulus had shown 15% discrepancy, which can be regarded as comparitively small.

Sugarcane bagasse fibre reinforced cement composites were studied by K.Bilba, M.A.Arsene and A.Ouensanga (2003). Various bagasse fibre-cement composites were prepared and influence various parameters on the setting of the composite materials were studied. Botanical components, thermal and chemical treatment of bagasse fibres were also studied.

The natural fibre composites may undergo a reduction in strength and toughness as a result of weakening of fibres by the combination of alkali attack and mineralisation through the migration of hydrogen products to lumens and spaces. Romildo D. Toledo Filho, Khosrow Ghavami, George L. England and Karen Scrivener (2003) reported their study on development of vegetable fibre-mortar composites of improved durability. So, several approaches were proposed by the authors to improve the durability of vegetable fibre-cement composites. These include carbonation of the matrix in a CO_2-rich environment; the immersion of fibres in slurried silica fume prior to incorporation in Ordinary Portland Cement matrix; partial replacement of Ordinary Portland Cement by undensified silicafume or blast furnace slag. The performance of modified vegetable fibre-mortar composites was analysed in terms of effects of aging in water, exposure to cycles of wetting and drying and open air weathering on the microstructures and flexural behaviour. It was suggested that immersion of natural fibres in a silica fume slurry before the addition to the cement based composites was found to be effective means of reducing embrittlement of the composite in the environments. Also early cure composites in a CO_2-rich environment and the partial replacement of OPC by undensified silicafume were the efficient approaches in obtaining natural fibres in improved durability.

Robert S.P. Coutts (2005) reviewed critically about the Australian research into natural fibre cement composites. It was mentioned that over the last three decades considerable research has been commited to find an alternative fibre to replace asbestos and glass fibres. V.Agopyan et.al (2005) reported the devlopments on vegetable fibre-cement based materials in Brazil. Taking into account the mechanical properties, with an adequate mix design, it is possible to develop a material with suitable properties for building purposes. To overcome the drawback, it was suggested that durability of natural fibres can be impoved by making alternative binders with controlled free lime using ground granulated blast furnace slag.

Romildo D. Toledo Filho et.al (2005) made some experiments on free, restrained and drying shrinkage of cement mortar composites reinforced with vegetable fibres. The free and restrained shrinkage were studied by subjecting the specimens to wind speed of 0.4-0.5 m/s at 40^{o} C temperature for upto 280min. Drying shrinkage tests were carried out at room temperature with about 41% relative humudity for 320 days. It was concluded that free plastic shrinkage is significantly reduced by the inclusion of 0.2% volume fraction of 25mm short sisal fibres in cement mortar. Also it was stated that the presence of sisal and coconut fibres promotes an effective self-healing of plastic cracking after 40 days at 100% RH. The drying shrinkage was increased by upto 27% when up to 3% of sisal and coconut fibres were present in that stud.

The capability to absorb energy, called toughness is important in actual service conditions. For that an experiemntal investigation was carried out by Ramakrishna.G and Sundararajan (2005) on impact strength of a few natural fibre reinforced cement mortar slabs. Four types of natural fibres such as coir, sisal, jute and hibiscus cannebinus with four different fibre contents such as 0.5%, 1.0%, 1.5% and 2.0% by weight of cement were used. The tests were carried using repeated projectile test apparatus and the performance of specimens was ascertained based on the parameters namely impact resistance, residual impact ratio, crack resistance ratio and the condition of fibre at ultimate. From this elaborative test results, it was concluded that coir fibres absorb more energy ie., 253.5 J at 2% fibre content and fibre length of 40 mm. Coir fibre reinforced slab specimens exhibit fibre pull out failure, whereas all other types of fibre reinforced specimens exhibit fibre fracture at ultimate failure.

Some studies have been also conducted by Ramakrishna.G and Sundararajan (2005) on the durability of natural fibres and the effect of corroded fibres on the strength of mortar. Coir fibres were found to retain higher percentages of their initial strength than all other fibres after the specified exposure in the various mediums.

Mechanical properties of date palm fibres and concrete reinforced with date palm fibres were tested and reported by A.Kriker et.al (2005) in two different climates. In addition to the above properties, continuity index, microstructure and toughness were also studied. The volume fraction and length of fibres chosen were 2-3% and 15-60mm respectively. It was concluded that male date palm fibre got more tensile strength. Also it was stated that observing micro-structue of the fibre-matrix interface cured in hot-dry and water environments. Based on the results and observations of that work, it was suggested that future research should be developed on the treatment of Male date palm surface fibre concretes to improve their mechanical properties using local industrial wastes, especially hot-dry climate.

Microstructure and mechanical poperties of waste fibre-cement composites were studied by H. Savastano Jr, P.G.Warden and R.S.P.Coutts (2005). Both secondary and back-scattered electron imaging and energy dispersive X-ray spectrography were used for compositational analysis. It was concluded that sisal waste fibres presented satisfactory bonding in matrices. BSE images and EDS analyses confirmed the fibre-matrix transition zone can be improved by using production process based on vaccum dewatering and pressure.

K.Murali Mohan Rao and K.Mohana Rao (2005) introduced and studied the extractio and tensile properties of new natural fibres used as fillers in a polymeric matrix enabling production of economical and light weight composites for load carrying structures. The cross sectional shape, the density and tensile properties of these fibres along with established natural fibres like sisal, bana, coconut and palm were determined experimentally under similar conditions and compared. The density of newly introduced fibres such as vakka, date and bamboo were less than the existing fibres.

CHAPTER 3

EXPERIMENTAL INVESTIGATION

3.1 INTRODUCTION

This chapter demonstrates the detailed exprimental programme of this investigation . It includes materials and fibres used, detailed methodology of experimetal programme, mix proportions, specimen details, reinforcement detailing and test set up. In this experimental investigation, initially three organic fibres such as plastic, nylon, tyre and three inorganic fibres such as coconut coir, sugarcane bagasse and rice husk have been taken. The properties are compared with well known steel fibres. Their physical properties such as aspect ratio, specific gravity, water absorption, density and ultimate tensile load are studied. However natural inorganic fibres are subjected to ageing process in different environments in which they may suffer a reduction in strength and toughness. So it is mandatory to study the microstructure properties of fresh as well as used (reacted with concrete for two years) natural fibres through SEM with EDS and X-ray difraction. Since the focus of this research is aimed on reuse of waste fibrous materials, an overall characterization with different characteristics are required to recommend the materials. This characterization is divided into many parts. Initially seven fibres (steel, nylon, plastic, tyre, coconut coir, sugarcane bagasse and rice husk) of three different aspect ratios (30, 60, 90) and three volume fractions (0.5%, 1.0% and 1.5%) have been taken. First the influence of different fibres on workability of concrete are studied. Whilst the slump test is commonly used to assess the workability of conventioanl concretes, it is not generally suitable for natural fibre reinforced concrete (Aziz M.A et.al 1984). Hence in this investigation, both slump test as well as Vee-bee test have been conducted to assess the workability of fibre reinforced concretes.

To achieve the optimim length and volume fractions of fibres for structural studies, mechanical strength properties such as compressive strength, split tensile strength, modulus of rupture, modulus of elasticity, shear strength and impact energy have been conducted. From the test results optimum length of 50mm and two different volume fractions 0.5% and 1.0% have been selected for further structural study. Behaviour of reinforced concrete beams with all seven fibres of two different volume fractions have been studied under both monotonic and cyclic (psuedo static) loading. From the performance behaviour of beams nylon, plastic and coir fibres of 1.0% volume fraction are slected for further study. Since the field studies have shown that beam-column joints are vulnerable during an intensive earthquake, the work has been extended to study the behaviour of beam-column joints with fibre composites.

Besides the strength behaviour, durability of proposed concrete is also important. Durability relates to its resistance to deterioration resulting from external and internal causes. The internal causes include volume change and permeability of concrete. So, in this study effects of plastic shrinkage, drying shrinkage and permeability on selcted fibre reinforced concrete have been studied.

3.2 MATERIALS AND MIX

The materials used in this investigation were: ordinary portland cement, coarse aggregate of crushed rock with a maximum size of 20 mm, fine aggregate of clean river sand and portable water. 8mm dia HYSD bars were used as main reinforcement. 6mm dia MS bars were used as stirrups. Commercially used MS wires (binding wires) were used as steel fibres. Locally available materials such as nylon, plastic, tyre, coconut coir, sugarcane bagasse, rice husk were taken from the waste stream and converted in to fibres of required length and diameter. The detailed properties are given in subsequent contents.

3.2.1 Cement

Ordinary Portland Cement of 43 grade conforming to IS 8112-1989 was used. Tests were carried out on vatious physical properties of cement and the results are shown in Table 3.1

3.2.2 Fine Aggregate

Natural river sand was used as fine aggregate. The properties of sand were determined by conducing tests as per IS: 2386 (Part- I). The results are shown in Table 3.2. The results obtained from sieve analysis are furnished in Table 3.3. The results indicate that the sand conforms to Zone II of IS: 383 – 1970

3.2.3 Coarse Aggregate

Crushed granite stones obtained from local quaries were used as coarse aggregate. The maximum size of coarse aggregate used was 20 mm. The properties of coarse aggregate were determined by conducting tests as per IS: 2386 (Part – III). The results are tabulated in Table 3.4

3.2.4 Water

Portable water free from salts was used for casting and curing of concrete as per IS: 456 – 2000 recommendations.

3.2.5 Fibres

Locally available waste materials were collected from different stream and properly shaped in the form of fibres. Uniform length of fibres was obtained by using cutting machine. Plastic fibres were colleceted from plastic pot industry waste. Nylon waste fibres were collected from nylon industries.

Table.3.1 Physical Properties of 43 Grade Ordinary Portland Cement

Physical Properties	**Values of OPC used**	**Requirements as per IS 8112-1989**
Standard Consistency	29.2%	-
Initial Setting Time	45 Minutes	Minimum of 30 minutes
Final Setting Time	265 Minutes	Maximum of 600 minutes
Specific gravity	3.15	-
Compressive strength in N/mm^2 at 3 days	29	Not less than
Compressive strength in N/mm^2 at 7 days	38.5	Not less than
Compressive strength in N/mm^2 at 28 days	48	Not less than

Bi-cycle rubber tyres were collected from local automobile workshops. Locally and easily available natural fibres such as coconut coir fibre, sugarcane baggasse and rice husk were collected from local coconut oil mills, sugar industries and rice mills respectively. Commercially available GI Steel wires which are used as binding wires for reinforcement tieing were used. The clear photographs of different fibres are shown in Fig. 3.2 to Fig.3.8. The geometrical and physical properties of different fibres are tabulated in Table 3.5.

Table.3.2 Sieve Analysis of Fine Aggregate

I.S. Sieve Size	Weight Retained (gm)	Cumulative Weight Retained (gm)	Cumulative Percentage Weight Retained	Cumulative Percentage Weight Passing
10 mm	2	2	0.4	99.6
4.75 mm	6	8	1.6	98.4
2.36 mm	20	28	5.6	94.4
1.18 mm	76	104	20.8	79.2
600 microns	224	328	65.6	34.4
300 microns	114	442	88.4	11.6
150 microns	54	496	99.2	0.8
< 150 microns	4	500	100	0.0

Remarks: Conforming to Zone II of Table 4 of IS: 383-1970

Table.3.3 Physical Properties of Fine Aggregate

(Tests as per IS: 2386 – 1968: Part III)

Physical properties	**Values**
Specific gravity	2.6
Fineness Modulus	2.83
Water Absorption	0.75%
Bulk density (kg/m^3)	1654
Free moisture content	0.1%

Table.3.4 Physical Properties of Coarse Aggregate

(Tests as per IS: 2386 – 1968 Part III)

Physical properties	Values
Specific gravity	2.6
Fineness Modulus	2.73
Water Absorption	0.5%
Bulk density (kg/m^3)	1590
Free moisture content (%)	0.2%
Aggregate Impact value (%)	11.2
Aggregate Crushing value (%)	25.12

Diameter of fibres were measured through microscope of capacity……. . The specific gravity was determined based on the method specified in IS: 2386 (Part III). Five gram of each fibre samples was accurately weighed in an electronic balance of accuracy….. and the water absorbed after 24 hours of continuous immersion was determined. Except for rice husk, the ultimate tensile strength was determined by the tension test using 4 kN tensile testing machine. A gauge length of 100 mm was chosen to measure the maximum elongation.

Table.3.5 Typical Properties of Fibres

Properties of Fibres	**Type of Fibre**						
	Steel	**Nylon**	**Plastic**	**Tyre**	**Coir**	**Sugarcane**	**Rice Husk**
Diameter/Equivalent Diameter (mm)	0.60	0.44	1.51	1.50	0.48	1.50	1.60
Aspect Ratio	83.3	113.6	33.1	33.1	104.2	33.1	12.5
Specific gravity	5.86	0.7	1.25	1.08	0.87	0.52	0.4
Water Absorption (%)	33.33	66.66	66.66	75	210	286.6	225.66
Density in kg/m^3	6879	657	763	530	2057	260	564

 **Fig.3.1 Steel Reinforcements**	 **Fig.3.2 Steel Fibres**
 **Fig.3.3 Nylon Fibres**	 **Fig.3.4 Plastic Fibres**
 **Fig.3.5 Tyre Fibres**	 **Fig.3.6 Coir Fibres**
Fig.3.7 Sugarcane Fibres	 **Fig.3.8 Rice Husk Fibres**

3.2.6 Reinforcements

Fe 415 steel bars of diametrs 8 mm and 12 mm were used as main reinforcements for beams and beam-column joints respectively. Mild steel 6 mm diameter rods were used as stirrups and ties.

3.2.7 Mix Proportion

A mix was designed as per IS 10262 – 1982 to achieve a minimum target strength of 20 N/mm^2. The same mix was used for all type of fibre reinforced concretes. The mix proportion was 1:1.38:3.09. A constant water cement ratio of 0.5 was used. The quantities of different ingredients per cubic meter of concrete mix were given in Table.3.6.

Table.3.6 Quantities of ingredients of Concrete Mix

Water	Cement	Fine Aggregate	Coarse Aggregate
185 kg	350 kg	483 kg	571 kg

3.3 CASTING AND CURING

A laboratory type concrete mixer machine was used to mix the ingredients of concrete.To avoid balling of fibers, the following procedure was followed in casting [ ]. First, aggregates and cement were mixed for one minute, water being added within two minutes. Then fibers were uniformly dispersed by hand throughout the mass with slow increment. Now concrete was allowed to mix for three minutes. All the specimens were well compacted using a table vibrator, and cured for 28 days.

3.4 EXPERIMENTAL SET UP

This section deals about the experimental set up, spcimen details and testing procedure of each test planned. The photographs of each test set up also presented in each sub sections.

3.4.1 Workability Studies

The rheological properties of fibre reinforced concrete are significant. The large surface area of fibres tends to restrain flowability and mobility of the mix. Interlocking of fibres and consequently the formation of fibre balls can be very damaging to the hardened material properties (M.Ziad Bayasi et.al 1992). There are two important fibre parameters that strongly influence the degree of damage to concrete workability caused by fibres. It is already mentioned in the introduction chapter, slump test is not sufficient for measuring the workability of fibre reinforced concrete. So, both slum test and vee be tests are planned.

3.4.1.1 Slump Test

The slump test was conducted as per IS: 7320 – 1974 The slump was measured in mm.

3.4.1.2 Vee-Bee Test

By using this method, consistency is being found by determining the time required for transforming b vibration, a concrete specimen in the shape of a conical frustum into a cylinder. The test was conducted as per Indian Standards.

3.4.1.3 Air Content

The exact air content in concrete is extremely important as it affects the various parameters of concrete. If the amount of air in a mix differs widely from the design value, the properties of the concrete may be seriously affected. Too little air results in insufficient workability and too much air will result in low strength. There are three methods for measuring air content of fresh concrete namely Gravimetric meter method, volumetric method and pressure method. Out of which pressure method is perhaps the best method for finding the air content of fresh concrete because of its superiority and ease of operation.

The water meter type has been used in this investigation. The vessel was filled with concrete, compacted in a standard manner and struck off level. A cover was then clamped in position. Water was added until the level eas reached ‘0’ mark on the tube of the cover and then pressure is applied by means of a bicycle pump. The pressure was transmitted to the air entrained in the concrete, which contracts accordingly. The water level has falled. The pressure has been incresed to a predetermined value as indicated by a small pressure gauge mounted on the cover. The glass gauge tube was so calibrated that the percentage of air by volume was indicated directly.

3.4.2 Mechanical Strength Studies

Plain concrete possesses very low tensile strength, limited ductility and little resistance to cracking. The presence of micro cracks is responsible for weakness of plain concrete. The strength characteristics and economic advantages of fibre reinforced concrete are better compared to conventional concrete. To study the fundamental strength characteristics, the following studies have been carried out.

3.4.2.1 Compressive Strength

According to Indian Standard specifications (IS: 516 – 1959), the compression test on cubes and cylinders were conducted.

3.4.2.2 Modulus of Elasticity

The modulus of elasticity of concrete is one of the most important mechanical properties of concrete since it impacts the serviceability and the structural performance of reinforced concrete structures. The closest approximation to the theoretical modulus of elasticity derived from a truly elastic response is initial tangent modulus. But it is not always easily determined from a compession method. In such a case chord modulus of elasticity is being used. The current method to determine the chord modulus of elasticity of concrete is Compressometer method. California test 522 (2000) procedure was followed in this study to eveluate the chord modulus of elasticity. Cylinderical specimens of size 100 mm dia and 300 mm length were casted and cured.

The load was applied continuously without shock. Without interruption, applied loading and longitudinal strain at pre-designated intervals were taken. The reading interval was fixed as 2kN to permit plotting stress-strain curve if desired. Along the above set of readings, the following two readings were also monitored and noted. These are

i) The applied load when the longitidinal strain is 50×10^{-6} m/m

ii) The longitudinal strain when the applied load is equal to 40 percent of the ultimate.

Here, longitudinal strain was defined as the total longitudinal deformation divided by the effective gauge length. The chord modulus of elasticity was obtained using the formula

$$E = \frac{(S_2 - S_1)}{(C - 0.00005)} \qquad (3.2)$$

Where,

E – Chord modulus of elasticity in Mpa

S_2 – Stress corresponding to 40% of ultimate load

S_1 – Stress corresponding to a longitudinal strain of 50×10^{-6} m/m

C – Longitudinal strain produced by stress S_2

3.4.2.2 Split Tensile Strength

Direct measurement of tensile strength of concrete is difficult. One of the indirect tension test methods is Split tension test. The Split tensile strength test was carried out on the compression testing machine. The casting and testing of the specimens were done as per IS 5816: 1999.

