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# Study of the Shear Behaviour of Steel And Polypropylene Fiber Reinforced Concrete Beams



**A Project Report**

*Submitted by*

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-

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*in partial fulfillment for the award of the degree  
of*

**Master of Engineering  
in  
Structural Engineering**



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**COIMBATORE – 641 006**

**(An Autonomous Institution Affiliated to Anna University, Coimbatore.)**

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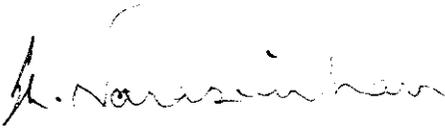
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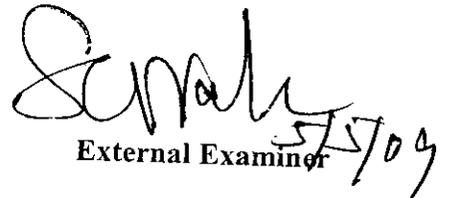
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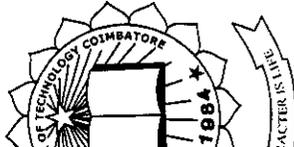
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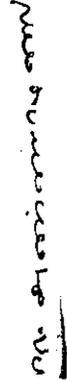
The Study of the shear behaviour of steel and  
polypropylene fiber reinforced concrete beams



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## ABSTRACT

This thesis presents a series of tests for characterizing the structural behavior of fiber reinforced concrete beams subjected to shear loading. The experimental program involves two types of fibers, steel fiber and a polypropylene fiber. As a reference, plain concrete and conventionally reinforced concrete specimens have also been casted and tested in the laboratory as per ASTM standards. The ultimate shear carrying capacities of the beams are calculated. The study confirms that the shear crack resistance of the material is greatly enhanced by the fibers. Fibers reduced the crack width to approximately a fifth of that in beams with stirrups. The use of steel fibers raises the ductility and fracture energy of concrete. Addition of steel fibers to concrete improves its post cracking behavior in tension. The shear resistance increased with increasing aspect ratio of fibers and volume fraction of fibers.

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## **LIST OF SYMBOLS AND ABBREVIATIONS**

ASTM - American Standard for Testing Materials

CA - Coarse Aggregate

FA - Fine Aggregate

FRC - Fiber Reinforced Concrete

IS - Indian Standards

PFRC - Polypropylene Fiber Reinforced Concrete

SFRC - Steel Fiber Reinforced Concrete

# CHAPTER 1

## INTRODUCTION

### 1.1 General

Both steel and polymeric fibers have been used to reinforce concrete and consequently increase its toughness and crack resistance. Fiber reinforced concrete can be used in some structural applications with a reduced amount or even without any conventional reinforcement. One application of the fibers is to increase the load-carrying capacity of concrete subjected to shear. Several design methods have been proposed that take into account the increase in shear strength due to fibers (Al-Tann and Al-Feel, 1990; Casanova and Rossi, 1997; Campione and Mindess, 1999; RILEM 2000a; Dupont and Vandewalle, 2000; Italian Guidelines CNR-DT 204/2006, 2006; among others). Each of the methods accounts for the fiber contribution by means of an index based on the toughness of the material. However, each formula uses a different index, obtained from different types of test configurations. Thus the application of the design methods can be difficult. Moreover, most of the design methods and test procedures have been developed only for the evaluation of steel fiber reinforced concrete.

The purpose of this work is to present the results of a study carried out to characterize the structural behavior of FRC beams under shear loading, considering fibers of different materials (steel and polymeric). Further, the study aims to evaluate the ability of predicting the ultimate shear capacity through code provisions or by correlations with results from other tests. At the same time, it is verified whether the design methods for SFRC can be extrapolated to polypropylene fiber reinforced concrete (PFRC).

Extensive research has been made in last 20 years to understand the shear properties of reinforced concrete beams. The reinforced concrete beams are presently designed with the help of certain provisions of different international building codes, but the latest research has shown that most of these are un-conservative for beams with large sizes and lower values of longitudinal reinforcement.

According to experimental results, the shear strength of simply supported beams is significantly affected by the compressive strength of concrete, the ratio of tensile reinforcement, the shear span to depth ratio ( $a/d$ ), and size of beam. In particular shear resistance mechanism start to change at  $a/d$  equal to 2.5. Based on this result, the shear resistance mechanism of slender beams with  $a/d > 2.5$  is usually assumed to be different from that of deep beams with  $a/d < 2.5$ . It has also been observed that the stirrups contribution has also been adversely affected in large beams.

The shear strength of concrete beams mainly depends on the following variables:

- Depth of members or size effect
- Shear span to effective depth  $a/d$  or moment to shear ratio
- Longitudinal Reinforcement or dowel action,
- Axial Force,
- The tensile strength of concrete
- Crushing strength of beam web,
- Yielding of stirrups,
- The aggregates sizes leading to aggregate interlocking
- Failure of Tension chord
- Failure of Stirrups anchorage
- Serviceability failure due to excessive crack width at service load.

The latest research has revealed that reduction in the shear capacity of all beams occur when the longitudinal reinforcement ratio is 1% or less for all sizes of beams and the provisions of different international building codes for the shear design are also not conservative for slender beams with  $a/d$  ratio  $> 2.5$ .

In this research, the shear effect of transverse steel & fibers instead of transverse steel has been studied. Four beams have been tested in this research. The test results and beam failure mechanism for both of beams has also been observed.

When principal tensile stresses within the shear region of a reinforced concrete beam exceed the tensile strength of a reinforced concrete, diagonal cracks develop in the beam,

eventually causing failure. After the formation of the first crack, the brittle nature of concrete cause collapse in the unreinforced beam.

In spite of the numerous research efforts directed at the shear capacity of concrete, there is still great discord conceding the mechanisms that govern shear in concrete. Proposed theories vary radically from the simple 45 degree truss model to the very complex non-linear Fracture mechanics. Yet nearly all the resulting design procedures are empirical or semi empirical at best and are obtained by a regression fit through experimental results.

Nowhere is this lack of understanding more evident than in shear design provisions of the ACI Code (ACI committee 318-1995) which consists of 43 empirical equations for different types of members and different loading conditions. Moreover, there is great discrepancy between design codes of different countries. Many of these codes do not even account for some basic and proven factors affecting the shear capacity of concrete members. Of these factors, much confusion is expressed with regards to the effect of absolute member size on the shear capacity of beam elements. The test results and beam failure mechanism for all four beams has also been observed.