3.4.2.3 Modulus of Rupture

The extreme fibre stress calculated at the failure of specimen is called Modulus of rupture. It is also an indirect measure to predict the tensile strength of concrete. Flexural strength test was conducted as per recommendations IS: 516 – 1959. In flexural strength test, beams of size 10 x 10 x 50 cm were casted.

3.4.2.5 Shear Strength

So far mechanical strength properties are representing the compressive and tensile strength of concrete only. No much work on shear strength of concrete was reported by researchers. But fibre reinforced concrete possesses significant improvement in shear strength (Baruah.P and Talukdar 2007). Bairagi N.K et.al (2001) proposed a method to determine the shear strength of fibre reinforced concrete.

Based on the literature, L-shaped shear test specimens were prepared from 150 mm cubes by inserting a wooden block of 90 mm x 60 mm in cross section and 150 mm high into the cube moulds before casting of concrete. The details of the shear test specimen are shown in Fig.3.24. All the test specimens were casted and cured for 28 days. The loading arrangement for shear test is shown in Fig.3.25. The speciemns were placed as shown in Fig.3.26 on compression testing machine. A 150 x 85 x 10 mm size MS plate was placed on 90 mm face of leftside portion. Mild steel bar of 12 mm diameter was placed over the centre of the plate. Another 22 mm diameter MS bar was placed at the edge of the plate. Ovar these bars, another

MS plate of size 150 x 110 x 10 mm was placed. Load was applied on the top plate which forms shear plane below the centre of 22 mm diameter bar. The loading was continued until the specimen failure. The shear strength was obtained using

$$f_s = \frac{P}{A}$$

Where,

f_s – Shear strength

P – Applied compression load

A – Shearing area

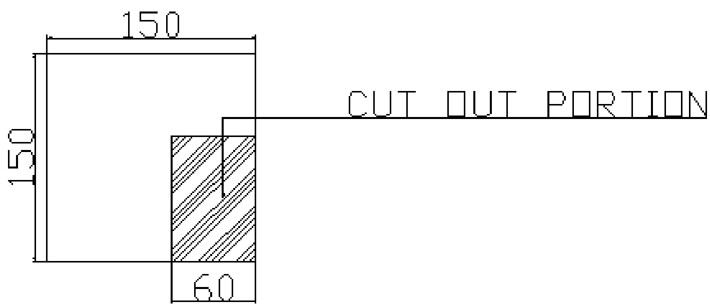

Fig. 3.24 Details of 'L' Shaped Specimen

Fig3.25 Loading Arrangement of Shear Test

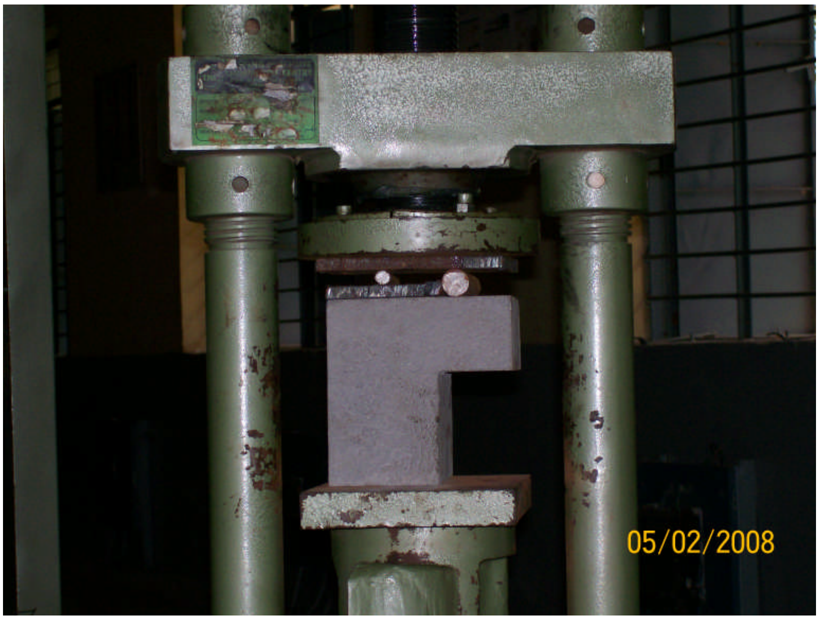

Fig.3.26 Shear Test on 'L'Specimen

3.4.3 Structural Behaviour Studies

This section provides the details about experimental methodology, test set up, detailing of specimens which were casted to study the structural properties such as ductility, energy absorption of beam and beam-column joint specimens under static and cyclic loading. Based on the above mentioned elaborative mechanical strength studies, two volume fractions such as 0.5% and 1.0% of all the fibres were selected and further used in this section.

3.4.3.1Static Flexural Behaviour of Beams

The benefit of addition of discrete fibres on the flexural strength of reinforced concrete beams has been investigated in this section. Based on the experimental investigations, load vs midspan deflection and moment vs rotation curves have been drawn. Seismic critical parameters such as ductiltiy, energy absorption capacity and stiffness degradation at three salient points such as first cracking, first yielding and ultimate points. This section is giving the specimen details including reinforcement detailing; loading sequence and experimental set up are discussed.

3.4.3.1.1 Specimen Details

This experimental programme consists of casting and testing of fifteen numbars of 2 m long reinforced concrete beams. All the beams were tested over a simply supported span of 1.8 m. The beam was designed as under reinforced section to sustain a minimum ultimate load of 25 kN. The details of reinforcements present in the test beam are shown in Fig.3.28. The beam consists of three 8 mm diameter bars at bottom as tensile reinforcement. Another two numbers of 8 mm diameter bars were placed at top. To hold the reinforcements and to act as shear reinforcements, 6 mm diameter stirrups were used at 100 mm center to center. HYSD Fe 415 steel was used as reinforcements.

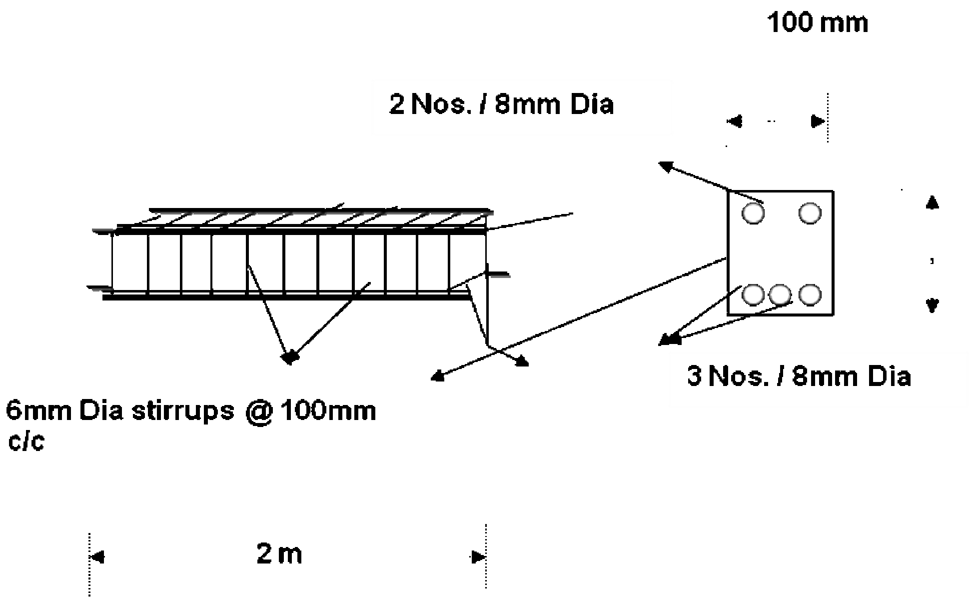

Fig.3.28 Reinforcement Detailing of Beam

Totally 15 reinforced concrete beams were casted and tested. For each fibre of two volume fractions such as 0.5% and 1.0%, 14 fibre reinforced concrete beams and one controlled beam have been casted. The designations for each specimen have been given as

CNC – Conventional Concrete Beam

SFRC ½ - Steel Fibre Reinforced Concrete with 0.5%

NFRC½ - Nylon Fibre Reinforced Concrete with 0.5%

PFRC ½ - Plastic Fibre Reinforced Concrete with 0.5%

TFRC ½ - Tyre Fibre Reinforced Concrete with 0.5%

CFRC ½ - Coir Fibre Reinforced Concrete with 0.5%

SCFRC½ - Sugarcane Fibre Reinforced Concrete with 0.5%

RFRC ½ - Rice Husk Fibre Reinforced Concrete with 0.5%

SFRC1- Steel Fibre Reinforced Concrete with 1.0%

NFRC1 - Nylon Fibre Reinforced Concrete with 1.0%

PFRC 1 - Plastic Fibre Reinforced Concrete with 1.0%

TFRC 1 - Tyre Fibre Reinforced Concrete with 1.0%

CFRC 1 - Coir Fibre Reinforced Concrete with 1.0%

SCFRC1 - Sugarcane Fibre Reinforced Concrete with 1.0%

RFRC 1 - Rice Husk Fibre Reinforced Concrete with 1.0%

3.4.3.1.2 Specimens Casting

All the beams specimens were cast in steel moulds. Laboratory type mixer machine was used to mix the ingredients of concrete. To avoid balling effect of fibers, the following procedure was followed in casting. First, aggregates and cement were put and allowed to mix for one minute and water was added within two minutes. Then, fibers were uniformly dispersed throughout the mass with slow increment. Finally, concrete was allowed to mix for three minutes. All the specimens were well compacted using table vibrator. The specimens were demolded after one day and then placed in a curing tank for 28 days of curing. For 12 hours prior to the testing, the specimens were allowed to air dry in laboratory.

3.4.3.1.3 Test Set Up

Tests were carried out at room temperature and as per the Indian standards. Structural properties are ascertained by conducting middle third loading test. The testing arrangement is shown in Fig.3.29. Four point bending was applied on reinforced concrete beams of beam span 1.8 m through hydraulic jack of capacity 100kN. The specimens were placed on a simply supported arrangement of 100 T Loading frame. The beams were suitably instrumented for measuring deflections at several locations including the midspan deflection with dial gauges and LVDTs. To avoid the excessive deformation at the support locations, additional dial gauges were placed at the top and bottom faces of ends. DEMEC (Demoundable mechanical strain gauge) was used to measure the concrete strain readings at top as well as the bottom fibre on mid section of the beam.

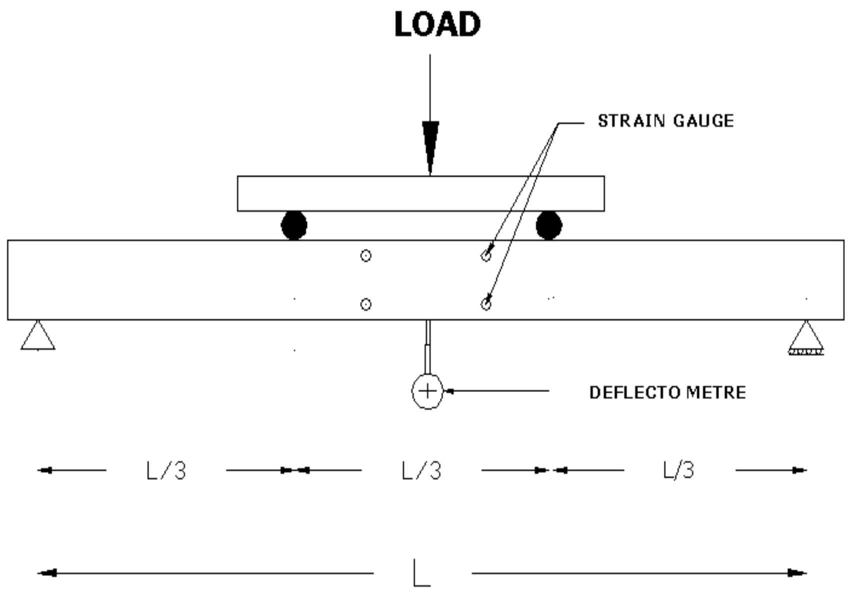

Fig.3.29 Loading Arrangement of Flexural Test

Fig.3.30 Experimental Set Up

First yielding of steel is very much important in structural characterization particularly in ductiltiy calculation. To read the strain values of main steel reinforcement, electrical strain gauges of 15 mm gauge length having a resistance of 118 – 124 Ω and a gauge factor of 2.14 were pasted on the bottom main reinforcements of the beam. The photograph of steel reinfrocement with electrical strain gauge is shown in Fig.3.31.

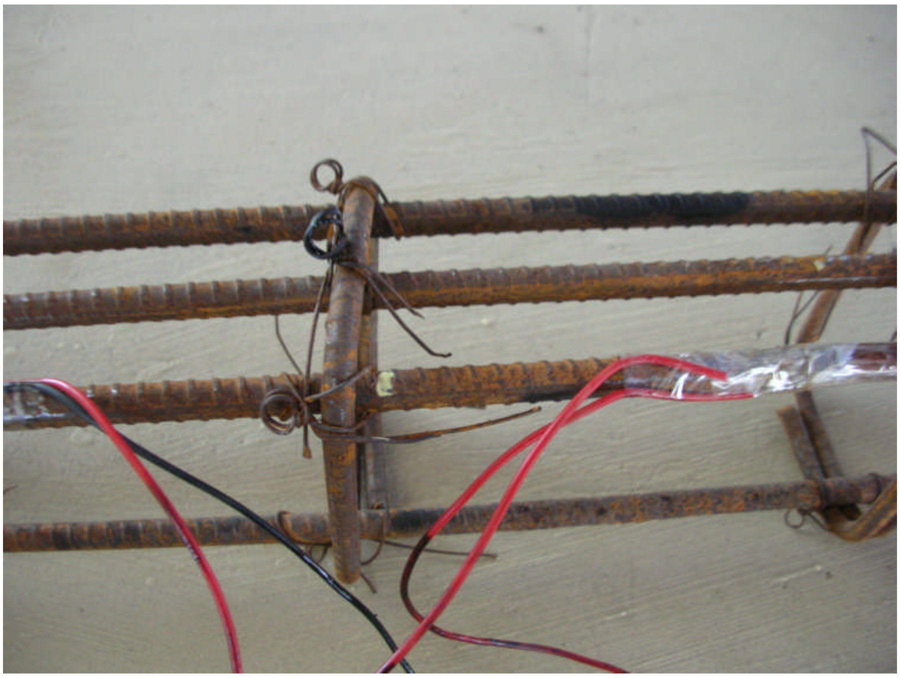

Fig.3.31 Reinforcements with Electrical Strain Gauges

Load was applied through a hydraulic jack of capacity 100 kN. For every 2.5 kN loading interval, the corresponding mid span as well as strain at mid section readings weer taken. Simultaneously, the cracking behaviour on the faces for full length of the beam was also observed caefully. The first craking was noted for all the beams and corresponding load, deflection and strain valued are reported.

3.4.3.2 Cyclic Flexural Behaviour of Beams

The specimens details of reinforced concrete beams, casting procedure, and test set up and measuring instrumentations followed in static loading tests are continued in cyclic test also. Due to the lagging in experimental facilities, the cyclic load was not given by push pull jacks. As like in static loading all the arrangements have been made. Instead of applying cyclic loading through jacks, the reinforced concrete beams were turned off for each cycle. Special care has been taken to locate the beams in exact position of supports and loading points for each cycles. To simulate seismic loading, cyclic loading with set up of 2.5 kN, 5 kN etc., was applied. Fig.3.32 shows the loading sequence of cyclic loading.

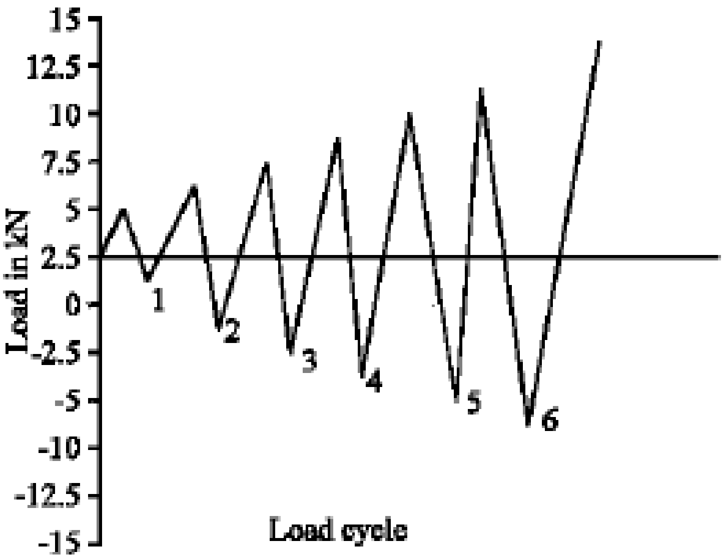

Fig.3.32 Loading Sequence of Cyclic Loading

3.4.3.3 Behaviour of Beam-Column Joints

Experiments were conducted on beam – column joints under cyclic loading using different fibres and compared with the conventionally reinforced concrete specimens. The performance was evaluated with respect to strength, ductility and energy absorption characteristics

3.4.3.3.1 Specimen Details

A total of four exterior beam-column joints were cast and tested under flexural cyclic loading in this investigation. The specimen details and testing methodology used in that study were based on the past literature (Ganesan 2000). The geometry and reinforcement detailing of the specimens are shown in Fig.3.33. The columns were reinforced with six 12 mm diameter high yield strength (HYSD) bars and the beam was provided with an equal reinforcement of two 12 mm diameter HYSD bars at top and bottom. HYSD bars of 6 mm diameter were used as transverse reinforcement in both the columns and beams. Since more volume fraction of fibres than 1% does not yield good workability as well as leads to strength reduction, it was restricted to 1%.