## **1.2 Advantages of Fibers**

- Improved long-term serviceability of the structure or product
- High ductility
- Prevents the occurrence of large crack widths
- Increases matrix tensile strength at high volume percentages of fibers
- Results in saving of expensive mortar, cement and sand
- Reduces crack during plastic and hardening stage
- Reduces water seepages and protects steel in concrete from corroding and Walls from dampening
- Protects corners in precast slabs and concrete flooring
- Increases abrasion resistance by over 40% thereby increasing life of roads, Walkways and floors.
- Reduces pitting of floor

### **1.3 Applications Of Fibers**

- Rock slope stabilization and support of excavated foundations in conjunction with rock and soil anchor systems
- Industrial floorings, road pavements, warehouses, channel linings
- Protect bridge abutments
- Rehabilitation of deteriorated marine structures such as light stations, bulk heads, piers, sea walls and dry docks
- Slip-formed cast-in-place tunnel linings
- RCC & PCC like lintel, beam, column, flooring and plastering walls
- Hollow blocks and precast
- Manhole cover, tanks and tiles

### **1.4 Usage in Indian Projects**

- KRCL-MSRDC tunnels
- Naftha Jakari hydro electric project
- KOL hydro electric project
- Baglihar hydro electric project
- Chamera hydro electric project
- Uri dam
- Sirsisilam project
- Tehri Dam project
- Salal Hydroelectric project
- Ranganadi Hydroelectric project, etc.,

## **1.5 The Mechanism of Shear Resistance in Reinforced Concrete Beams without Web Reinforcement.**

### **1.5.1 The formation of Diagonal Cracks**

In reinforced concrete members, flexure and shear combine to create a biaxial state stress. Cracks form when the principal tensile stresses exceed the tensile strength of the concrete. In a region of large bending moments, these stresses are greatest at the extreme tensile fiber of the member and are responsible for the initiation of flexural cracks perpendicular to the axis of the member.

In the region of high shear force, significant principal tensile stresses, also referred to as diagonal tension, may generated at approximately 45 deg to the axis of the member. These may result in inclined (Diagonal tension) cracks.

The shear failure is one the failure modes of RC structures of which the mechanism is much different from flexural failure. In actual RC structures, there is a combination of forces such as shear and flexural moment, axial force, torsion moment, and their failure modes are very complicated.

The shear failure follows a formation of diagonal cracks. It is brittle failure compared with flexural tension failure. Therefore, in the case of design involving the ductility of structures such as seismic design, this type of failure has to be avoided by assigning the safety factor greater than that for flexural failure.

### **1.5.2 Diagonal Cracking Capacity**

The diagonal tension failure occurs immediately after the diagonal crack is formed. Therefore, the shear stress at the diagonal cracking can be assumed to be the ultimate shear strength in the case of the diagonal tension failure.

From numerous experimental data on the shear strength of RC beams without shear reinforcement, the empirical equation was proposed by Okamura and Hagias in 1980. Based on this equation, the modification has been made to incorporate the size effect directly in 1986. This modification was proposed by Niwa and Okamura. This revised equation has been adopted into JSCE Shear Design Specification.

### **1.5.3 Equilibrium in shear span of a beam**

In the case of simply supported beam subjected to two-point loading, the moment and shear distribution is such that the moment is constant in the mid span and in two side spans, shear force is constant. These two sides are called “shear span”. For the elastic beam, the flexure stress  $s$ , shear stress  $t$ , and the principal tensile stress  $s_1$  are determined according to the beam theory. Since concrete material is weak in tension, the magnitude and direction of principal tensile stresses are important. At the location of zero shear stress, i.e., the extreme tensile fiber, the principal tensile stress takes the horizontal direction. At the point of zero normal stress, i.e., the neutral axis, the principal tensile stress is equal to shear stress, and its direction is 45 degrees with respect to the member axis.

The internal and external forces that maintain equilibrium for this free body, bounded on one side by a crack, can be identified. It may be seen that the total external transverse force  $V$ , is resisted by the combination of

- A shear force across the compression zone  $V_c$ ,
- A dowel force transmitted across the crack by the flexural reinforcement  $V_d$ ,
- The vertical components of inclined shearing stresses  $V_a$  transmitted across the inclined crack by means of interlocking of the aggregate particles.

#### **1.5.4 Shear stress at the formation of diagonal crack.**

In a RC beam without shear reinforcement under shear force, once diagonal crack is formed, the beam will fail very suddenly. However, the nominal shear stress at the formation of diagonal crack cannot be obtained by the elastic theory, because it involves many factors such as concrete strength, shear span-effective depth ratio ( $a/d$ ), longitudinal reinforcement ratio, effective depth, etc. After a flexural crack occurs, the shear stress along the crack plane is considered to be resisted by the following effects.

- Direct shear resistance in the flexural compression zone.
- Aggregate interlocking along the crack surface
- Dowel action of longitudinal steel.

#### **1.5.5 Aggregate interlock along crack plane**

Along the diagonal crack of concrete, the shear transfer due to the effect of aggregate interlock can be expected. This effect is especially large, when the crack width is small and concrete strength is high. Since the crack width is proportional to the stress in steel which depends on the longitudinal reinforcement ratio, as the reinforcement ratio increases, the effect of aggregate interlock becomes larger.

The above discussion is based on the assumption of the same ratio of the dimension of section to the maximum size of coarse aggregates. The effect of aggregate interlock depends on the relation between the sectional dimension and the aggregate size, and for the same aggregate size, the effect of aggregate interlock on the small RC section is more pronounced than that in large section, Since the maximum size of coarse aggregates in ordinary RC beams practically does not change even when the dimension of the section is increased, the nominal shear strength of large beams tends to decrease. This is a classical explanation for the size effect in the shear strength.

### **1.5.6 Dowel action of longitudinal reinforcement.**

A part of the shear force can be transferred by the dowel action of longitudinal reinforcement. The main factors influencing this action are flexural rigidity of longitudinal reinforcement and flexural rigidity of surrounding concrete. Actually, there are additional factors involving this effect such as the number and arrangement of longitudinal reinforcement, spacing of flexural cracks, etc. However, and the contribution of each factor has not been formulated so far. At present, the dowel action is represented by using the reinforcement ratio and concrete compressive strength.

## **1.6 The Mechanism of Shear Resistance in Reinforced Concrete Beams without Web Reinforcement**

### **1.6.1 The role of web reinforcement**

The inclusion of web reinforcement such as stirrups does not change fundamentally the previously described mechanism of shear resistance. The concrete cantilevers, which are the principle elements of beam mechanism, will act as tied cantilevers. In addition to the force  $\Delta t'$  resisted by the combination of aggregate interlock, dowel, and flexural action of the cantilevers, another bond force  $\Delta t'$  can be sustained by what is traditionally termed "truss action".

The presence of stirrups is beneficial to beam action in a number of other aspects as well. Stirrups contribution to the strength of the shear mechanisms by the following means:

- Improving the contribution of the dowel action. A stirrup can effectively support a longitudinal bar that is being crossed by a flexural shear crack close to stirrups.
- Surprising flexural tensile stresses in the cantilever blocks by means of the diagonal compression force  $c_d$  resulting from truss action.

- Limiting the opening of the diagonal cracks within the elastic range, thus enhancing and preserving shear transfer by aggregate interlock.
- Providing the confinement, when the stirrups are sufficiently closely spaced, thus increasing the compression strength of localities particularly affected by the arch action.
- Preventing the breakdown of bond when splitting cracks develop in anchorage zones because of dowel and anchorage forces.

### **1.7 Shear Resistant Mechanisms of FRC Beams**

In a FRC beam, it seems logical to consider that, during failure; the shear force transferred along the web, due to aggregate interlocking is increased since the crack opening is reduced by the fiber action. Also, due to the toughness of FRC, tensile stresses can be resisted across the crack (through fiber bridging) once the shear crack appears; giving rise to a vertical force component that contributes to balancing the shear force acting on the beam.

For the shear design of beams, the European and American Codes consider that the ultimate shear capacity,  $V_u$ , is the sum of the contributions of the concrete and the stirrups,  $V_{cd}$  and  $V_{wd}$ , respectively:

$$V_u = V_{cd} + V_{wd}$$

Accordingly, several authors (Al-Tann & Al-Feel, 1990; Casanova & Rossi, 1997; Compione & Mindess, 1999; RILEM, 2000a) have suggested the addition of another factor,  $V_{fd}$ , to take into account the fiber contribution:

$$V_u = V_{cd} + V_{wd} + V_{fd}$$

Where  $V_{fd}$  is a function of different parameters. For example, RILEM (2000) defines the contribution of steel fibers to the ultimate shear capacity as:

$$V_{fd} = k_f k_1 t_{fd} b_w d$$

Where  $k_f$  = takes into account the contribution of the flanges of a T-section:

$$K_f = 1 + [h_f/b_w][h_f/d]^n, \quad K_f = 1.5$$

$H_f$  = flange thickness (mm)

$B_f$  = flange width (mm)

$B_w$  = web width (mm)

$N$  =  $B_f - B_w / h_f$  ;

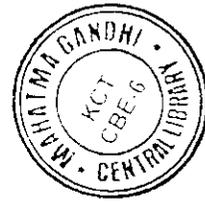
$K_1 = 1600 - d / 1000$   $k_1 = 1$  ; (d in mm);  $k_1$  accounts for the size effect

$t_{fd}$  = design value of the maximum tangential stress due to fibers;

$t_{fd} = 0.12 f$  (N/mm<sup>2</sup>) ; Where  $f$  is the equivalent residual stress.

## 1.8 Objectives

- To investigate the effect of shear span to effective depth “a/d” ratio and contribution of stirrups on the shear capacity of concrete slender beams.
- To investigate the failure mechanism of reinforced concrete beam and Fiber reinforced concrete beams (Steel and Polypropylene).
- To study the shear strength of reinforced concrete beam and Fiber reinforced concrete beams (Steel and Polypropylene).
- To compare the shear strength of reinforced concrete beam and Fiber reinforced concrete beams (Steel and Polypropylene).



## 1.9 Organization of the Thesis

In chapter 1, a general introduction about the ordinary reinforced, Fiber reinforced concrete slender beams and the mechanism of shear resistance in reinforced concrete beam with and without web reinforcement are given. The objectives of the present study are also given.

In chapter 2, literature reviews on the shear strength of reinforced concrete beam and Fiber reinforced concrete beams (Steel and Polypropylene) are given.

In chapter 3, the experimental program was clearly explained. This includes the material properties, testing procedure and experimental arrangement and crack pattern of the beam etc.

In chapter 4, the experimental results are presented. This concludes the main findings of the study and also points out on future work that was necessary for follow up.

## CHAPTER - 2

### LITERATURE REVIEW

**BATSON ET AL. (1972a)**, using mortar beams 4×6×78 in. (100×150×2000), conducted a series of tests to determine the effectiveness of straight steel fibers as web reinforcement in beams with conventional flexural reinforcement. In tests of 96 beams, the fiber size, type, and volume concentration were varied, along with the shear-span-to-depth ratio  $a/d$ , where  $a$  = shear span (distance between concentrated load and face of support) and  $d$  = the depth to centroid of reinforcing bars. (Shear capacity of rectangular beams may be considered a function of moment-to-shear ratio  $a/d$  or  $M/Vd$ .) Third-point loading was used throughout the test program.

It was found that, for a shear-span-to-depth of 4.8, the no fiber beams failed in shear and developed a shear stress at failure of 277 psi (1.91 MPa). For a fiber volume percent of .88, the average shearing stress at failure was 310 psi (2.14 MPa) with a moment-shear failure; for 1.76 volume percent, 330 psi (2.28 MPa) with a moment failure. The later represents an increase of 27 percent over the confiner beams. The shear stress at failure for beams with #3 (9.5 mm diameter) stirrups at (50 mm) spacing in the outer thirds averaged 315 psi (2.17 MPa). All shearing stresses were computed by the equation  $\tau = VQ/Ib$ .

It was found that, for a shear-span ratio decreased and fiber volume increased, higher shear stresses were developed at failure. For example, for an  $a/d$  of 3.6 and a volume

percent of fiber of .88, the shear stress at failure was 444 psi (3.06MPa) with a moment failure; for an a/d of 2.8 and a volume percent of fiber of 1.76, the shear stress at failure was 550 psi (3.79MPa) with a moment failure.

**PAUL AND SINNAMON (1975)** studied the effect of straight steel fibers on the shear capacity of concrete in a series of seven tests similar to those of **BATSON ET AL. (1972a)**. The objective was to determine a procedure for predicting the shear capacity of concrete. Their results agreed closely with **BATSON**, especially for beams with similar a/d ratios.

**WILLIAMSON (1978)**, working with conventionally reinforced beams (305×546×7010 mm), found that when 1.66 percent by volume of straight steel fibers were used in place of stirrups, the shear capacity of the beams was increased 45 percent over a beam without stirrups. Nevertheless, the beams failed in shear. This is consistent with the results of other investigators. When steel fibers with deformed ends were used (1.1 percent by volume), the shear capacity of the beams was increased 45 to 67 percent and the beams failed in flexure.

**WILLIAMSON (1978)** concluded that, based upon the use of steel fibers with deformed ends, steel fibers can increase the shear strength of concrete beams enough to prevent catastrophic diagonal tension failure and to force the beam to fail in flexure. In his report, **WILLIAMSON (1978)** presents an analysis showing that steel fibers can present an economical alternative to the use of stirrups in reinforced concrete design.