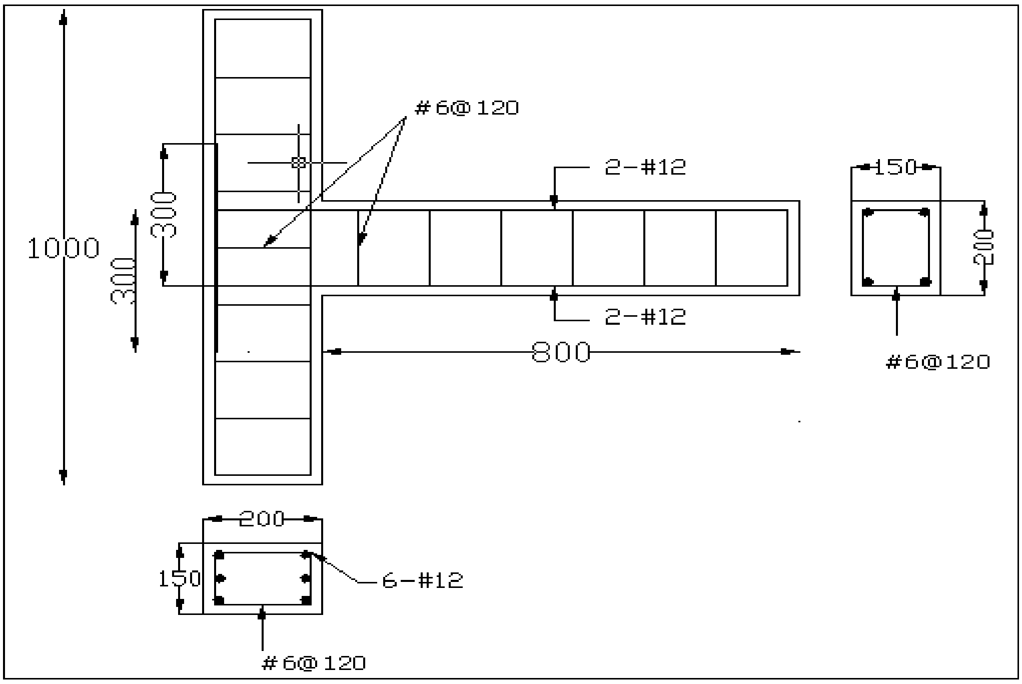

Fig.3.33 Reinforcement Detailing of Beam-Column Joint

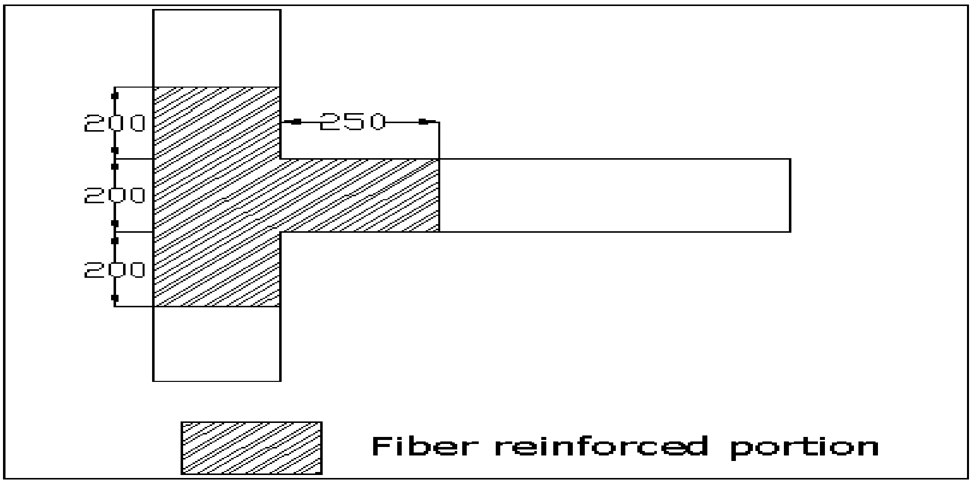

Fig.3.34 Fibre Portion in Beam – Column Joint

3.4.3.3.2 Testing of Specimens

Tests were carried out at room temperature in a universal testing machine (UTM) of 100 t capacity. As shown in Fig.3.35, to simulate the column axial load as well as to maintain the position, vertically placed specimens were subjected to a constant axial load of 75 kN which consists of 20 percent of the axial load capacity of the column. A dial gauge was provided at the free end of the beam to measure deflection and DEMEC gauge pellets were pasted at the top and bottom of the junction as well as at centre of the column to record the strains. The beam was loaded at free end through a hydraulic jack and the corresponding deflection and strain readings were taken. To read the intensity of loading precisely, load cell with a least count of 0.01 kN was used. The sequence of cyclic loading is also shown in Fig.3.36. The increment of loading selected was 4 kN. Load-Deflection and Moment-Curvature curves were plotted based on the experimental readings

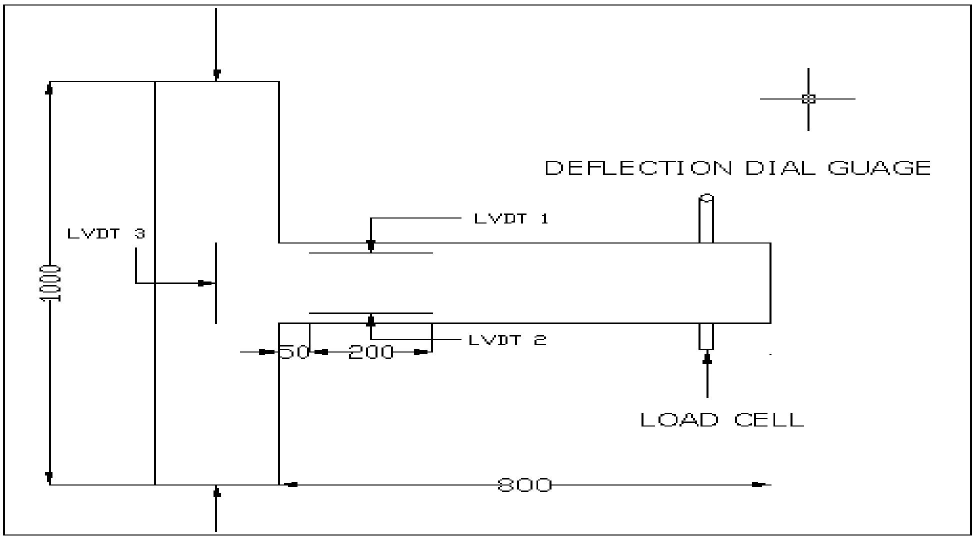

Fig.3.35 Loading Arrangement of Beam – Column Joint

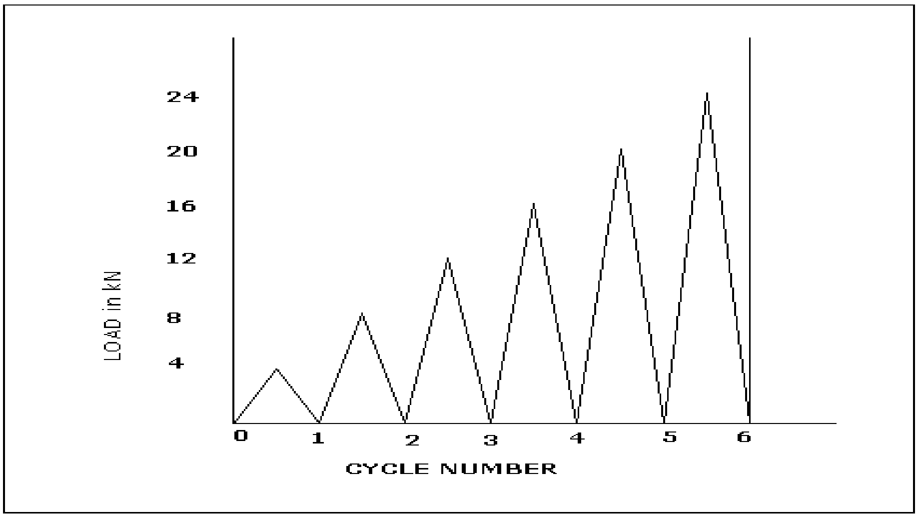

Fig.3.36 Loading Sequence of Beam – Column Joint Test

Fig.3.37 Reinforcement Detailing of Beam – Column Joint

Fig.3.38 Casting of Beam – Column Joint

Fig.3.39 Testing of Beam – Column Joint

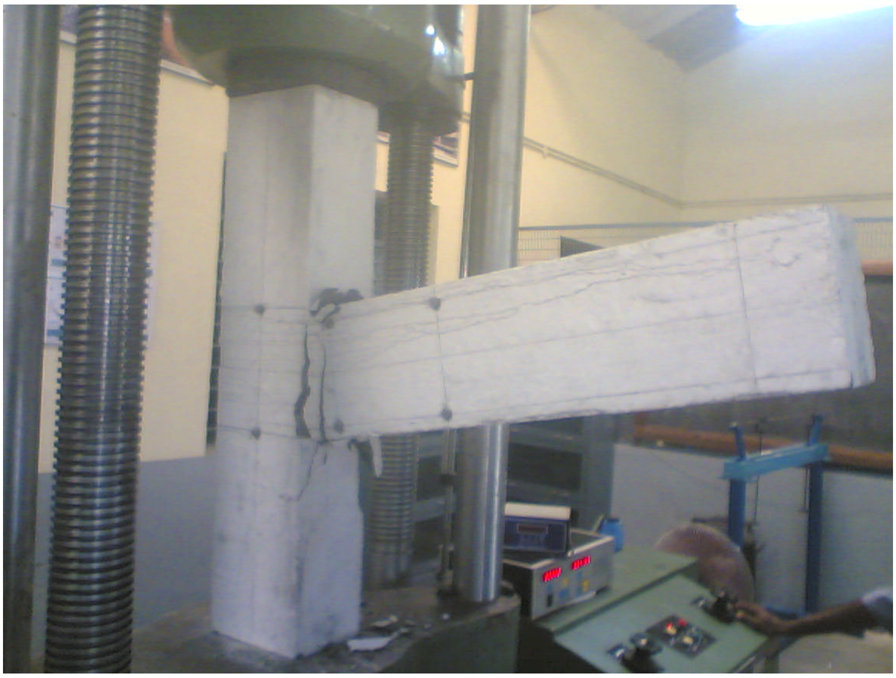

Fig.3.40 Final Shape of Beam – Column Joint

CHAPTER 4

ANALYTICAL INVESTIGATION

4.1 INTRODUCTION

Since the focus of the present study is based on the locally available waste materials, validation of experimental results through some means will give more confident to future researchers. For this purpose, a three dimensional non-linear finite element modeling and analysis has been carried out for different concrete beams using commercial software ANSYS 10. The fraction of the entire volume of the fibre present along the longitudinal axis of the reinforced concrete beams alone has been modeled explicitly as it is expected that these fibres would contribute to the mobilization of forces required to ustain the applied loads across the nterfaces through their bridging action (Padmarajaiah S.K and Ananth Ramaswamy 2002). The load – displacement curve obtained for different type of concrete beams from ANSYS 10 are compared with experimental curves.

One of the important advantages of using fibres in seismic design of reinforced concrete beams is to limit the damage at critical locations of the structures by improving their energy absorption and ductility characteristics. Experimental results of cyclic loading tests have repoted the enhancement in seismic parameters. To facilitate the quantification of damage in analysis and design, a numerical damage is a valuable tool. Two well known damage index models have been selected for the computation of damage index, namely the one proposed by Kratzig et.al (1989) and the simplified damage model proposed by Sreekala S et.al (2007). Based on the static and cyclic experimental results, damage indices have been computed and compared.

4.2 Finite Element Modelling Using ANSYS 10

The step-by-step procedure of FE analysis has been briefly described here.

1. The structure has been descretized into finite elements. The geometry, loading and boundary conditions have been defined. The concrete has been modeled using SOLID65 elements; the steel reinforcements (fibers / prestressing steel (S) / high strength deformed bars (HYSD) have been modeled using LINK8 elements. SOLID65, LINK8 are the elements available in the ANSYS 10 element library.
2. The number of load steps and sub-steps required has been specified to capture the nonlinearity effectively. In ANSYS 10 program, the load steps and sub-steps is specified with a 'time' parameter established at the end of each step. At each load increment, the stiffness matrix will get updated in the Newton Raphson iteration procedure in the ANSYS.
3. The program continues the iteration process until the convergence criteria are satisfied or the maximum number of iteration specified for the sub-step is reached. If the convergence criteria are satisfied, the next increment of load is applied and the procedure is repeated. Otherwise, the unbalanced load is applied back to the structure and the iterations continued. If the maximum iterations are exceeded, the program terminates.

The FE model used in this finite element analysis is shown in Fig.4.3.1. The full length beam has been modeled as SOLID 65 element. The mesh consisted of 20 concrete elements along the longitudinal axis, 4 elements across the width and 6 element along the depth. Studies by Bazzant and Oh (1983) and Bazant and Cedolin (1993) indicated that the smallest element dimension in FE analysis is fixed based on the size of coarse aggregate used. So, the mesh size has been fixed as 25mm which is greater than the maximum size of coarse aggregate used in the experimental investigation. The test results of cylinder compressive strength and split tensile strength based on the companion specimens cast and tested along the reinforced concrete beams were used as material properties of concrete in 'ANSYS'.

To obtain the crack pattern of the beams at the end, shear transfer co-efficients in opening is assumed as 0.2 and in closing is assumed as 0.9. The remainig parameters such as Hydrostatic stress state, Bi-axial compressive stress, Uni-axial tensile stress and Tensile cracking fracture are assumed to be the default values have been set so as to represent William and Warnke model (1975). In ANSYS, smeared representation of crack is used in ‘SOLID 65’. Before cracking concrete, concrete is assumed to an elastic isotropic material. After cracking, concrete is assumed to be orthotropic having stiffness based on a bi-linear softening stress-strain response. The steel reinforcements such as stirrups, deformed main bars and hanger bars have been modeled as ‘LINK 8’three dimensional truss element. Addition of fibres in concrete increases the post-cracking stiffness of the concrete through the bridging of fibres across the crack interfaces. This induces the resistance to the ctack opening when the concrete strain is greater than six times the ultimate strain. For that fibres have been modeled discretely.

The effectiveness of fibres in increasing the tensile strength of the concrete, at least partially depends on the number of fibres per cross sectional area of concrete (Padmarajaiah S.K and Ananth Ramaswamy 2002). The fraction of the entire volume of the fibre present along the longitudinal axis alone was modeled explicity as it was expected to contribute to the mobilization of forces required to sustain the applied loads after concrete cracking and provide resistance to crack propogation. Another important point is during the casting of fibre reinforced concrete , due to the vibration process fibres in concrete tend to settle down and reorient in horizontal planes. Hence the area of discrete reinforcement representing the fibre (A_f) is arrived based on the equation

$$A_f = \alpha' V_f A_{ct} \qquad (4.32)$$

Where, α' is the Orientation factor which is assumed as 0.64 for 2D and 3D orientation of fibres in beam (Souroushian and Lee 1990). A_{ct} is the tributary area of the concrete over which the fibre present. The tributary area concept is illustrated in Fig.4.3.2. The elements lying in the boundaries carry half of the area of fibres used in the interior of the FE mesh. Rate independent multi-linear isotropic hardening option with Von-Mises yield creterion has been used to define the material property of fibres. The elasto-plastic material behaviour of the fibres was given as input into ‘ANSYS’.

4.3 Evaluation of Damage Indices

Damage indices are the valuable tools which provide a means by which different design or retrofit options can be compared objectively. Among the different models proposed by earlier researchers, the followint two has been selcted in this study.

Kratzig et.al (1989) proposed a model based on the hysteretic energy absorbed by a member. The first load cycle at a given amplitude is termed as primary half cycle. The subsequent cycle is termed as followers. The overall damage index has given as

$$D = D^{+} + D^{-} - D^{+}D^{-} \qquad (4.31)$$

Where,

$$D^{+} = \frac{\Sigma E^{+}_{p,i} + \Sigma^{+}_{i}}{\Sigma^{+}_{f} + \Sigma^{+}_{i}} \qquad (4.32)$$

$\Sigma E^{+}_{p,i}$ - Energy absorbed in a primary half cycle

Σ^{+}_{I} - Energy of a Follower half cycle

Σ^{+}_{f} - Energy absorbed in monotonic test test to failure Similar equation can be used for negative cycle also.

Sreekala R et.al (2006) proposed a simplified damage index model based on an equivalent normalized ductiltiy factor '(R/R_f)'. Some curves have been proposed between (R/R_f) ratio and Damage index 'D' for different shear span to depth ratios (a/d).

The damage index can be calculated by using the relation,

$$D = (R/R_f) \qquad (4.33)$$

Where,

$$R = \frac{\mu_{cy-\delta}}{\mu_{st}} \qquad (4.34)$$

$$R_f = \frac{\mu_{cy-\delta f}}{\mu_{st}} \qquad (4.35)$$

$\mu_{cy-\delta}$ = Cyclic Displacement Ratio in multiples of yield displacement

μ_{st} = Monotonic Ductility

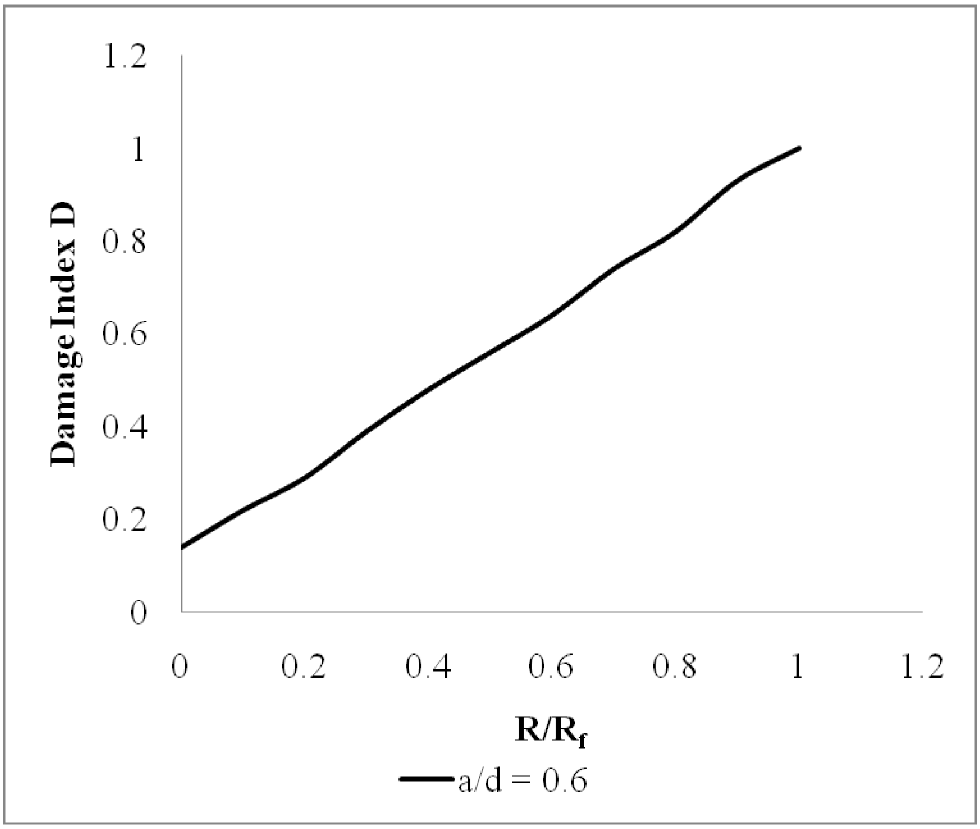

Fig.4.3.3 Proposed Curve for Damage Index (Sreekala R et.al)

The value of (R/R_f) for conventional and fibre reinforced concrete beams have been calculated from the experimental hysteretic curves. For the corresponding (R/R_f) value, Damage indices have been obtained from the Fig.4.3.3. This will give the clear indication about the increment or reduction in damage of structural element under unanticipated loading conditions.