Tests of crimped-end fibers have shown considerable increase in the shear capacity of reinforced concrete in other studies. Some of the tests at the New Jersey Institute of Technology (Craig 1983) have shown increases of more than 100 percent. Twelve full-scale test beams with 1.0 and 1.5 percent by volume of (0.5×30mm) long crimped-end fibers was tested with the following span-to-depth ratio:  $a/d = 1.0, 1.5, 2.0, 2.5$  and  $3.0$ . The beams had a (150×300mm) section. The increase in the shear capacity for the 1.0 and 1.5 percent fiber content with  $a/d = 1.5$  were 130 and 140 percent, respectively. Similarly, the increase at  $a/d = 3.0$  was 108 percent for 1.0 volume percent of fiber. The combination of stirrups and fibers showed slow and controlled cracking and better distribution of tensile cracks, and minimized the penetration of shear cracks into the compression zone.

It was found that when fibers with crimped ends were the only shear reinforcement, there was a significant decrease in diagonal tension cracking in beams.

**BOLLANO (1980)** investigated the behavior of steel fibers as shear reinforcement in two-span continuously reinforced concrete beams. These tests indicate the behavior in shear for the common range of  $M/Vd$  ratios for negative moment regions ( $M/Vd = 2$  to  $3$ , equivalent to  $a/d$  for simple beams). It is generally assumed that the  $M/Vd$  concept can be used equally well in simply supported and continuous beams, but this is not entirely true for the beams investigated. The  $a/d$  ratio was  $4.8$  and the  $M/Vd$  ratio was  $3.0$ .

**CRISWELL (1976)** conducted a number of shear tests, all of which demonstrated an increased shear capacity with the use of steel fibers. All of his tests were made with concrete containing 1.0 percent by volume of straight fibers. The results of four shear-friction specimens showed a 20 percent increase in shear strength; bolt pullout tests showed shear strength in excess of 64 percent greater than that for the no fiber concrete.

**SHARMA (1986)** tested 7 beams with steel fiber reinforcement, of which 4 also contained stirrups. The fibers had deformed ends. Based on these tests and those by **BATSON ET AL (1972a)** and **WILLIAMSON and KNAB (1975)**, he proposed the following equation for predicting the average shear stress  $V_{cf}$  in the SFRC beams.

$$V_{cf} = 2/3 f_t' (d/a)^{0.25}$$

$f_t'$  – Tensile strength of concrete obtained from results of indirect tension tests.

$d/a$  – Effective depth-to-shear-span ratio.

**ACI 318-95** code considers the shear capacity of slender reinforced concrete beams without stirrups as the shear stress at which diagonal cracking begins. The shear capacity can be calculated using one of two equations. The first one, ACI 11-3, only considers the compressive strength of concrete and the beam dimensions, while the second, ACI 11-6, also includes the influence of the longitudinal reinforcement. The compressive strength of concrete for both equations is limited to less than 70 MPa.

**ACI 318-99** current design codes including ACI 318, and many researches including **Zsutty**, proposed various empirical shear strength equation, which are defined by functions of the primary design parameter ; the compressive strength of concrete, the ratio of tensile reinforcement,  $a/d$ , and the size of a beam. Although these equations are

convenient for the use because of their simple forms, most of the empirical strength equations do not accurately predict the test result with a wide range of design parameters.

Zsutty's equation was developed in the 1970's using regression analysis of experimental data. It has proven to be relatively accurate in predicting the shear strength of NSC beams. Hence, this equation has become widely used. This equation takes into account the compressive strength of concrete, longitudinal reinforcement ratio, and shear span to depth ratio. Zsutty's equation ignores the effect of the beam depth on the ultimate shear strength.

**European code** calculates the shear capacity of reinforced concrete beams without web reinforcement accounting for the influence of the concrete compressive strength, longitudinal reinforcement ratio, and the size effect.

**CSA** simplified design method is similar to the ACI method except that it neglects the influence of the longitudinal reinforcement and the shear span to depth ratio. It does, however, include a term to account for the size effect for beam depths greater than 300 mm.

**Bezant and Kim** The size effect is very evident in normal-strength concrete. The shallower specimens were consistently able to resist higher shear stresses than the deeper ones developed a theoretical model based on fracture mechanics.

**ASCE-ACI committee 445 and AL-Nahlawi and Wight**, however, the current strut – and – tie model does not accurately predict the strength of slender beam that fail by diagonal tension cracking.

## **CHAPTER 3**

### **EXPERIMENTAL PROGRAMMES**

#### **3.1 INTRODUCTION**

In the present work the experimental program consisted of testing two normal reinforced concrete and two fiber reinforced concrete beams of identical cross section (100×150mm). The span length of the beam was 2000 mm. The mix was designed to resist the cylinder compressive strength of 25 MPa. The major variables were the shear span to effective depth ratio. Fig 3.13 shows that the experimental set up of the beams. Mix proportions of the concrete used for the test are presented in Table 3.4

#### **3.2 PROPERTIES OF CONSTITUENT MATERIALS**

The properties of the constituents used in this experimental investigation are given below.

##### **3.2.1 Cement**

Cement is the most important ingredient in concrete. One of the important criteria for the selection of cement is its ability to produce improved microstructure in concrete. The most commonly used cement is called normal Portland cement, designated as Type 1 by the American Society of Testing and Materials, ASTM. Concrete made with this cement is tested for standard strength after a curing period of 28 days. Normal Portland cement can be used to produce fiber-reinforced concrete. The properties of cement like Consistency, initial setting time, final setting time and specific gravity were studied and the obtained results were as shown in Table 3.1

**Table 3.1 Properties of PPC**

Test	Result obtained	As per IS 4031-1998
Consistency	33	-
Initial setting time	90 minutes	Not less than 30 min.
Final setting time	5 hours	Not more than 600 min.
Specific gravity	3.15	-

**3.2.2 Aggregates**

The aggregates suitable for plain concrete are suitable for FRC. The aggregate are normally divided into two categories, namely, fine and coarse. The grading requirements for fine aggregates used in concrete are listed in ASTM specification C-33. At least 80 percent of the particles should be smaller than 3mm. coarse aggregates can be normal-weight, lightweight, or heavyweight in nature. Fiber-reinforced concretes containing normal-weight or lightweight aggregates have been successfully used in a number of field applications. The properties of Fine aggregate and Coarse aggregate are given in Tables

**Table 3.2 Properties of Fine aggregate**

Test	Result obtained	As per IS 383-1970
Fineness modulus	2.7	Medium sand
Specific gravity	2.6	2.55 minimum
Bulk density Kg/m <sup>3</sup>	1607	-

**Table 3.3 Properties of Coarse aggregate**

<b>Test</b>	<b>Result obtained</b>	<b>As per IS 383-1970</b>
Fineness modulus	5	5 to 7
Specific gravity	2.64	2.6 minimum
Bulk density Kg/m <sup>3</sup>	1580	—

### **3.2.3 Water**

Water is an important ingredient of concrete as it actively participates in the chemical reactions with cement to form the hydration product, calcium-silicate-hydrate (C-S-H) gel. As per Neville (2000), the quantity of water added should be the minimum requirement for chemical reaction of unhydrated cement, as the excess water would end up only in the formation of undesirable voids in the hardened cement paste of concrete.

### **3.2.4 Fiber**

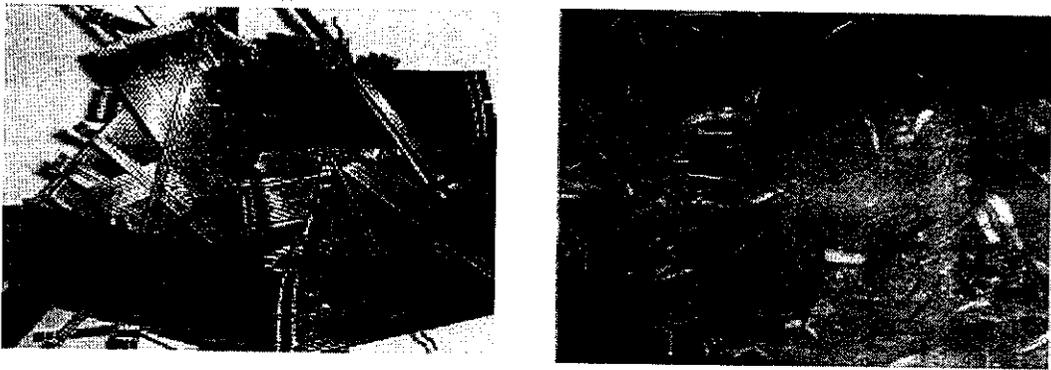
The fibers can be broadly classified as

- Metallic fibers
- Polymeric fibers
- Mineral fibers
- Naturally occurring fibers

Metallic fibers are made of either steel or stainless steel. The polymeric fibers in use include acrylic, aramid, carbon, nylon, polyester, polyethylene and polypropylene fibers. Glass fiber is the predominantly used mineral fiber. Various types of organic and inorganic naturally occurring fibers such as cellulose are also being used to reinforce the cement matrix.

## MATERIALS

A concrete with 25 Mpa compressive strength was considered for the mixes. Two types of fibers were considered, Dramix 80/60 BN hook-ended steel fibers, and polypropylene fibers. In all cases fiber volume was fixed at 0.4%. Table 1 summarizes the cast specimens



**Fig 3.1 Photos of Glued DRAMIX Steel Fiber and Polypropylene Fiber**

### 3.3 MIX DESIGN

In this experimental investigation, 0.4 percentage of fiber volume fraction is used in M25 mix.

#### 3.3.1 Fiber Reinforced Concrete Mixture

Concrete mixture design is vast and generally based on performance criteria. Based on the information given above, some simple guidelines for the design of fiber reinforced concrete.

The wet mixture used was prepared first without the fibers. The slump of the concrete before fiber addition should be (50 to 76 mm) greater than the final slump desired. Normally, the mixture would be prepared using the water-cement ratio found to give the

best results and meeting the specifications of the research. The use of high-range water-reducing admixture can be advantageous, but was not essential. With the mixes operating at normal charging speed, add the individual fibers, ball-free (i.e., as a rain of individual fibers) to the mixer. After all the required fibers were introduced into the mixer, the mixer should be slowed to the rated mixing speed and mixed for approximately 10 to 15 revolutions to obtain the uniform concrete mix.

### 3.3.2 Mixture Proportion

Mixture proportions of fiber reinforced concrete with and without ferrous and non-ferrous fibers with the design compressive strength of M25 Mpa is used in this investigation. PPC conforming to IS 1489-1991 was used. The bulk density of cement was taken as 1450 Kg/m<sup>3</sup>. No industrial by-product was incorporated. The combined aggregate was selected to match the standard grading curves used in the design of Portland cement concrete mixtures. For instance, the aggregate may comprise 704 Kg/m<sup>3</sup> (60%) of 20mm aggregates, 469 Kg/m<sup>3</sup> (40%) of 12.5mm aggregates and 552 Kg/m<sup>3</sup> of fine aggregate to meet the requirement of standard grading curves. The water-cement ratio was 0.45. Initially 0.4 percentage of fiber volume fraction was incorporated in cubes, cylinders and prisms.

**Table 3.4 Design Mixture proportions**

Design Mix	Cement content Kg/m <sup>3</sup>	F.A content Kg/m <sup>3</sup>	Total C.A content Kg/m <sup>3</sup>	60% of 20mm C.A content Kg/m <sup>3</sup>	40% of 12.5mm C.A content Kg/m <sup>3</sup>	Water content Kg/m <sup>3</sup>
M 25	414	552	1173	704	469	186

## **3.4 STRENGTH RELATED PROPERTIES**

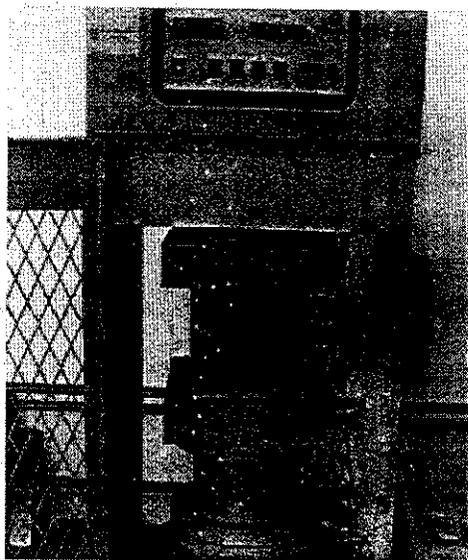
### **3.4.1 Cube Compressive Strength Test**

This clause deals with the procedure for determining the compressive strength of concrete specimens. The testing machine may be of any reliable type, of sufficient capacity for the tests and capable of applying the load at the rate specified in 140 kg/sq cm/min. The permissible error shall be not greater than  $\pm 2$  percent of the maximum load. Tests shall be made at recognized ages of the test specimens, the most usual being 7 and 28 days. The ages shall be calculated from the time of the addition of water to the dry ingredients. At least three specimens, preferably from different batches, shall be made for testing at each selected age.

Specimens stored in water shall be tested immediately on removal from the water and while they are still in the wet condition. Surface water and grit shall be wiped off the specimens and any projecting fins removed. Specimens when received dry shall be kept in water for 24 hours before they are taken for testing. The dimensions of the specimens to the nearest 0.2 mm and their weight shall be noted before testing.