CHAPTER 5

RESULTS AND DISCUSSIONS

5.1 INTRODUCTION

The results obtained from the above elaborative experimental as well as analytical investigations are discussed in this chapter. Initially the workability properties such as slump value, vee-bee time and air content percentage have been discussed. Then the results of mechanical strength properties such as cube compressive strength, cylinder compressive strength, split tensile strength, modulus of rupture, modulus of elasticity, shear strength and impact energy have discussed. Comparision of experimental and predicted values of proposed expressions has been made for all the fibre concrete mixes. Even the natural fibres have given comparable strength properties, durability of such natural fibres are questionable. So the natural fibres have been allowed to react with concrete for about two years under alternative wetting and drying conditions. After two years they have been taken from the concrete and subjected to microstructural SEM study. The results obtained from the investigations are discussed. Next the structural properties of conventionally reinforced and fibre reinforced concrete beams under static loading are discussed. The critical seismic parameters such as ductility, energy absorption capacity and stiffness degradation along both system and section are discussed. The experimental results are validated analytically by two means. One of the ways is validationg the ultimate moment obtained from the moment - rotation curves using a mechanical model which was proposed by earlier researchers. The another one is a finite element analysis which has been conducted using ANSYS software. Apart from these analyses, a inverse type finite element analyses has been carried out using MATLAB and the results are discussed in the subsequent heading. Then the damage indices based on the expressions proposed by earlier researchers for different fibre reinforced concrete beams have been evaluated from the experimental results and discussed. Finally the results obtained from the beam – column joint test are discussed.

5.2 Workability Properties

Fresh mix characteristics are more emphasized in fibre concrete compared to the plain concrete. There are two parameters that strongly influence the degree of damage to concrete workability caused by fibres. These are fibre volume fraction and aspect ratio. Generally increasing volume fraction and/or increasing aspect ratio of fibres results in further reduction of fresh concrete workability. Considering the similar effects of fibre volume fraction and aspect ratio on freshly mixed fibre concretes, it was proposed that the multiplication of these two parameters v_f and (l/d) provides a single variable representing the effect of certain fibre reinforcement condition on specific concrete mix.

In this study, fibres such as steel, nylon, plastic, tyre, coir, sugarcane and rice husk of three different volume fractions like 0.5%, 1.0% and 1.5% were taken. Regarding length fo fibres, except rice husk (since it has its own length) the remainig fibres of three different lengths like 30mm, 60mm and 90mm were chosen. Totally 58 different mixes were prepared and tested.

5.2.1 Slump Test

The decrease in the height of slumped cone is called 'slump of concrete'. Fig. 5.1 and 5.2 show the measured values of slump value of fresh fibrous mixes versus their fibre reinforcement index (F) of inorganic fibres such as steel, nylon, plastic, tyre and organic fibres such as coir, sugarane and rice husk respectively. Table 5.1 shows the test results of slump test. It is obvious in all the figures that fresh concrete mix workability is damaged by increasing the fibre reinforcing index. The rate of drop in workability with increase of fibre reinforcing index seems to be comparable.

Table 5.1 Slump Test Results

Mix	Fibre Reinforcing Index						
	0.0	**0.15**	**0.3**	**0.45**	**0.6**	**0.9**	**1.35**
SFC	120	96	90	78	65	52	45
NFC	120	108	96	84	71	68	62
PFC	120	100	94	80	69	58	50

TFC	120	95	82	68	53	42	35
CFC	120	96	85	70	59	52	42
SCFC	120	92	80	64	50	42	36
RFC	120	95	82	74	-	-	-

At specific fibre reinforing index value, tyre fibre seems to give higher slump value. Next to tyre, sugarcane, plastic, steel and coir have given the higher slumps respectively. Due to higher surface area, tyre and plastic fibres possessed less workability in incresed fibre reinforcing index values. Also because of water absorption capabilty, organic natural fibres have given the lesser workability. Though rice husk is a organic fibre, its less specific area did not affect the workability significantly while comparing with others. Among all the fibres, nylon possesed higher workability by having less water absorption capability as well as less surface area.

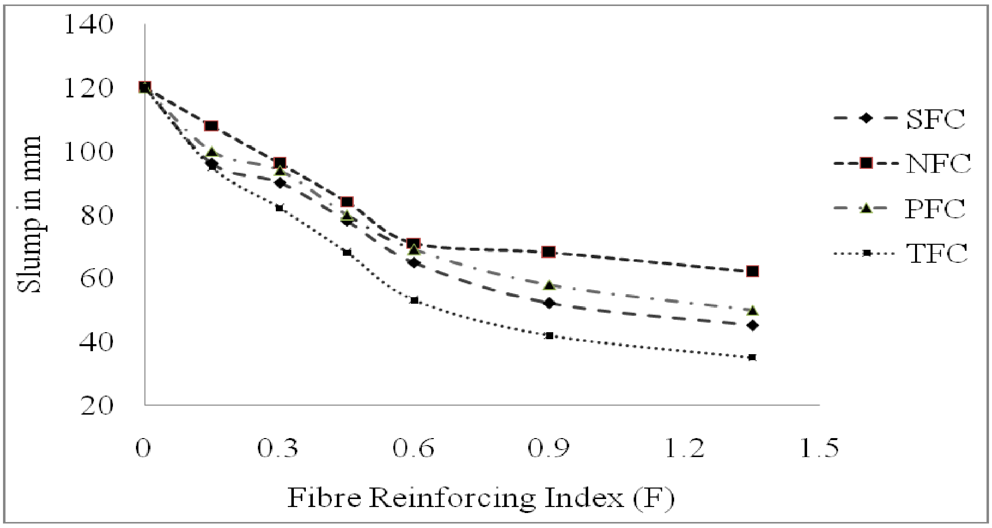

Fig 5.1 Comparision of FRI and Slump of Inorganic Fibres

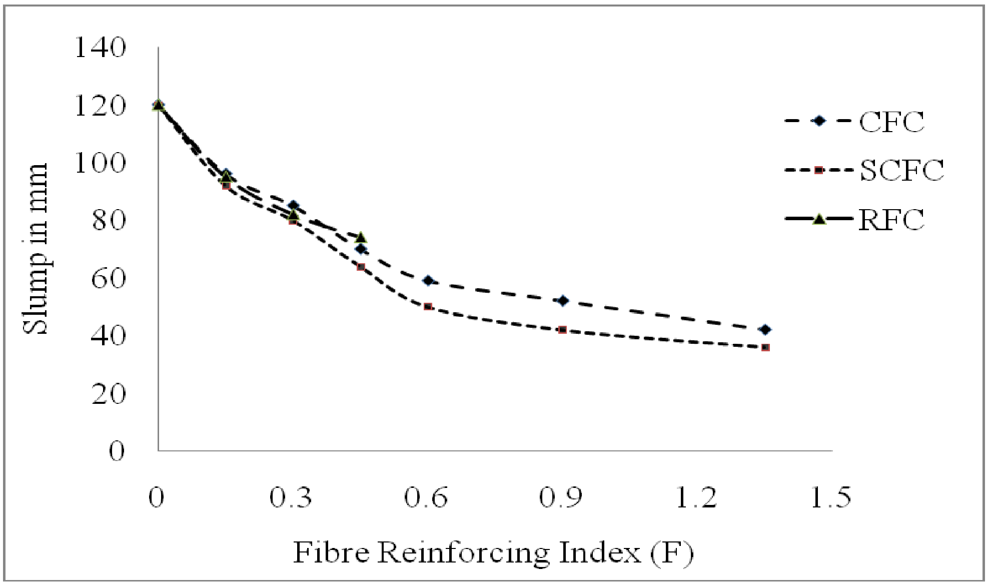

Fig 5.2 Comparision of FRI Index and Slump of Organic Fibres

5.2.2 Vee-Bee Test

Table 5.2 and Fig. 5.3 – 5.4 have given the vee-bee test results. Fig. 5.3 shows the variation of vee-bee seconds with fibre reinforcing index for inorganic fibres and Fig 5.4 shows for organic fibres. From Fig. 5.4, it is observed that increase in fibre reinforcing index increses the vee bee time for all the types of mixes. Tyre fibre mix possess the highest vee-bee seconds value whih denotes it will create more damage in workability of concrete.

Table 5.2 Vee-Bee Test Results

Mix	Fibre Reinforcing Index						
	0.0	0.15	0.3	0.45	0.6	0.9	1.35
SFC	1.50	2.25	3.00	3.50	3.75	4.25	5.00
NFC	1.50	1.75	2.50	2.75	3.50	3.75	4.25
PFC	1.50	2.00	2.75	3.25	3.75	4.50	5.25
TFC	1.50	2.25	3.00	3.25	3.75	4.75	6.25
CFC	1.50	2.00	2.50	3.25	3.75	4.25	5.00
SCFC	1.50	2.25	3.00	3.50	4.00	4.75	5.25

RFC	1.50	1.75	2.00	2.25	3.25	3.75	4.50

Vee bee seconds for steel fibre concrete stated from 1.5 seconds for zero fibre reinforcing index to 4.5 seconds for fibre reinforcing index 1.35. Nylon fibre concrete possesses lowest vee bee seconds values while comparing with other fibres. As like in slump test, here also tyre fibre concrete yields more vee bee seconds. Though it has lesser value at lower fibre reinforcing index values, it rearches the highest at higher fibre reinforcing indices. The remaining inorganic fibre called plastic fibre shows the intermideate value of vee bee seconds.

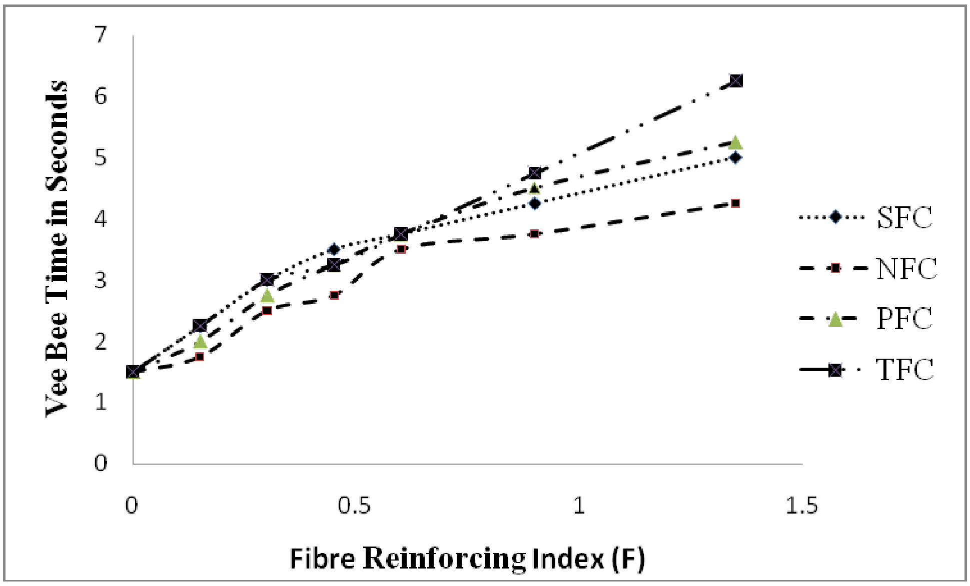

Fig 5.3 Comparision of FRI and Vee-Bee sec of Inorganic Fibres

While considering the organic fibres, rice husk did not give very big damage to vee seconds deviation. But sugarcane starts from 1.50 seconds for zero fibre reinforcing index and ends at 5.25 seconds for fibre reinforcing index of 1.35. Coir fibre shows the intermediate value of vee bee seconds. So from the vee bee test results also it is proved that fibres having large surface area and higher water absorption capacity tends the concrete to less workable, where as fibres like nylon having less surface area and water absorbing capability did not affect much than others. Anyway it is also observed that addition of any fibre damages the workability from 150% to 250%.

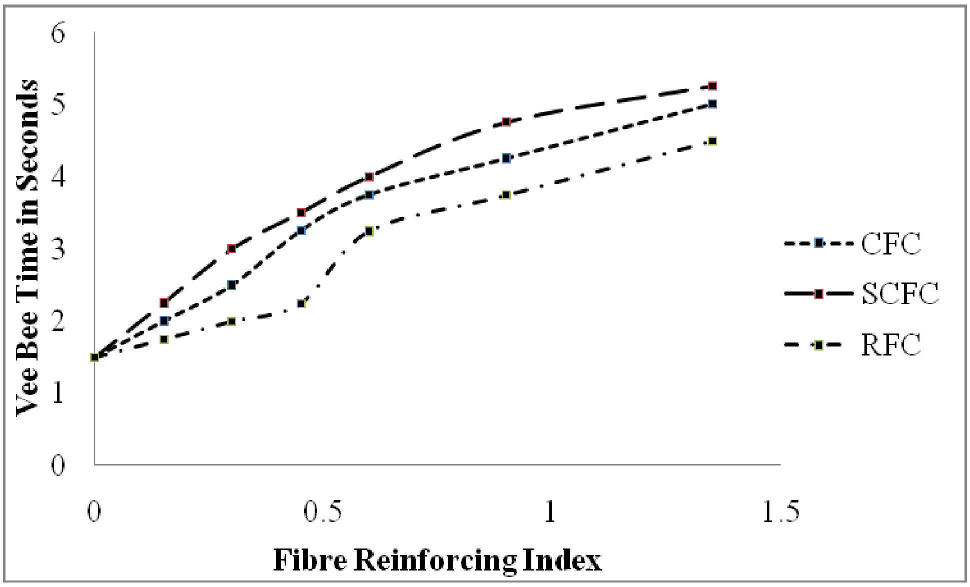

Fig 5.4 Comparision of FRI and Vee-Bee sec of Organic Fibres

5.2.3 Air Content

The experimental results of pressure method which was conducted to measure the air content are shown in Fig.5.5 – 5.4. There is no any direct relation for air content with fibre reinforcing indeices. So, in this study comparisions were made with volume fractions and length of fibres. From the Figures, it can be known that increase in volume fraction of any fibre increases the air content values. But length of fibre concrened, decrese in length of fibres in thinner fibres increases the air content where as in thicker fibres increase in length of fibres increases the percentage of air content.

Among the seven fibres used, steel, nylon, coir and sugarcane are compritively thinner than the remaining fibres such as plastic, tyre and sugarcane. For constant volume fraction, decreasing the length of thinner fibres increases the air content. This may be due to the increase in number of fibres per unit volume of concrete area. Air content of conventional concrete was measured as 2%. This is increased upto 4.8% for steel fibres of 30mm length and 1.5% volume fraction. For nylon fibres, air content increses upto 4.6%, for coir fibres upto 5.2%, and for rice husk upto 4.6%.

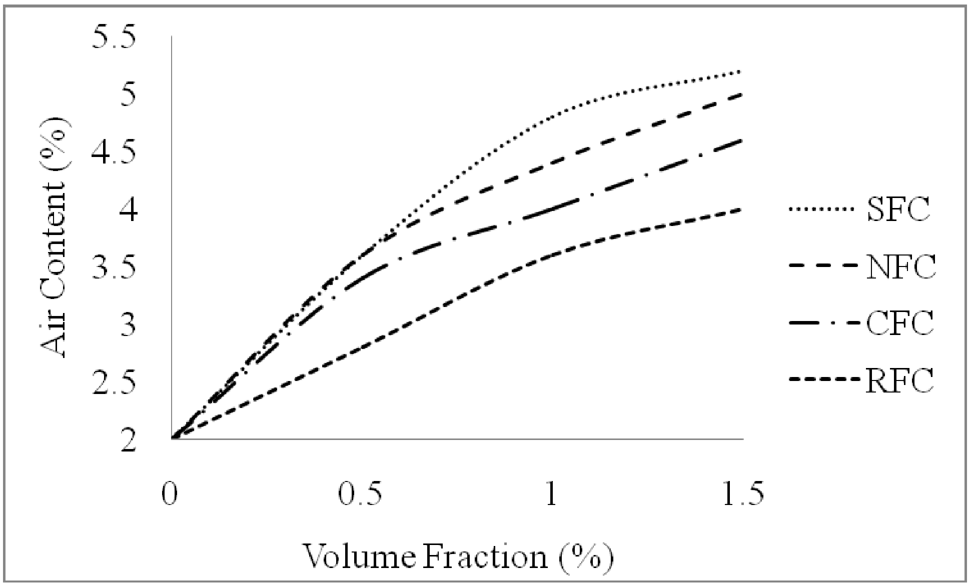

Fig 5.5 Comparision Between Volume Fraction of 30 mm long Thin Fibres and Air Content

Similarly in thick fibres also volume of fibres in concrete increases the air content of the concrete. Though at the lower volume fractions, tyre and sugarcene fibres influenced the air content comparitively lightly, at higher volume fraction it exceeds the air conten of plastic fibre concrete. The air content of fibre concrete deviates from 50% to 140% from conventional concrete air content. Similary for 60mm length and 90 mm length fibres, effect of volume fraction of fibres on air content percentage are shown in Fig. 5.7 to 5.10. In all the figures, at any particular length of fibre, volume fraction increases the air content significantly.