The bearing surfaces of the testing machine shall be wiped clean and any loose sand or other material removed from the surfaces of the specimen which are to be in contact with the compression platens. In the case of cubes, the specimen shall be placed in the machine in such a manner that the load shall be applied to opposite sides of the cubes as cast, that is, not to the top and bottom. The axis of the specimen shall be carefully aligned with the centre of thrust of the spherically seated platen. No packing shall be used between the faces of the test specimen and the steel platen of the testing machine. As the spherically seated block is brought to bear on the specimen, the movable portion shall be rotated gently by hand so that uniform seating may be obtained. The load shall be applied without shock and increased continuously at a rate of approximately 140 kg/sq cm/min until the resistance of the specimen to the increasing load breaks down and no greater load can be sustained. The maximum load applied to the specimen shall then be recorded and the appearance of the concrete and any unusual features in the type of failure shall be noted.

The measured compressive strength of the specimen shall be calculated by dividing the maximum load applied to the specimen during the test by the cross-sectional area, calculated from the mean dimensions of the section. Average of three values shall be taken as the representative of the batch provided the individual variation is not more than  $\pm 15$  percent of the average. Otherwise repeat tests shall be made.



**Fig 3.2 Experimental set up of Compressive Strength Test**

**Table 3.6 Avg. Test Result of Compressive Strength in MPa**

Sl.no	Design mix	Type of fiber added	Percentage of fiber added	Avg. comp. strength in Mpa (after 7 days).	Avg. comp. strength in Mpa (after 28 days).
1.	M 25	Nil	Nil	15.80	25.60
2.	M 25	SF	0.4	16.80	27.00
3.	M 25	PF	0.4	17.24	27.40

### 3.4.2 Tensile Strength Test.

Split tensile strength is an indirect method of finding the tensile strength of concrete. It is easy to perform and gives more uniform strength than the tension test. The specimen is loaded horizontally between the loading surface of the compression testing machine and is loaded until the failure of the cylinder and the experimental setup is as shown in fig 3.3

Split tensile test was conducted at the age of 7<sup>th</sup> and 28th days after casting. Three specimens from each mix were tested and the average values of the test result are shown in Table 3.7.



**Fig 3.3 Experimental set up of Split Tensile Strength Test**

**Table 3.7 Avg. Test Result of Split Tensile Strength in MPa**

Sl.no	Design mix	Type of fiber added	Percentage of fiber added	Avg. tensile strength in Mpa (after 7 days).	Avg. tensile strength in Mpa (after 28 days).
1.	M 25	Nil	Nil	1.92	2.80
2.	M 25	SF	0.4	2.35	3.34
3.	M 25	PF	0.4	2.18	3.10

### 3.4.3 Flexural Strength Test.

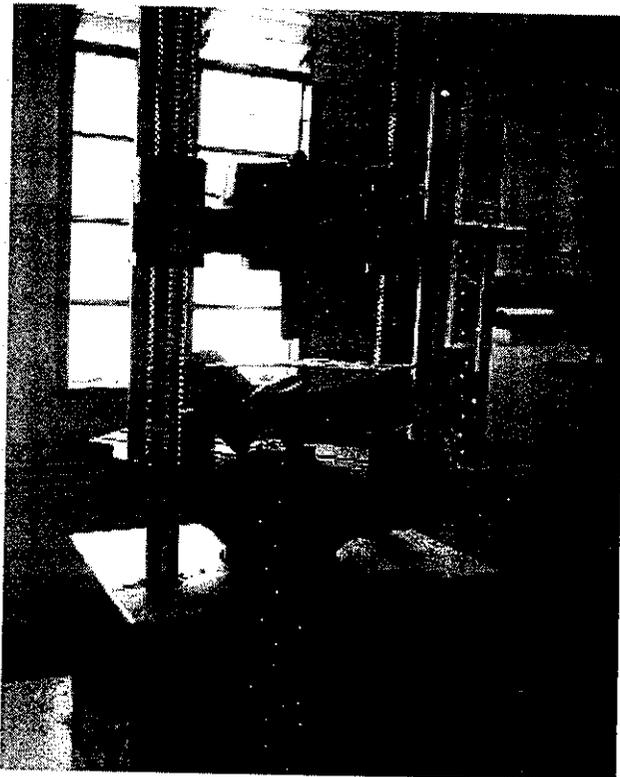
The standard size shall be  $15 \times 15 \times 70$  cm. Alternatively, if the largest nominal size of the aggregate does not exceed 19 mm, specimens  $10 \times 10 \times 50$  cm may be used.

The testing machine may be of any reliable type of sufficient capacity for the tests and capable of applying the load at the rate of 140 kg/min. The permissible errors shall be not greater than  $\pm 0.5$  percent of the applied load where a high degree of accuracy is required and not greater than  $\pm 1.5$  percent of the applied load for commercial type of use. The bed of the testing machine shall be provided with two steel rollers, 38 mm in diameter, on which the specimen is to be supported, and these rollers shall be so mounted that the distance from centre to centre is 60 cm for 15.0 cm specimens or 40 cm for 10.0 cm specimens.

The load shall be applied through two similar rollers mounted at the third points of the supporting span that is, spaced at 20 or 13.3 cm centre to centre. The load shall be divided equally between the two loading rollers, and all rollers shall be mounted in such a manner that the load is applied axially and without subjecting the specimen to any torsional stresses or restraints. One suitable arrangement which complies with these requirements is indicated in Fig. 3.4

Test specimens stored in water at a temperature of  $24^{\circ}$  to  $30^{\circ}\text{C}$  for 48 hours before testing shall be tested immediately on removal from the water whilst they are still in a wet condition. The dimensions of each specimen shall be noted before testing. No preparation of the surfaces is required.

The bearing surfaces of the supporting and loading rollers shall be wiped clean, and any loose sand or other material removed from the surfaces of the specimen where they are to make contact with the rollers. The specimen shall then be placed in the machine in such a manner that the load shall be applied to the uppermost surface as cast in the mould, along two lines spaced 20.0 or 13.3 cm apart. The axis of the specimen shall be carefully aligned with the axis of the loading device. No packing shall be used between the bearing surfaces of the specimen and the rollers. The load shall be applied without shock and increasing continuously at a rate of loading of 400 kg/min for the 15.0 cm specimens and at a rate of 180 kg/min for the 10.0 cm specimens. The load shall be increased until the specimen fails, and the maximum load applied to the specimen during the test shall be recorded. The appearance of the fractured faces of concrete and any unusual features in the type of failure shall be noted.



**Fig 3.4 Flexural strength test on prism**

**Table 3.8 Avg. Test Result of Flexural Strength in MPa**

Sl.no	Design mix	Type of fiber added	Percentage of fiber added	Avg. Flexural strength in Mpa (after 7 days).	Avg. Flexural strength in Mpa (after 28 days).
1.	M25	Nil	Nil	2.45	3.48
2.	M25	SF	0.4	3.00	4.28
3.	M25	PF	0.4	2.72	3.90

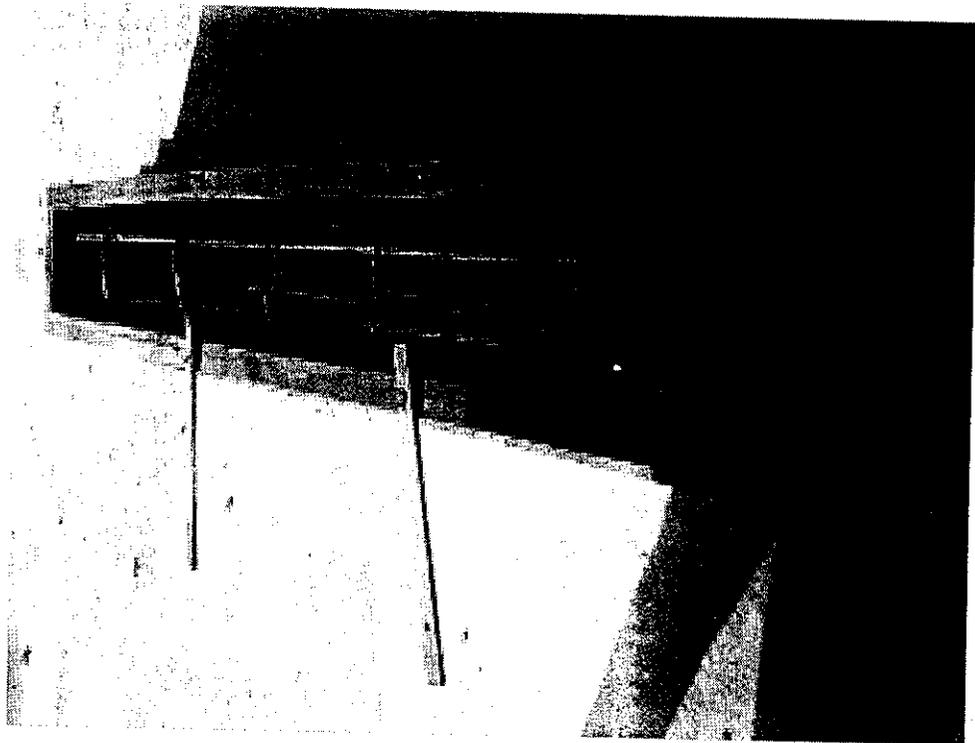
**Table 3.9 Casted Specimen Details**

Type of concrete	No of cubes (150×150×150 mm).	No of cylinders (150×300 mm).	No of prism (100×100×500mm).	No of beams (150×250×2100mm).
Ordinary R.C	6	6	6	1 with Stirrups & 1 with minimum Stirrups.
SFRC	6	6	6	1 with minimum Stirrups.
PFRC	6	6	6	1 with minimum Stirrups.

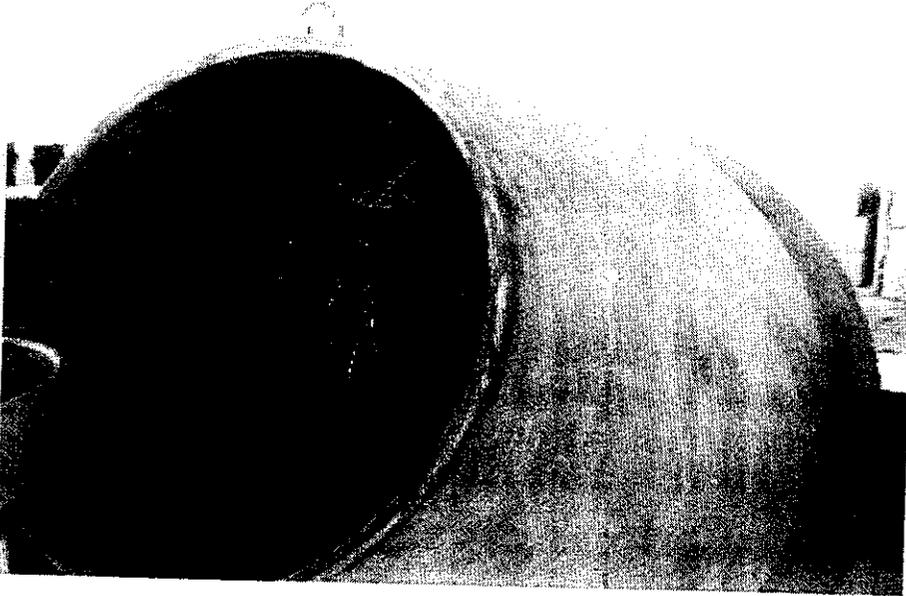
### 3.5 CASTING AND PREPARATION OF TEST SPECIMENS

All the reinforced beam specimens were cast at the structural laboratory. The raw materials for concrete mixes already described in the previous section which is mixed by using a rotary mixer. The wooden mould were prepared and lubricated with oil before the concrete was poured. The reinforcement bars were cut to the required lengths. The longitudinal bars and stirrups were secured to each other at correct spacing by means of binding wires. A mixing time of 3 to 5 minutes was given to ensure uniform mixing. Then the steel rebar's were tied and put into the mould. The specimens were remolded

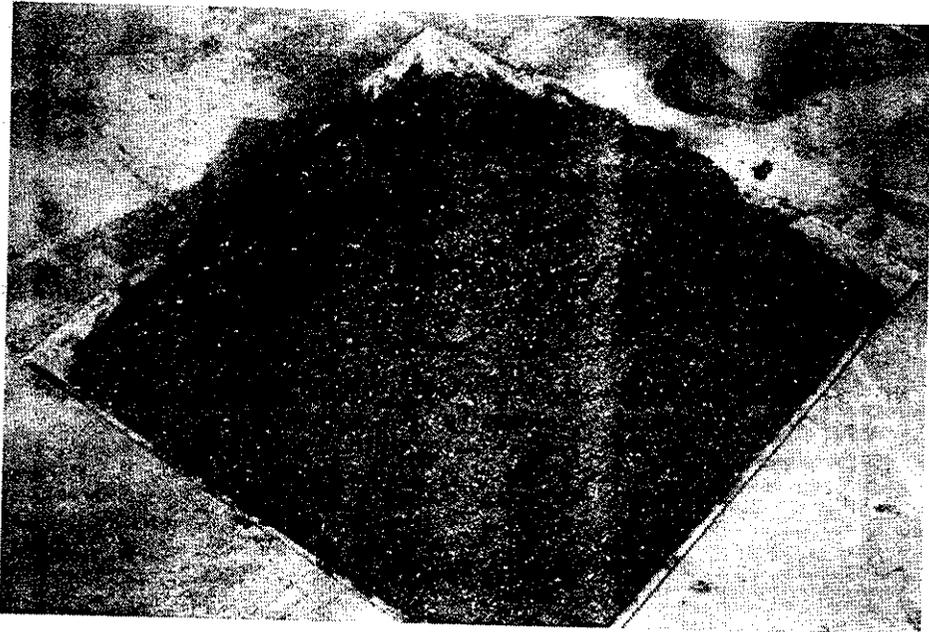
after 24 hours and cured for 28 days using gunny bags. After curing period, the beams were kept for 24 hours in a dry state. After drying they were cleaned with sand paper to remove all grit and dirt. Then all the specimens were prepared by white washing on all sides. White washing was done to facilitate easy detection of crack propagation.