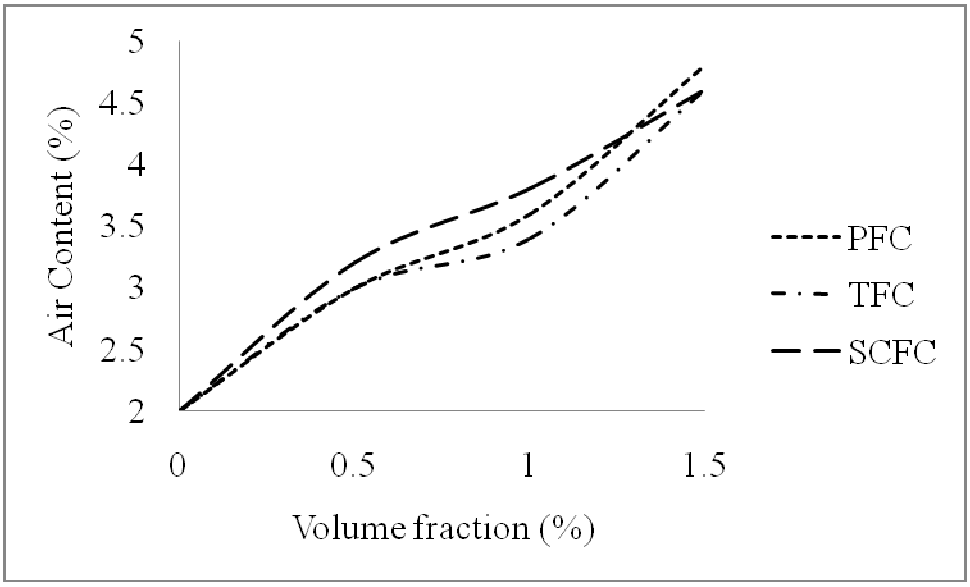

Fig 5.6 Comparision between Volume Fraction of 30 mm long Thick Fibres and Air Content

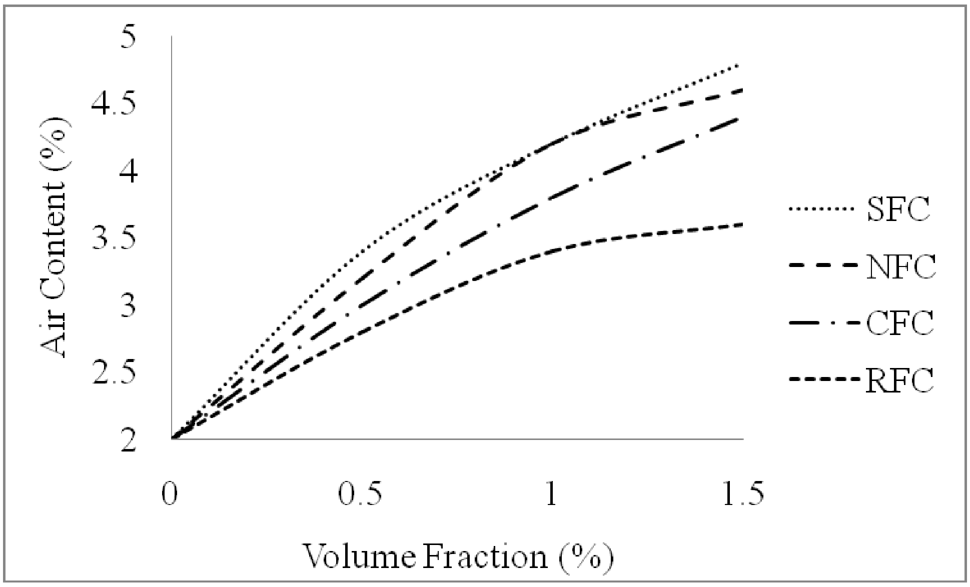

Fig 5.7 Comparision between Volume Fraction of 60mm long Thin Fibres and Air Content

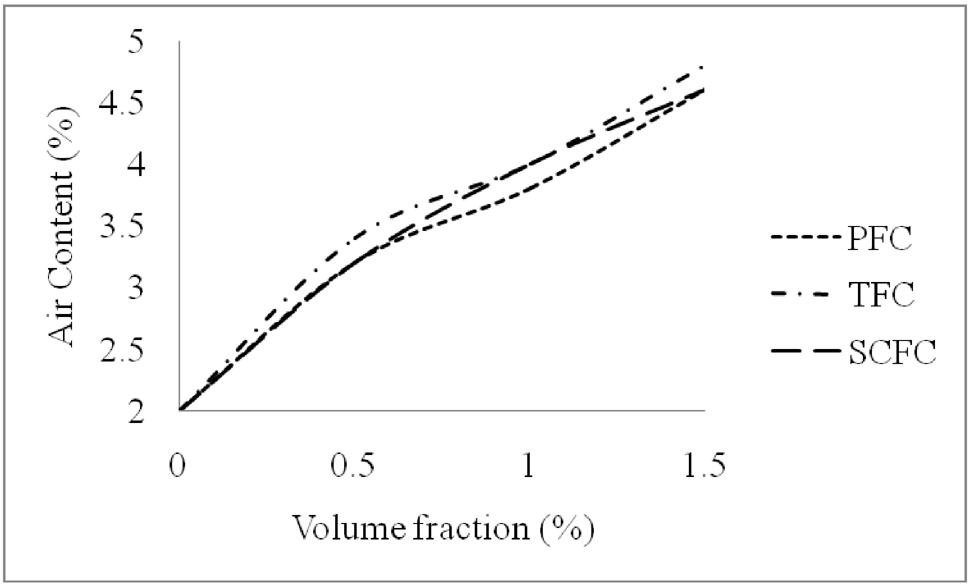

Fig 5.8 Comparision between Volume Fraction of 60mm long Thick Fibres and Air Content

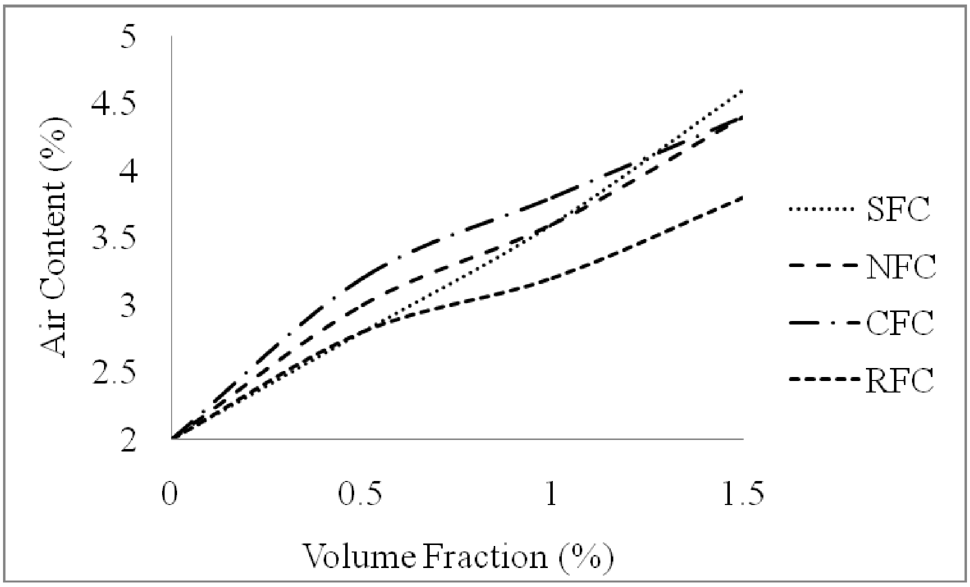

Fig 5.9 Comparision between Volume Fraction of 90mm long Thin Fibres and Air Content

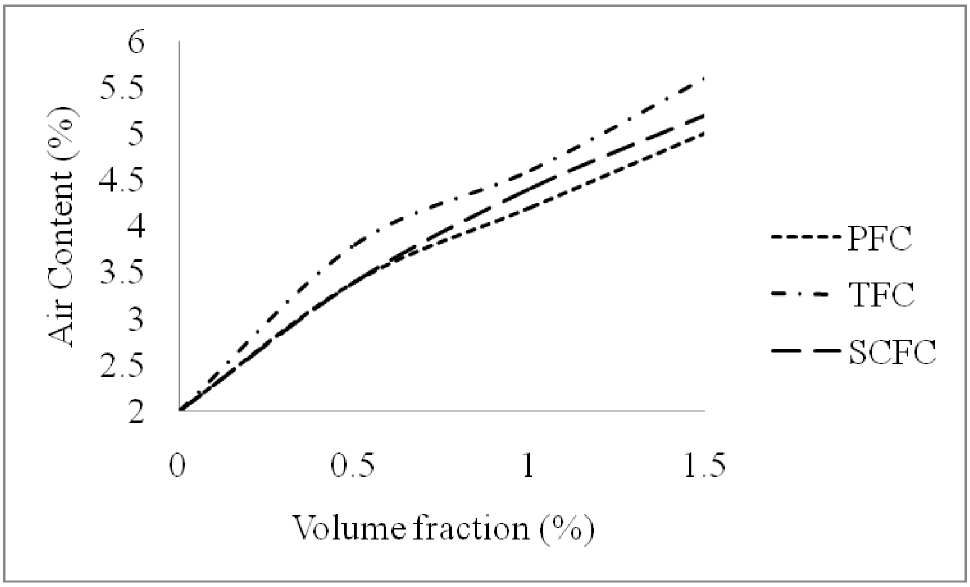

Fig 5.10 Comparision between Volume Fraction of 90mm long Thick Fibres and Air Content

While considering the effect of length of fibres on air content of fresh concrete at three different constant volume fractions such as 0.5%, 1.0% and 1.5%, thin fibres increases the air content with decrease in length whereas thick fibres increases the air content with increase in length. Fig.5.11 to 5.16 shows the effect of length of different fibres at three constant volume fraction of fibres. From Fig.5.11, Fig.5.13 and Fig.5.15, it is observed that thin fibres such as steel, nylon, coir increases the air content n decreasing the length from 90 mm to 30 mm. On other hand, if thick fibres are consideed, plastic, tyre and sugarcane fibres increases the air content while increasing the length of fibres. While observing the graphs, the decay pattern of all the fibres of three different lengths at three volume fractions is nearly same.

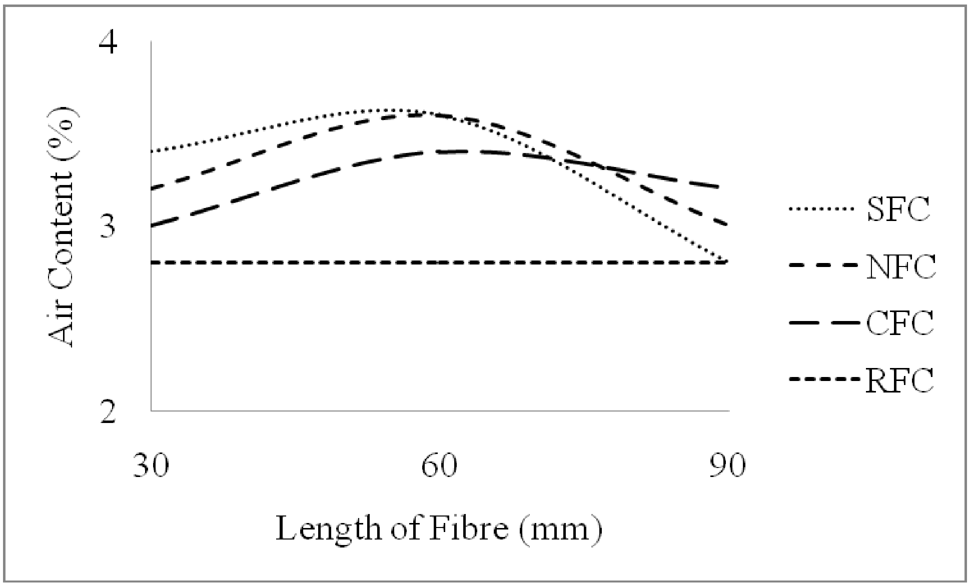

Fig 5.11 Comparision between Length of Thin Fibres at 0.5% Volume Fraction and Air Content

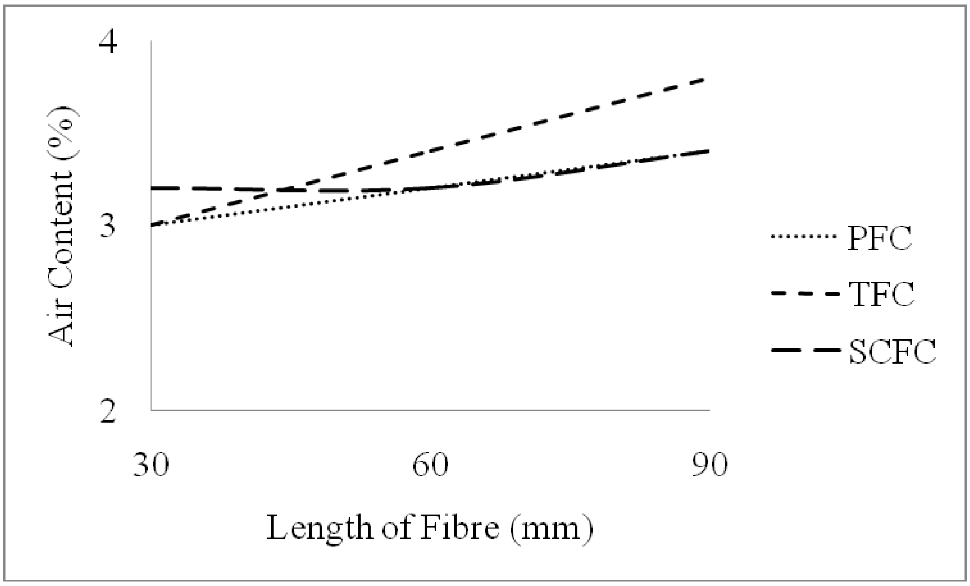

Fig 5.12 Comparision Between Length of Thick Fibres at 0.5% Volume Fraction and Air Content

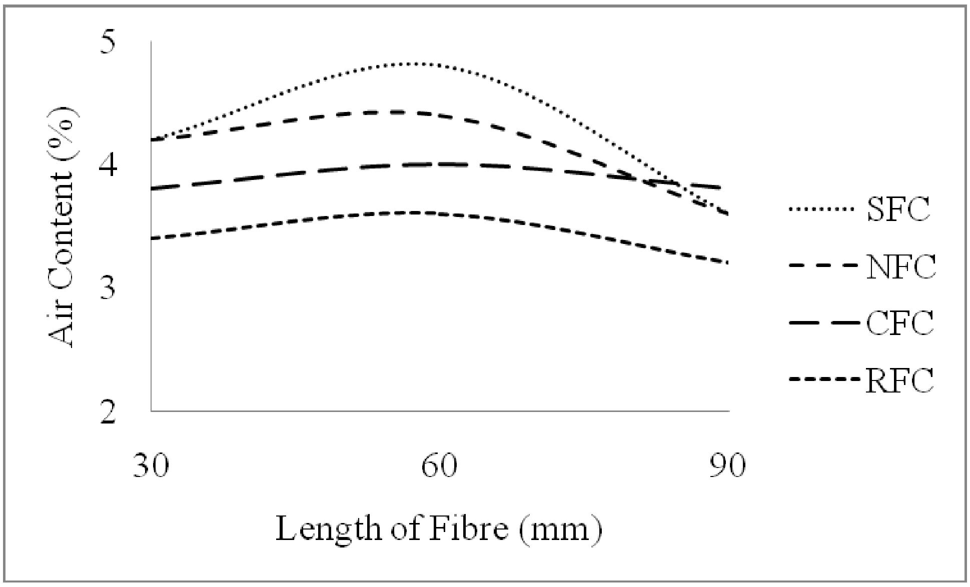

Fig 5.13 Comparision between Length of Thin Fibres at 1.0% Volume Fraction and Air Content

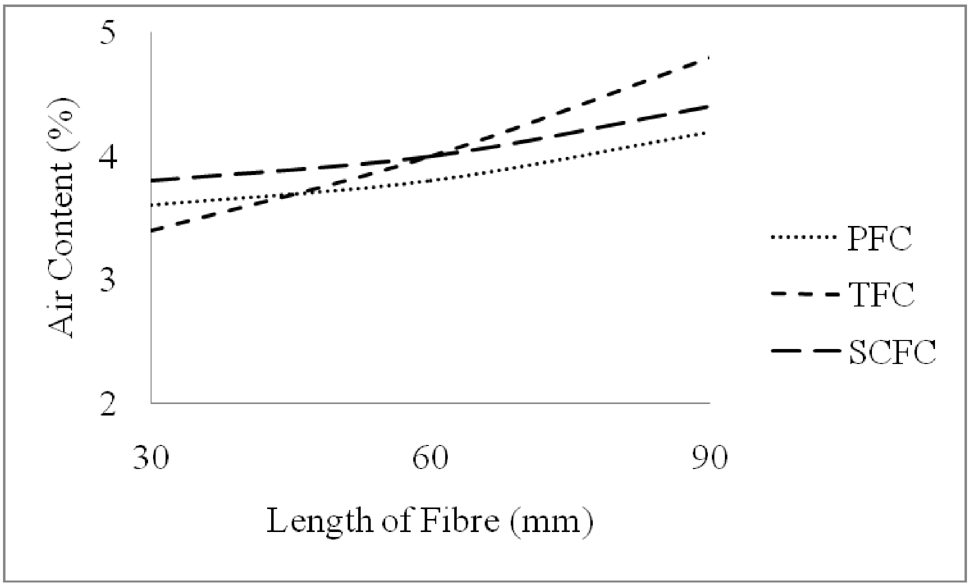

Fig 5.14 Comparision Between Length of Thick Fibres at 1.0% Volume Fraction and Air Content

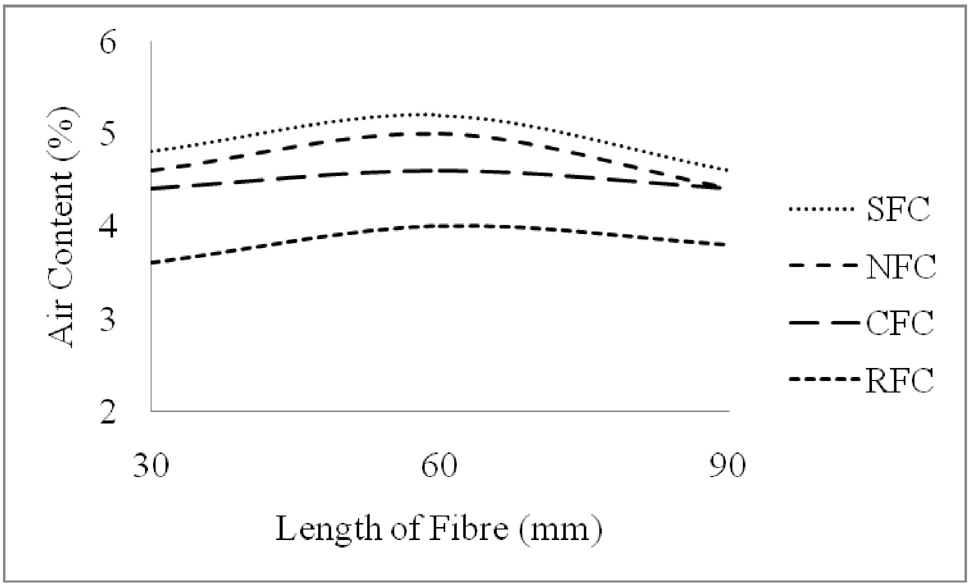

Fig 5.15 Comparision between Length of Thin Fibres at 1.5% Volume Fraction and Air Content

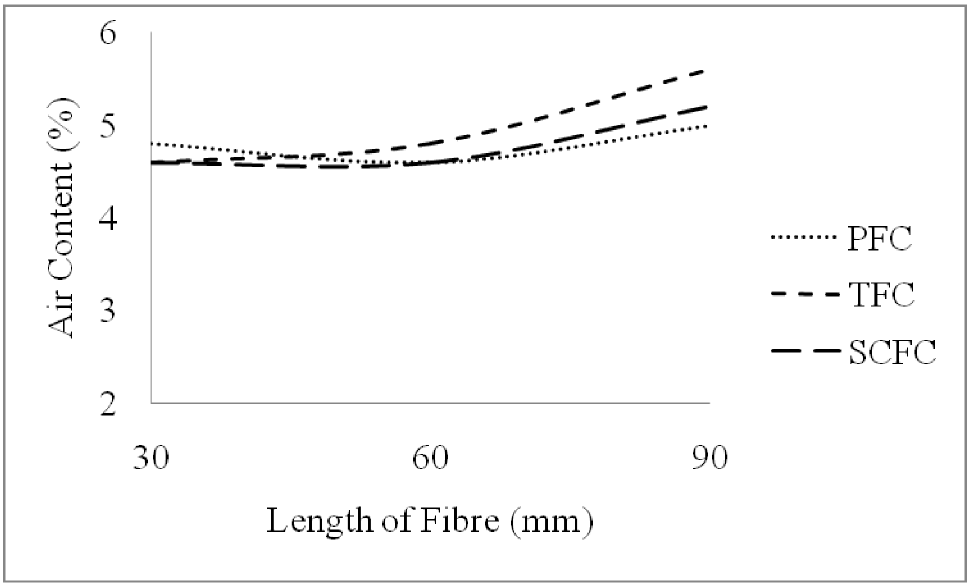

Fig 5.16 Comparision between Length of Thick Fibres at 1.5% Volume Fraction and Air Content

5.3 Mechanical Strength Studies

To evaluate the mechanical strength characteristics of concrete reinforced with different fibre materials, detailed experimental investgation was carried out and the results are discussed in the forthcoming sections. Totally eight important characteristics are discussed. Along with the fundamental material strength parameters such as cube compressive strength, cylinder compressive strength, split tensile strength, modulus of rupture, modulus of elasticity additional strength parameters such as shear strength and impact energy at both first crack as well as ultimate stages are also studied. Effect of type of fibre and fibre reinforcing index which was calculted by multiplying the volume fraction of fibres and aspect ratio of fibres on the abovementioned parameters are also discussed.

Based on the experimental results, regression analysis was carried out to form some relations between different parameters and equtions are also proposed. The proposed equations include the relation between each parameter with cyclinder compressive strength which is the most common strength parameter obtained in all the works and the relation between the parameters and fibre reinforcing index values. Using the proposed regression models, one can easily compute all the strength parameters by means of basic concrete property known as compressive strength. Another application of these parameters is in analytical invesigations. In both mechanical model and numerical finite element model formulation, the strength characteristics are used to assess the flexural performance of different structural members.

5.3.1 Cube Compressive Strength

Totally 174 cube specimens of size 150 mm with 58 mixes were casted and tested. Three volume fractions were considered for six fibres of three different lengths and one fibre of constant length. Results for compressive strength based on the average values of three test data for all mixtures are presented fibre wise from Table 5.5 to 5.11. Based on the experimental results a linear regression model is proposed for each fibre concrete. A sample comparision graph for steel fibre concrete is plotted to study the effect of fibre reinforcing index on conventional concrete stength which is shown in Fig. 5.17. The first term of the model shows the contribution of controlled concrete strength and the second term represents the contribution of fibre dosage and fibre geometry. The predicted value of the compressive strength of different mixes has been compared with the experimental results in Table 5.4.

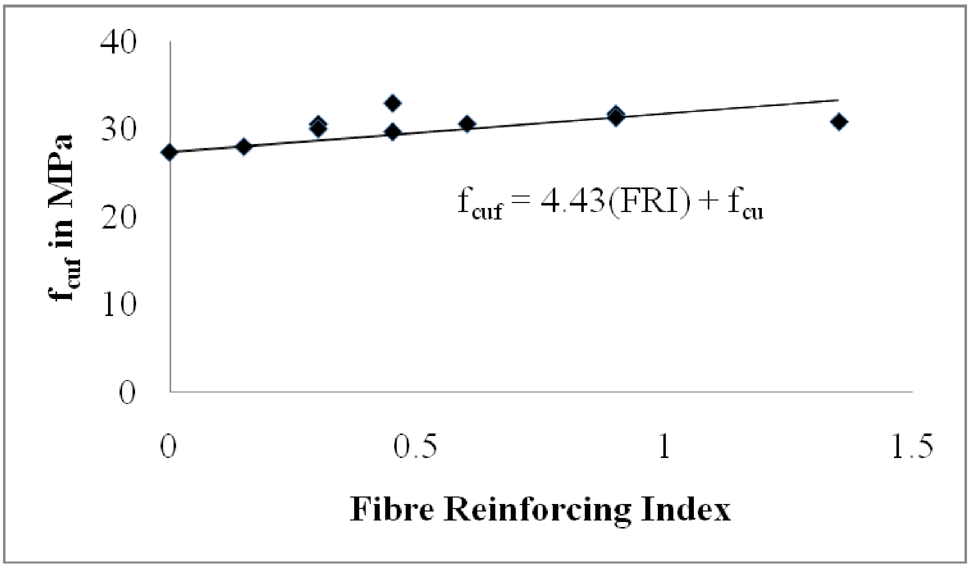

Fig. 5.17 Relation between Cube Compressive Strength of Steel Fibre Concrete and Fibre Reinforcing Index

Table 5.4 Predicted Regression Models for Cube Compressive Strength of Different Fibre Concretes

Type of Fibre Concrete	Predicted Model
Steel Fibre Concrete	$f_{cuf} = 4.43(FRI)+f_{cu}$
Nylon Fibre Concrete	$f_{cuf} = 2.83(FRI)+f_{cu}$
Plastic Fibre Concrete	$f_{cuf} = 2.43\ (FRI)+f_{cu}$
Tyre Fibre Concrete	$f_{cuf} = 1.80\ (FRI)+f_{cu}$
Coir Fibre Concrete	$f_{cuf} = 1.92\ (FRI)+f_{cu}$
Sugarcane Fibre Concrete	$f_{cuf} = 1.97\ (FRI)+f_{cu}$
Rice Husk Fibre Concrete	$f_{cuf} = 1.51\ (FRI)+f_{cu}$

From the proposed equations, it is observed that at a particular fibre reinforcing index value, steel fibres contributed more in compressive strength than others. Next to steel fibres, the contributors are nylon, plastic, sugarcene, coir, tyre and rice husk. The predicted values of compressive strength for different fibre concretes have been compared with the experimental results. The comparision indicates that the proposed compressive strength model predicts the test data accurately.

Table 5.4 also shows the percentage increase in compressive strength of fibre concrete over conventional concrete. Table 5.5 shows the compressive strength values of steel fibre concrete. The compressive strength of SFC with fibre reinforcing index 0.45 (Volume fraction = 0.5% and Aspect ratio = 90) shows the highest value. It is nearly 20% increment over conventional concrete. While considering the different volume fractions, 0.5% only gives the higher value. Increase in the volume fraction reduces the increment of compressive strength for higher Aspect ratio. It may be due to the balling effect during the concrete mixing. While considering the Aspect ratios, increase in Aspect ratio of fibres increases the stremgth almost for al the mixes. But if 1.5% volume fraction is used, the strength of higher Aspect ratio fibres has been reduced than the strength of lower Aspect ratio.

Table 5.5 Compressive Strength of Steel Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{cuf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	-	27.33	27.33	1
SFC	0.5%	30	0.15	27.95	28	1.01
	0.5%	60	0.30	30.52	28.66	0.94
	0.5%	90	0.45	32.88	29.33	0.9
	1.0%	30	0.30	29.98	28.66	0.96
	1.0%	60	0.60	30.54	29.99	0.99
	1.0%	90	0.90	31.67	31.32	0.99
	1.5%	30	0.45	29.65	29.33	0.99
	1.5%	60	0.90	31.24	31.32	1.01
	1.5%	90	1.35	30.78	33.32	1.09

Table 5.6 Compressive Strength of Nylon Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{cu} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	27.33	27.33	1

NFC	0.5%	30	0.15	27.44	27.76	0.98
	0.5%	60	0.30	27.97	28.18	0.99
	0.5%	90	0.45	29.94	28.61	0.99
	1.0%	30	0.30	27.85	28.18	0.98
	1.0%	60	0.60	29.28	29.03	0.98
	1.0%	90	0.90	32.12	29.88	0.99
	1.5%	30	0.45	28.11	28.61	0.97
	1.5%	60	0.90	29.23	29.88	0.96
	1.5%	90	1.35	30.04	31.16	0.93

Table 5.6 shows the comparision between the predicted values and experimental values of Nylon Fibre Concrete. Though the increment in cube compressive strength of NFC is less than SFC, it is much better than conventional concrete. Here also the increase in volume fraction increases the strength only upto 1%. Beyond the 1%, the strength

Table 5.7 Compressive Strength of Plastic Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{cu} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	27.33	27.33	1
PFC	0.5%	30	0.15	27.58	27.7	0.99
	0.5%	60	0.30	28.76	28.06	1.02
	0.5%	90	0.45	29.08	28.43	1.02
	1.0%	30	0.30	27.88	28.06	0.99
	1.0%	60	0.60	29.21	28.79	1.01
	1.0%	90	0.90	30.78	29.52	1.04
	1.5%	30	0.45	27.94	28.43	0.98
	1.5%	60	0.90	28.68	29.52	0.97
	1.5%	90	1.35	29.00	30.62	0.95

Table 5.8 Compressive Strength of Tyre Fibre Concrete

Type of	Volume	Aspect	FRI	f_{cu} in MPa	Model/

Concrete	fraction (v_f) in %	ratio (l/d)	(v_f x l/d)	Expt.	Model	Expt.
CC	-	-	0	27.33	27.33	1
TFC	0.5%	30	0.15	27.34	27.6	0.99
	0.5%	60	0.30	28.12	27.87	1
	0.5%	90	0.45	28.58	28.14	1.01
	1.0%	30	0.30	27.88	27.87	1
	1.0%	60	0.60	28.52	28.41	1
	1.0%	90	0.90	29.68	28.95	1.02
	1.5%	30	0.45	27.88	28.14	0.99
	1.5%	60	0.90	28.94	28.95	0.99
	1.5%	90	1.35	29.12	29.76	0.97

Table 5.9 Compressive Strength of Coir Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{cu} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	27.33	27.33	1
CFC	0.5%	30	0.15	27.34	27.6	0.99
	0.5%	60	0.30	27.96	27.87	1
	0.5%	90	0.45	28.45	28.14	1.01
	1.0%	30	0.30	27.28	27.87	0.97
	1.0%	60	0.60	28.74	28.41	1.01
	1.0%	90	0.90	30.54	28.95	1.05
	1.5%	30	0.45	28.21	28.14	1
	1.5%	60	0.90	29.44	28.95	1.01
	1.5%	90	1.35	28.62	29.76	0.96

Table 5.10 Compressive Strength of Sugarcane Fibre Concrete

Type of	Volume	Aspect	FRI	f_{cu} in MPa		Model/

Concrete	fraction (v_f) in %	ratio (l/d)	(v_f x l/d)	Expt.	Model	Expt.
CC	-	-	0	27.33	27.33	1
SCFC	0.5%	30	0.15	27.96	27.63	1.01
	0.5%	60	0.30	28.44	27.93	1.01
	0.5%	90	0.45	29.22	28.22	1.04
	1.0%	30	0.30	27.58	27.93	0.98
	1.0%	60	0.60	29.04	28.52	1.01
	1.0%	90	0.90	30.78	29.11	1.05
	1.5%	30	0.45	27.98	28.22	0.99
	1.5%	60	0.90	28.63	29.11	0.98
	1.5%	90	1.35	28.63	29.99	0.95

Table 5.11 Compressive Strength of Rice Husk Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{cu} in MPa		Model/ Expt.
				Expt.	Model	
CC	-			27.33		
RFC	0.5%			27.33		
	1.0%			27.94		
	1.5%			27.98		

5.3.2 Cylinder Compressive Strength

Totally 174 cylinder specimens of size 150 mm diameter and 300 mm height with 58 mixes were casted and tested. Three volume fractions were considered for six fibres of three different lengths and one fibre of constant length. Results for cylinder compressive strength based on the average values of three test data for all mixtures are presented fibre wise from Table 5.13 to 5.19. Based on the experimental results a linear regression model is proposed for each fibre concrete. A sample comparision graph for nylon fibre concrete is plotted to study the effect of fibre reinforcing index on conventional concrete stength which is shown in Fig. 5.18. The first term of the model shows the contribution of controlled concrete strength and the second term represents the contribution of fibre dosage and fibre geometry. The predicted value of the cylinder compressive strength of different mixes has been compared with the experimental results in Table 5.12.

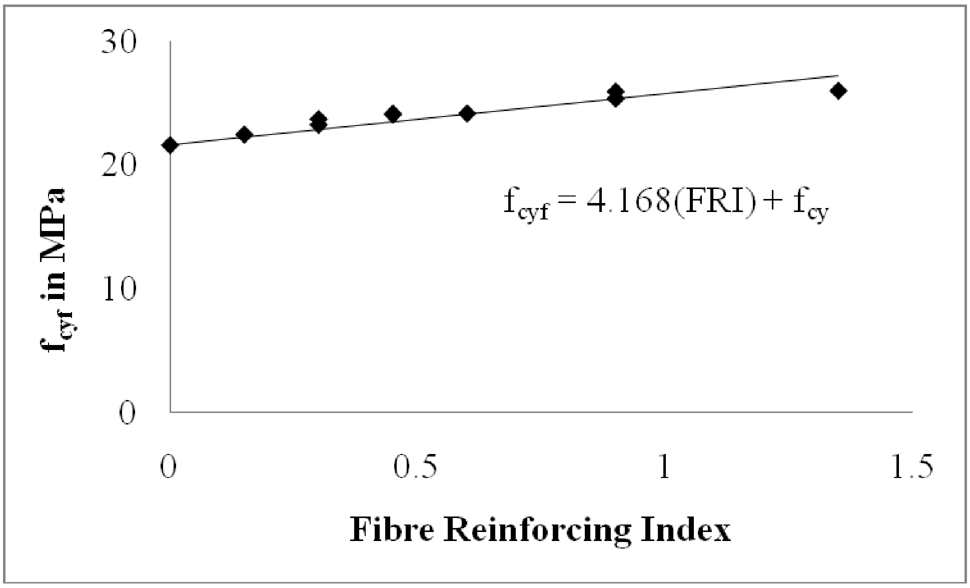

Fig. 5.18 Relation Between Cylinder Compressive Strength of Nylon Fibre Concrete and Fibre Reinforcing Index

Table 5.12 Predicted Regression Models for Cylinder Compressive Strength of Different Fibre Concretes

Type of Fibre Concrete	Predicted Model
Steel Fibre Concrete	$f_{cyf} = 4.19(FRI)+f_{cy}$
Nylon Fibre Concrete	$f_{cyf} = 4.17(FRI)+f_{cy}$
Plastic Fibre Concrete	$f_{cyf} = 3.62(FRI)+f_{cy}$
Tyre Fibre Concrete	$f_{cyf} = 4.17(FRI)+f_{cy}$
Coir Fibre Concrete	$f_{cyf} = 3.42(FRI)+f_{cy}$
Sugarcane Fibre Concrete	$f_{cyf} = 3.33(FRI)+f_{cy}$
Rice Husk Fibre Concrete	$f_{cyf} = 5.43(FRI)+f_{cy}$

From the proposed equations, it is observed that at a particular fibre reinforcing index value, rice husk, steel, nylon and tyre fibres contributed more in cylinder compressive strength than others. Plastic, coir, sugarcene are following them. The predicted values of cylinder compressive strength for different fibre concretes have been compared with the experimental results. The comparision indicates that the proposed cylindervcompressive strength model predicts the test data accurately. Table 5.14 also shows the percentage increase in cylinder compressive strength of nylon fibre concrete over conventional concrete.

Table 5.13 Cylinder Compressive Strength of Steel Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{cyf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	-	21.56	21.56	1.00
SFC	0.5%	30	0.15	23.12	22.19	0.96
	0.5%	60	0.30	23.6	22.82	0.97
	0.5%	90	0.45	24.02	23.45	0.98
	1.0%	30	0.30	23.42	22.82	0.98
	1.0%	60	0.60	24.34	24.08	0.99
	1.0%	90	0.90	25.08	25.34	1.02
	1.5%	30	0.45	24.34	23.45	0.97
	1.5%	60	0.90	25.78	25.34	0.99
	1.5%	90	1.35	26.04	27.22	1.05

Table 5.13 shows the cylinder compressive strength values of steel fibre concrete.

Table 5.14 Cylinder Compressive Strength of Nylon Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{cyf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	21.56	21.56	1
NFC	0.5%	30	0.15	22.42	22.19	0.99
	0.5%	60	0.30	23.68	22.82	0.97
	0.5%	90	0.45	24.12	23.44	0.98

	1.0%	30	0.30	23.22	22.82	0.99
	1.0%	60	0.60	24.14	24.07	1
	1.0%	90	0.90	25.34	25.32	1
	1.5%	30	0.45	24.06	23.44	0.98
	1.5%	60	0.90	25.92	25.32	0.98
	1.5%	90	1.35	25.98	27.19	1.05

Table 5.15 Cylinder Compressive Strength of Plastic Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{cyf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	21.56	21.56	1
PFC	0.5%	30	0.15	22.94	22.11	0.97
	0.5%	60	0.30	23.68	22.65	0.96
	0.5%	90	0.45	24.34	23.19	0.96
	1.0%	30	0.30	23.84	22.65	0.96
	1.0%	60	0.60	24.12	23.74	0.99
	1.0%	90	0.90	25.01	24.82	1
	1.5%	30	0.45	24.21	23.19	0.96
	1.5%	60	0.90	24.42	24.82	1.02
	1.5%	90	1.35	25.1	26.45	1.06

Table 5.16 Cylinder Compressive Strength of Tyre Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{cyf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	21.56	21.56	1
TFC	0.5%	30	0.15	22.22	22.19	1
	0.5%	60	0.30	23.43	22.82	0.98

	0.5%	90	0.45	23.58	23.44	1
	1.0%	30	0.30	22.01	22.82	1.04
	1.0%	60	0.60	22.84	24.07	1.06
	1.0%	90	0.90	24.85	25.32	1.02
	1.5%	30	0.45	23.55	23.44	1
	1.5%	60	0.90	23.56	25.32	1.08
	1.5%	90	1.35	23.54	27.19	1.16

Table 5.17 Cylinder Compressive Strength of Coir Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{cyf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	21.56	21.56	1
CFC	0.5%	30	0.15	22.5	22.08	0.99
	0.5%	60	0.30	23.02	22.59	0.99
	0.5%	90	0.45	23.75	23.1	0.98
	1.0%	30	0.30	23.08	22.59	0.98
	1.0%	60	0.60	24.12	23.62	0.98
	1.0%	90	0.90	24.67	24.64	1
	1.5%	30	0.45	23.81	23.1	0.98
	1.5%	60	0.90	24.64	24.64	1
	1.5%	90	1.35	25.22	26.18	1.04

Table 5.18 Cylinder Compressive Strength of Sugarcane Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{cyf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	21.56	21.56	1
SCFC	0.5%	30	0.15	23.89	22.06	0.93
	0.5%	60	0.30	24.65	22.56	0.92
	0.5%	90	0.45	24.88	23.06	0.93

	1.0%	30	0.30	23.64	22.56	0.96
	1.0%	60	0.60	24.08	23.56	0.98
	1.0%	90	0.90	25.46	24.56	0.97
	1.5%	30	0.45	23.84	23.06	0.97
	1.5%	60	0.90	23.88	24.56	1.03
	1.5%	90	1.35	23.9	26.06	1.1

Table 5.19 Cylinder Compressive Strength of Rice Husk Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{cyf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-			27.33		
RFC	0.5%			27.33		
	1.0%			27.94		
	1.5%			27.98		

5.3.3 Split Tensile Strength

Totally 174 cylinder specimens of size 150 mm diameter and 300 mm height with 58 mixes were casted and tested. Three volume fractions were considered for six fibres of three different lengths and one fibre of constant length. Results for split tensile strength based on the average values of three test data for all mixtures are presented fibre wise from Table 5.21 to 5.27. Based on the experimental results a linear regression model is proposed for each fibre concrete. A sample comparision graph for plastic fibre concrete is plotted to study the effect of fibre reinforcing index on conventional concrete stength which is shown in Fig. 5.19. The first term of the model shows the contribution of controlled concrete strength and the second term represents the contribution of fibre dosage and fibre geometry. The predicted value of the split tensile strength of different mixes has been compared with the experimental results in Table 5.20.

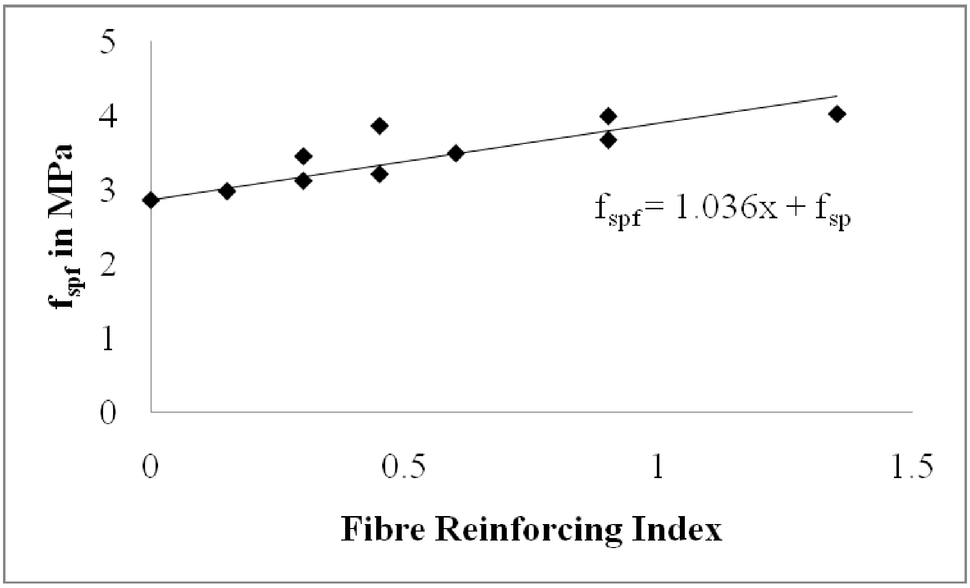

Fig. 5.19 Relation Between split tensile Strength of Plastic Fibre Concrete and Fibre Reinforcing Index

Table 5.20 Predicted Regression Models for split tensile Strength of Different Fibre Concretes

Type of Fibre Concrete	Predicted Model
Steel Fibre Concrete	f_{cyf} = 1.68(FRI)+f_{cy}
Nylon Fibre Concrete	f_{cyf} = 1.03(FRI)+f_{cy}
Plastic Fibre Concrete	f_{cyf} = 1.04(FRI)+f_{cy}
Tyre Fibre Concrete	f_{cyf} = 0.72(FRI)+f_{cy}
Coir Fibre Concrete	f_{cyf} = 0.83(FRI)+f_{cy}
Sugarcane Fibre Concrete	f_{cyf} = 0.71(FRI)+f_{cy}
Rice Husk Fibre Concrete	f_{cyf} = 0.26(FRI)+f_{cy}

From the proposed equations, it is observed that at a particular fibre reinforcing index value, rice husk, steel, nylon and tyre fibres contributed more in cylinder compressive strength than others. Plastic, coir, sugarcene are following them. The predicted values of cylinder compressive strength for different fibre concretes have been compared with the experimental results. The comparision indicates that the proposed cylinder compressive strength model predicts the test data accurately. Table 5.14 also shows the percentage increase in cylinder compressive strength of nylon fibre concrete over conventional concrete.

Table 5.21 Split Tensile Strength of Steel Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{spf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	-	2.86	2.86	1
SFC	0.5%	30	0.15	3.22	3.12	0.97
	0.5%	60	0.30	3.56	3.37	0.95
	0.5%	90	0.45	3.75	3.62	0.97
	1.0%	30	0.30	3.53	3.37	0.96
	1.0%	60	0.60	3.92	3.87	0.99
	1.0%	90	0.90	4.42	4.38	1
	1.5%	30	0.45	3.81	3.62	0.96
	1.5%	60	0.90	4.28	4.38	1.03
	1.5%	90	1.35	4.92	5.13	1.05

Table 5.13 shows the cylinder compressive strength values of steel fibre concrete.

Table 5.22 Split Tensile Strength of Nylon Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{spf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	2.86	2.86	1
NFC	0.5%	30	0.15	3.12	3.02	0.97
	0.5%	60	0.30	3.45	3.17	0.92
	0.5%	90	0.45	3.58	3.33	0.94
	1.0%	30	0.30	3.22	3.17	0.99
	1.0%	60	0.60	3.48	3.48	1
	1.0%	90	0.90	3.78	3.79	1.01
	1.5%	30	0.45	3.46	3.33	0.97
	1.5%	60	0.90	3.92	3.79	0.97

	1.5%	90	1.35	3.94	4.26	1.09

Table 5.23 Split Tensile Strength of Plastic Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{spf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	2.86	2.86	1
PFC	0.5%	30	0.15	2.98	3.02	1.02
	0.5%	60	0.30	3.45	3.18	0.93
	0.5%	90	0.45	3.86	3.33	0.87
	1.0%	30	0.30	3.12	3.18	1.02
	1.0%	60	0.60	3.49	3.49	1
	1.0%	90	0.90	3.99	3.8	0.96
	1.5%	30	0.45	3.21	3.33	1.04
	1.5%	60	0.90	3.67	3.8	1.04
	1.5%	90	1.35	4.02	4.27	1.07

Table 5.24 Split Tensile Strength of Tyre Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{spf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	2.86	2.86	1
TFC	0.5%	30	0.15	2.85	2.97	1.05
	0.5%	60	0.30	3.74	3.08	0.83
	0.5%	90	0.45	3.95	3.19	0.81
	1.0%	30	0.30	2.67	3.08	1.16

	1.0%	60	0.60	3.56	3.3	0.93
	1.0%	90	0.90	3.85	3.51	0.92
	1.5%	30	0.45	3.48	3.19	0.92
	1.5%	60	0.90	3.24	3.51	1.09
	1.5%	90	1.35	3.28	3.84	1.18

Table 5.25 Split Tensile Strength of Coir Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{spf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	2.86	2.86	1
CFC	0.5%	30	0.15	2.98	2.99	1.01
	0.5%	60	0.30	3.06	3.11	1.02
	0.5%	90	0.45	3.12	3.24	1.04
	1.0%	30	0.30	3.24	3.11	0.96
	1.0%	60	0.60	3.92	3.36	0.86
	1.0%	90	0.90	3.98	3.61	0.91
	1.5%	30	0.45	3.22	3.24	1.01
	1.5%	60	0.90	3.64	3.61	1
	1.5%	90	1.35	3.48	3.99	1.15

Table 5.26 Split Tensile Strength of Sugarcane Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{spf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	2.86	2.86	1
SCFC	0.5%	30	0.15	2.86	2.97	1.04
	0.5%	60	0.30	2.86	3.08	1.08
	0.5%	90	0.45	3.14	3.18	1.02

	1.0%	30	0.30	3.95	3.08	0.78
	1.0%	60	0.60	3.28	3.29	1.01
	1.0%	90	0.90	3.86	3.5	0.91
	1.5%	30	0.45	3.22	3.18	0.99
	1.5%	60	0.90	3.47	3.5	1.01
	1.5%	90	1.35	3.5	3.82	1.1

Table 5.27 Split Tensile Strength of Rice Husk Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{spf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-			27.33		
RFC	0.5%			27.33		
	1.0%			27.94		
	1.5%			27.98		

Apart from the comparision between the split tensile strength of different concrete mix, another relationship has been proposed between the split tensile strength and the cylinder compressive strength. Table.5.28 shows the proposed relations between split tensile and cylinder compressive strength of different concrete mixes.

Table.5.28 Relation between Split Tensile and Compressive Strength

Concrete Type	Relationship
Conventional Concrete	$f_{sp} = 0.62\sqrt{f_{cy}}$
Steel Fibre Concrete	$f_{sp} = 0.78\sqrt{f_{cy}}$
Nylon Fibre Concrete	$f_{sp} = 0.71\sqrt{f_{cy}}$
Plastic Fibre Concrete	$f_{sp} = 0.71\sqrt{f_{cy}}$
Tyre Fibre Concrete	$f_{sp} = 0.70\sqrt{f_{cy}}$
Coir Fibre Concrete	$f_{sp} = 0.69\sqrt{f_{cy}}$
Sugarcane Fibre Concrete	$f_{sp} = 0.67\sqrt{f_{cy}}$
Rice Husk Fibre Concrete	$f_{sp} = 0.61\sqrt{f_{cy}}$

5.3.4 Modulus of Rupture

Totally 174 prism specimens of size 500 mm x 100 mm x 100 mm with 58 mixes were casted and tested. Three volume fractions were considered for six fibres of three different lengths and one fibre of constant length. Results for Modulus of Rupture based on the average values of three test data for all mixtures are presented fibre wise from Table 5.29 to 5.35. Based on the experimental results a linear regression model is proposed for each fibre concrete. A sample comparision graph for plastic fibre concrete is plotted to study the effect of fibre reinforcing index on conventional concrete stength which is shown in Fig. 5.20. The first term of the model shows the contribution of controlled concrete strength and the second term represents the contribution of fibre dosage and fibre geometry. The predicted value of the split tensile strength of different mixes has been compared with the experimental results in Table 5.28.

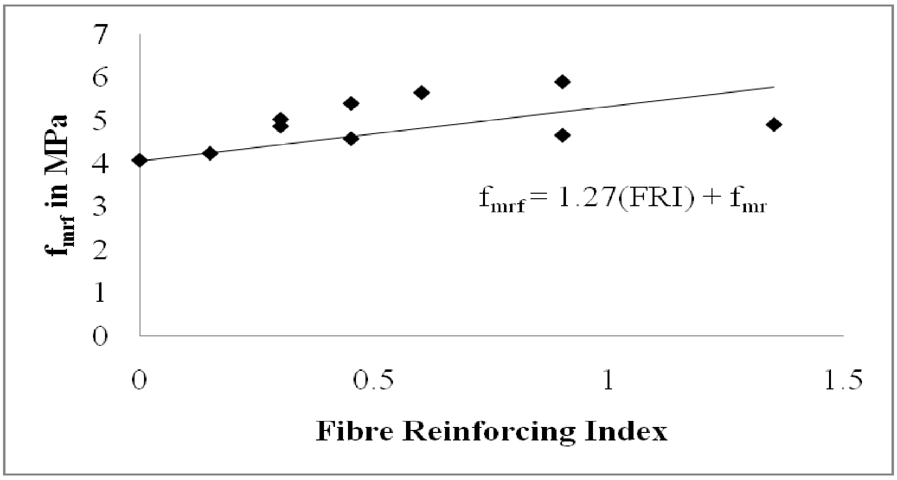

Fig. 5.19 Relation between Modulus of Rupture of Tyre Fibre Concrete and Fibre Reinforcing Index

Table 5.20 Predicted Regression Models for Modulus of Rupture of Different Fibre Concretes

Type of Fibre Concrete	Predicted Model
Steel Fibre Concrete	$f_{mrf} = 2.62(FRI)+f_{mr}$
Nylon Fibre Concrete	$f_{mrf} = 1.56(FRI)+f_{mr}$
Plastic Fibre Concrete	$f_{mrf} = 2.01(FRI)+f_{mr}$
Tyre Fibre Concrete	$f_{mrf} = 1.27(FRI)+f_{mr}$

Coir Fibre Concrete	$f_{mrf} = 1.38(FRI)+f_{mr}$
Sugarcane Fibre Concrete	$f_{mrf} = 0.94(FRI)+f_{mr}$
Rice Husk Fibre Concrete	$f_{mrf} = 2.15(FRI)+f_{mr}$

From the proposed equations, it is observed that at a particular fibre reinforcing index value, rice husk, steel, nylon and tyre fibres contributed more in cylinder compressive strength than others. Plastic, coir, sugarcene are following them. The predicted values of cylinder compressive strength for different fibre concretes have been compared with the experimental results. The comparision indicates that the proposed cylindervcompressive strength model predicts the test data accurately. Table 5.14 also shows the percentage increase in cylinder compressive strength of nylon fibre concrete over conventional concrete.

Table 5.21 Modulus of Rupture of Steel Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{mrf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	-	4.06	4.06	1
SFC	0.5%	30	0.15	4.45	4.46	1.01
	0.5%	60	0.30	4.84	4.85	1.01
	0.5%	90	0.45	5.42	5.24	0.97
	1.0%	30	0.30	4.8	4.85	1.02
	1.0%	60	0.60	5.54	5.64	1.02
	1.0%	90	0.90	6.02	6.42	1.07
	1.5%	30	0.45	5.02	5.24	1.05
	1.5%	60	0.90	6.43	6.42	1
	1.5%	90	1.35	7.91	7.6	0.97

Table 5.13 shows the cylinder compressive strength values of steel fibre concrete.

Table 5.22 Modulus of Rupture of Nylon Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{mrf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	4.06	4.06	1

NFC	0.5%	30	0.15	4.22	4.3	1.02
	0.5%	60	0.30	4.86	4.53	0.94
	0.5%	90	0.45	5.23	4.77	0.92
	1.0%	30	0.30	4.34	4.53	1.05
	1.0%	60	0.60	4.97	5	1.01
	1.0%	90	0.90	5.34	5.47	1.03
	1.5%	30	0.45	4.56	4.77	1.05
	1.5%	60	0.90	5.34	5.47	1.03
	1.5%	90	1.35	6.23	6.17	1

Table 5.23 Modulus of Rupture of Plastic Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{mrf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	4.06	4.06	1
PFC	0.5%	30	0.15	4.23	4.35	1.03
	0.5%	60	0.30	4.98	4.63	0.93
	0.5%	90	0.45	5.68	4.91	0.87
	1.0%	30	0.30	5.34	4.63	0.87
	1.0%	60	0.60	5.96	5.19	0.88
	1.0%	90	0.90	6	5.75	0.96
	1.5%	30	0.45	4.67	4.91	1.06
	1.5%	60	0.90	5.84	5.75	0.99
	1.5%	90	1.35	6.05	6.59	1.09

Table 5.24 Modulus of Rupture of Tyre Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{mrf} in MPa		Model/ Expt.
				Expt.	Model	

CC	-	-	0	4.06	4.06	1
TFC	0.5%	30	0.15	4.22	4.26	1.01
	0.5%	60	0.30	4.85	4.45	0.92
	0.5%	90	0.45	5.38	4.64	0.87
	1.0%	30	0.30	5.01	4.45	0.89
	1.0%	60	0.60	5.63	4.83	0.86
	1.0%	90	0.90	5.88	5.21	0.89
	1.5%	30	0.45	4.56	4.64	1.02
	1.5%	60	0.90	4.64	5.21	1.13
	1.5%	90	1.35	4.89	5.78	1.19

Table 5.25 Modulus of Rupture of Coir Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{mrf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	4.06	4.06	1
CFC	0.5%	30	0.15	4.55	4.27	0.94
	0.5%	60	0.30	5.01	4.48	0.9
	0.5%	90	0.45	5.43	4.69	0.87
	1.0%	30	0.30	4.56	4.48	0.99
	1.0%	60	0.60	5.02	4.89	0.98
	1.0%	90	0.90	5.88	5.31	0.91
	1.5%	30	0.45	4.28	4.69	1.1
	1.5%	60	0.90	4.95	5.31	1.08
	1.5%	90	1.35	5.44	5.93	1.1

Table 5.26 Modulus of Rupture of Sugarcane Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{mrf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	4.06	4.06	1

SCFC	0.5%	30	0.15	4.33	4.21	0.98
	0.5%	60	0.30	4.56	4.35	0.96
	0.5%	90	0.45	4.98	4.49	0.91
	1.0%	30	0.30	4.44	4.35	0.98
	1.0%	60	0.60	4.79	4.63	0.97
	1.0%	90	0.90	5.32	4.91	0.93
	1.5%	30	0.45	4.33	4.49	1.04
	1.5%	60	0.90	4.56	4.91	1.08
	1.5%	90	1.35	5	5.33	1.07

Table 5.27 Modulus of Rupture of Rice Husk Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{mrf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-			27.33		
RFC	0.5%			27.33		
	1.0%			27.94		
	1.5%			27.98		

Apart from the comparision between the split tensile strength of different concrete mix, another relationship has been proposed between the split tensile strength and the cylinder compressive strength. Table.5.28 shows the proposed relations between split tensile and cylinder compressive strength of different concrete mixes.

Table.5.28 Relation between Split Tensile and Compressive Strength

Concrete Type	Relationship
Conventional Concrete	$f_{mr} = 0.73\sqrt{f_{cy}}$
Steel Fibre Concrete	$f_{mr} = 0.94\sqrt{f_{cy}}$
Nylon Fibre Concrete	$f_{mr} = 0.93\sqrt{f_{cy}}$
Plastic Fibre Concrete	$f_{mr} = 0.90\sqrt{f_{cy}}$
Tyre Fibre Concrete	$f_{mr} = 0.93\sqrt{f_{cy}}$
Coir Fibre Concrete	$f_{mr} = 0.93\sqrt{f_{cy}}$

Sugarcane Fibre Concrete	$f_{mr} = 0.91\sqrt{f_{cy}}$
Rice Husk Fibre Concrete	$f_{mr} = 0.90\sqrt{f_{cy}}$

5.3.5 Modulus of Elasticity

Totally 174 cylinder specimens of size 150 mm diameter and 300 mm height with 58 mixes were casted and tested. Three volume fractions were considered for six fibres of three different lengths and one fibre of constant length. Results for modulus of elasticity based on the average values of three test data for all mixtures are presented fibre wise from Table 5.21 to 5.27. Based on the experimental results a linear regression model is proposed for each fibre concrete. A sample comparision graph for coir fibre concrete is plotted to study the effect of fibre reinforcing index on conventional concrete stength which is shown in Fig. 5.19. The first term of the model shows the contribution of controlled concrete strength and the second term represents the contribution of fibre dosage and fibre geometry. The predicted value of the modulus of elasticity of different mixes has been compared with the experimental results in Table 5.20.

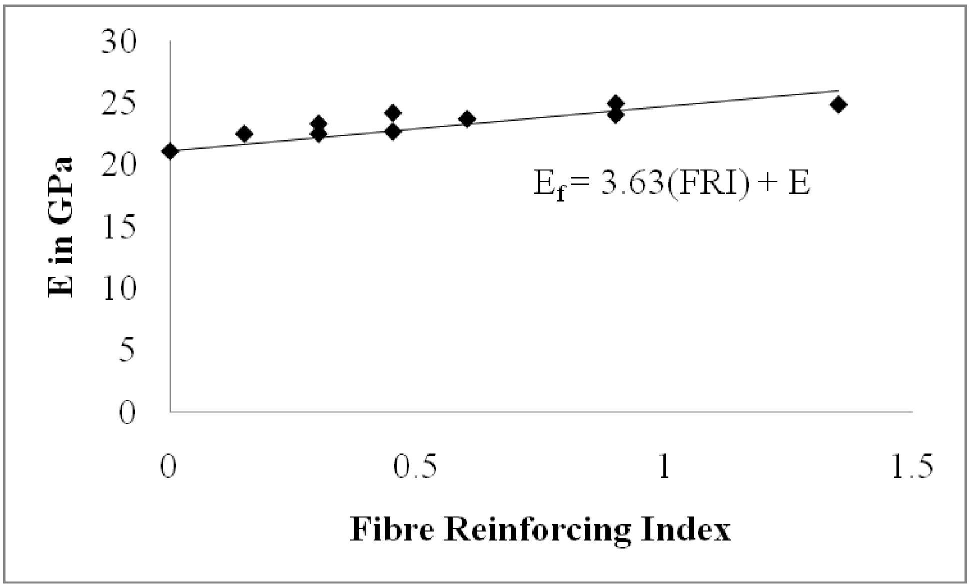

Fig. 5.19 Relation Between Modulus of Elasticity of Coir Fibre Concrete and Fibre Reinforcing Index

Table 5.20 Predicted Regression Models for Modulus of Elasticity of Different Fibre Concretes

Type of Fibre Concrete	Predicted Model

Steel Fibre Concrete	E_f = 7.18(FRI)+E
Nylon Fibre Concrete	E_f = 5.55(FRI)+E
Plastic Fibre Concrete	E_f = 4.43(FRI)+E
Tyre Fibre Concrete	E_f = 5.82(FRI)+E
Coir Fibre Concrete	E_f = 3.63(FRI)+E
Sugarcane Fibre Concrete	E_f = 3.10(FRI)+E
Rice Husk Fibre Concrete	E_f = 5.62(FRI)+E

From the proposed equations, it is observed that at a particular fibre reinforcing index value, rice husk, steel, nylon and tyre fibres contributed more in cylinder compressive strength than others. Plastic, coir, sugarcene are following them. The predicted values of cylinder compressive strength for different fibre concretes have been compared with the experimental results. The comparision indicates that the proposed cylindervcompressive strength model predicts the test data accurately. Table 5.14 also shows the percentage increase in cylinder compressive strength of nylon fibre concrete over conventional concrete.

Table 5.21 Modulus of Elasticity of Steel Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	E_f in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	-	21.06	21.06	1
SFC	0.5%	30	0.15	24.57	22.14	0.91
	0.5%	60	0.30	25.18	23.22	0.93
	0.5%	90	0.45	25.87	24.30	0.94
	1.0%	30	0.30	24.49	23.22	0.95
	1.0%	60	0.60	26.90	25.37	0.95
	1.0%	90	0.90	27.29	27.53	1.01
	1.5%	30	0.45	25.56	24.30	0.96
	1.5%	60	0.90	27.32	27.53	1.01
	1.5%	90	1.35	28.40	30.76	1.09

Table 5.13 shows the cylinder compressive strength values of steel fibre concrete.

Table 5.22 Modulus of Elasticity of Nylon Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	E_f in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	21.06	21.06	1
NFC	0.5%	30	0.15	23.45	21.90	0.94
	0.5%	60	0.30	24.90	22.73	0.92
	0.5%	90	0.45	25.57	23.56	0.93
	1.0%	30	0.30	24.00	22.73	0.95
	1.0%	60	0.60	25.48	24.39	0.96
	1.0%	90	0.90	26.20	26.06	1
	1.5%	30	0.45	24.58	23.56	0.96
	1.5%	60	0.90	25.48	26.06	1.03
	1.5%	90	1.35	26.39	28.56	1.09

Table 5.23 Modulus of Elasticity of Plastic Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	E_f in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	21.06	21.06	1
PFC	0.5%	30	0.15	23.45	21.73	0.93
	0.5%	60	0.30	24.56	22.39	0.92
	0.5%	90	0.45	24.83	23.06	0.93
	1.0%	30	0.30	23.23	22.39	0.97
	1.0%	60	0.60	24.35	23.72	0.98
	1.0%	90	0.90	24.56	25.05	1.02
	1.5%	30	0.45	23.94	23.06	0.97
	1.5%	60	0.90	24.35	25.05	1.03
	1.5%	90	1.35	25.78	27.05	1.05

Table 5.24 Modulus of Elasticity of Tyre Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	E_f in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	21.059	21.059	1
TFC	0.5%	30	0.15	22.4	21.939	0.98
	0.5%	60	0.30	23.12	22.809	0.99
	0.5%	90	0.45	24.452	23.679	0.97
	1.0%	30	0.30	23.462	22.809	0.98
	1.0%	60	0.60	25.689	24.559	0.96
	1.0%	90	0.90	26.064	26.299	1.01
	1.5%	30	0.45	25.4	23.679	0.94
	1.5%	60	0.90	26.48	26.299	1
	1.5%	90	1.35	27.345	28.919	1.06

Table 5.25 Modulus of Elasticity of Coir Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	E_f in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	21.06	21.06	1
CFC	0.5%	30	0.15	22.45	21.61	0.97
	0.5%	60	0.30	23.26	22.15	0.96
	0.5%	90	0.45	24.12	22.70	0.95
	1.0%	30	0.30	22.45	22.15	0.99
	1.0%	60	0.60	23.64	23.24	0.99
	1.0%	90	0.90	24.88	24.33	0.98
	1.5%	30	0.45	22.63	22.70	1.01
	1.5%	60	0.90	23.97	24.33	1.02
	1.5%	90	1.35	24.79	25.97	1.05

Table 5.26 Modulus of Elasticity of Sugarcane Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	E_f in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	21.06	21.06	1
SCFC	0.5%	30	0.15	22.00	21.53	0.98
	0.5%	60	0.30	23.34	21.99	0.95
	0.5%	90	0.45	24.12	22.46	0.94
	1.0%	30	0.30	22.39	21.99	0.99
	1.0%	60	0.60	23.06	22.92	1
	1.0%	90	0.90	24.79	23.85	0.97
	1.5%	30	0.45	22.99	22.46	0.98
	1.5%	60	0.90	23.14	23.85	1.04
	1.5%	90	1.35	23.87	25.25	1.06

Apart from the comparision between the split tensile strength of different concrete mix, another relationship has been proposed between the split tensile strength and the cylinder compressive strength. Table.5.28 shows the proposed relations between split tensile and cylinder compressive strength of different concrete mixes.

Table.5.28 Relation between Split Tensile and Compressive Strength

Concrete Type	Relationship
Conventional Concrete	$f_{mr} = 0.73\sqrt{f_{cy}}$
Steel Fibre Concrete	$f_{mr} = 0.94\sqrt{f_{cy}}$
Nylon Fibre Concrete	$f_{mr} = 0.93\sqrt{f_{cy}}$
Plastic Fibre Concrete	$f_{mr} = 0.90\sqrt{f_{cy}}$
Tyre Fibre Concrete	$f_{mr} = 0.93\sqrt{f_{cy}}$
Coir Fibre Concrete	$f_{mr} = 0.93\sqrt{f_{cy}}$
Sugarcane Fibre Concrete	$f_{mr} = 0.91\sqrt{f_{cy}}$
Rice Husk Fibre Concrete	$f_{mr} = 0.90\sqrt{f_{cy}}$

5.3.6 Shear Strength

Totally 174 'L' shaped specimens with 58 mixes were casted and tested. Three volume fractions were considered for six fibres of three different lengths and one fibre of constant length. Results for modulus of elasticity based on the average values of three test data for all mixtures are presented fibre wise from Table 5.21 to 5.27. Based on the experimental results a linear regression model is proposed for each fibre concrete. A sample comparision graph for coir fibre concrete is plotted to study the effect of fibre reinforcing index on conventional concrete stength which is shown in Fig. 5.19. The first term of the model shows the contribution of controlled concrete strength and the second term represents the contribution of fibre dosage and fibre geometry. The predicted value of the modulus of elasticity of different mixes has been compared with the experimental results in Table 5.20.

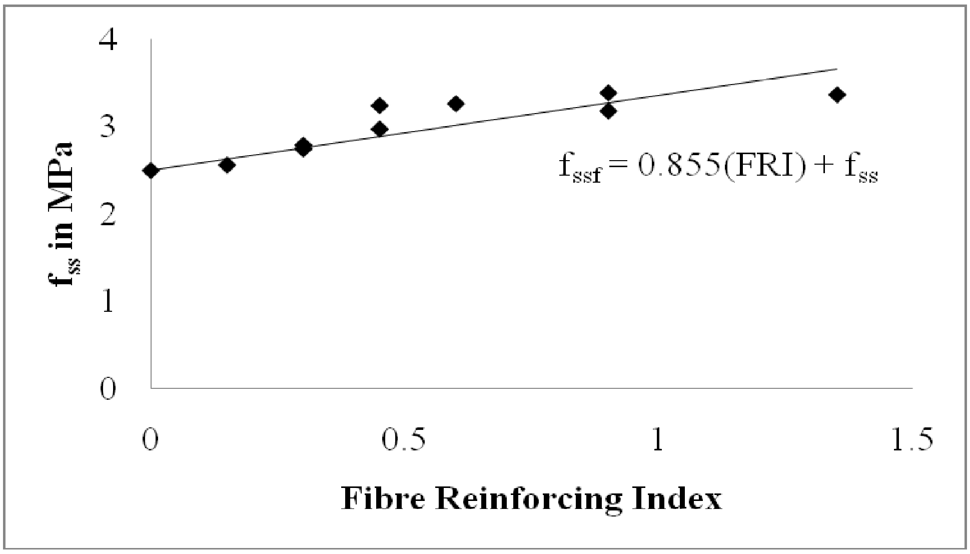

Fig. 5.19 Relation between Cracking shear strength of Sugarcane Fibre Concrete and Fibre Reinforcing Index

Table 5.20 Predicted Regression Models for Modulus of Elasticity of Different Fibre Concretes

Type of Fibre Concrete	Predicted Model
Steel Fibre Concrete	f_{ssf}= 1.73(FRI)+f_{ss}
Nylon Fibre Concrete	f_{ssf}= 1.85(FRI)+f_{ss}
Plastic Fibre Concrete	f_{ssf}= 2.06(FRI)+f_{ss}
Tyre Fibre Concrete	f_{ssf}= 1.49(FRI)+f_{ss}
Coir Fibre Concrete	f_{ssf}= 1.08(FRI)+f_{ss}
Sugarcane Fibre Concrete	f_{ssf}= 0.86(FRI)+f_{ss}
Rice Husk Fibre Concrete	f_{ssf}= 0.95(FRI)+f_{ss}

From the proposed equations, it is observed that at a particular fibre reinforcing index value, rice husk, steel, nylon and tyre fibres contributed more in cylinder compressive strength than others. Plastic, coir, sugarcene are following them. The predicted values of cylinder compressive strength for different fibre concretes have been compared with the experimental results. The comparision indicates that the proposed cylindervcompressive strength model predicts the test data accurately. Table 5.14 also shows the percentage increase in cylinder compressive strength of nylon fibre concrete over conventional concrete.

Table 5.21 Shear Strength of Steel Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{ssf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	-	6.12	6.12	1
SFC	0.5%	30	0.15	8.24	6.38	0.78
	0.5%	60	0.30	9.31	6.64	0.72
	0.5%	90	0.45	9.88	6.9	0.7
	1.0%	30	0.30	8.43	6.64	0.79
	1.0%	60	0.60	9.98	7.16	0.72
	1.0%	90	0.90	10.23	7.68	0.76
	1.5%	30	0.45	9.06	6.9	0.77
	1.5%	60	0.90	9.85	7.68	0.78
	1.5%	90	1.35	10.45	10.31	0.99

Table 5.13 shows the cylinder compressive strength values of steel fibre concrete.

Table 5.22 Shear Strength of Nylon Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{ssf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	6.12	6.12	1
NFC	0.5%	30	0.15	7.68	6.4	0.84
	0.5%	60	0.30	7.94	6.68	0.85
	0.5%	90	0.45	8.38	6.96	0.84
	1.0%	30	0.30	7.21	6.68	0.93
	1.0%	60	0.60	8.04	7.23	0.9
	1.0%	90	0.90	9.56	7.79	0.82
	1.5%	30	0.45	7.32	6.96	0.96
	1.5%	60	0.90	8.58	7.79	0.91
	1.5%	90	1.35	9.56	8.62	0.91

Table 5.23 Shear Strength of Plastic Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{ssf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	6.12	6.12	1
PFC	0.5%	30	0.15	6.78	6.43	0.95
	0.5%	60	0.30	7.72	6.74	0.88
	0.5%	90	0.45	8.06	7.05	0.88
	1.0%	30	0.30	6.89	6.74	0.98
	1.0%	60	0.60	7.46	7.36	0.99
	1.0%	90	0.90	8.35	7.98	0.96
	1.5%	30	0.45	6.77	7.05	1.05
	1.5%	60	0.90	7.92	7.98	1.01
	1.5%	90	1.35	8.02	8.91	1.12

Table 5.24 Shear Strength of Tyre Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{ssf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	6.12	6.12	1
TFC	0.5%	30	0.15	6.56	6.35	0.97
	0.5%	60	0.30	7	6.57	0.94
	0.5%	90	0.45	7.21	6.8	0.95
	1.0%	30	0.30	6.89	6.57	0.96
	1.0%	60	0.60	6.95	7.02	1.02
	1.0%	90	0.90	7.34	7.47	1.02
	1.5%	30	0.45	6.98	6.8	0.98
	1.5%	60	0.90	7.01	7.47	1.07
	1.5%	90	1.35	6.89	8.14	1.19

Table 5.25 Compressive Shear Strength of Coir Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{ssf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	6.12	6.12	1
CFC	0.5%	30	0.15	6.98	6.29	0.91
	0.5%	60	0.30	7.56	6.45	0.86
	0.5%	90	0.45	8.43	6.61	0.79
	1.0%	30	0.30	6.85	6.45	0.95
	1.0%	60	0.60	7.78	6.77	0.88
	1.0%	90	0.90	8.91	7.1	0.8
	1.5%	30	0.45	7.04	6.61	0.94
	1.5%	60	0.90	8.22	7.1	0.87
	1.5%	90	1.35	8.54	7.58	0.89

Table 5.26 Compressive Shear Strength of Sugarcane Fibre Concrete

Type of Concrete	Volume fraction (v_f) in %	Aspect ratio (l/d)	FRI (v_f x l/d)	f_{ssf} in MPa		Model/ Expt.
				Expt.	Model	
CC	-	-	0	6.12	6.12	1
SCFC	0.5%	30	0.15	6.78	6.25	0.93
	0.5%	60	0.30	7.32	6.38	0.88
	0.5%	90	0.45	7.58	6.51	0.86
	1.0%	30	0.30	6.45	6.38	0.99
	1.0%	60	0.60	6.89	6.64	0.97
	1.0%	90	0.90	7.56	6.9	0.92
	1.5%	30	0.45	6.89	6.51	0.95
	1.5%	60	0.90	7.01	6.9	0.99
	1.5%	90	1.35	7.58	7.29	0.97

Apart from the comparision between the split tensile strength of different concrete mix, another relationship has been proposed between the split tensile strength and the cylinder compressive strength. Table.5.28 shows the proposed relations between split tensile and cylinder compressive strength of different concrete mixes.

Table.5.28 Relation between Split Tensile and Compressive Strength

Concrete Type	**Relationship**
Conventional Concrete	$f_{mr} = 0.73\sqrt{f_{cy}}$
Steel Fibre Concrete	$f_{mr} = 0.94\sqrt{f_{cy}}$
Nylon Fibre Concrete	$f_{mr} = 0.93\sqrt{f_{cy}}$
Plastic Fibre Concrete	$f_{mr} = 0.90\sqrt{f_{cy}}$
Tyre Fibre Concrete	$f_{mr} = 0.93\sqrt{f_{cy}}$
Coir Fibre Concrete	$f_{mr} = 0.93\sqrt{f_{cy}}$
Sugarcane Fibre Concrete	$f_{mr} = 0.91\sqrt{f_{cy}}$
Rice Husk Fibre Concrete	$f_{mr} = 0.90\sqrt{f_{cy}}$