**Fig 3.4 Beam mould with reinforcement.**



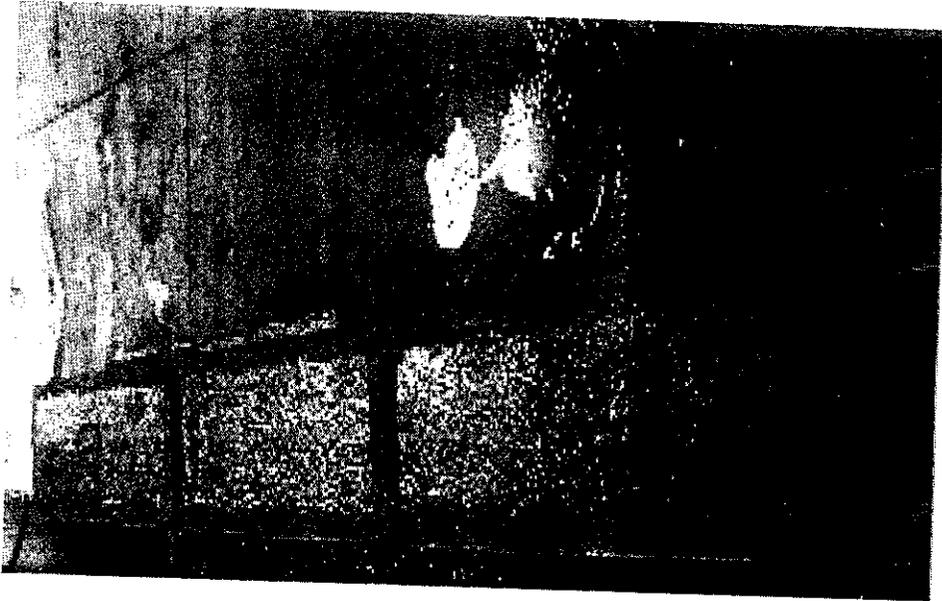
**Fig 3.5 Addition of Fibers into the concrete**



**Fig 3.6 Mixed concrete.**



**Fig 3.7 Concrete Vibration process.**



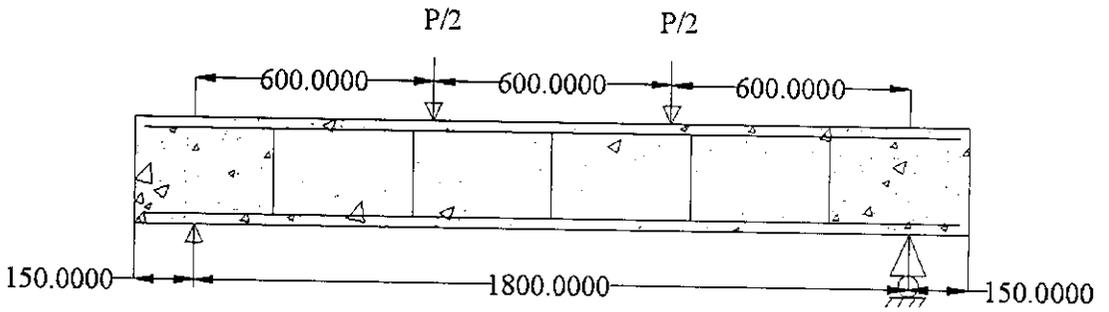
**Fig 3.8 Finished stage of beam.**



**Fig 3.9 Curing process (with gunny bags).**

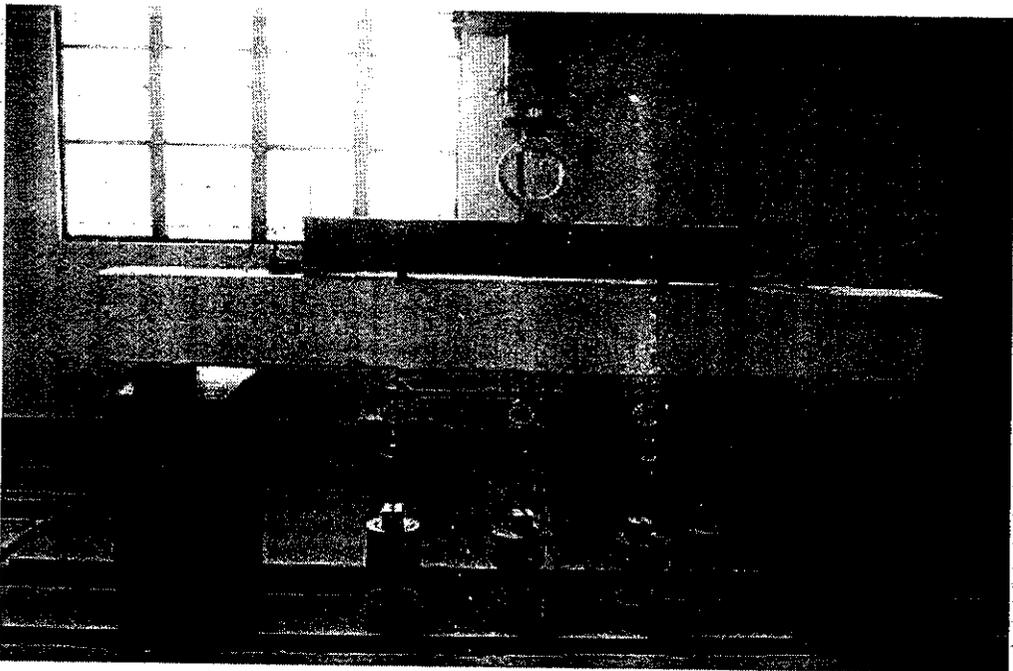
### **3.6 EXPERIMENTAL SET UP**

All the four beams were tested at the age of 28 days in a two point loading conditions, with load applied to the test beam as the two equal concentrated loads by means of a steel beam with shear-span to effective depth ratio of 2.67 as shown in Fig 3.12. Two point loads were applied to the beams by hydraulic testing machine up to the failure. The deflection was measured at the three points using dial gauge, one at the mid span and the other under the loading points. The loads were applied in small increments and at every increment of loading, the deflection was recorded. The load and deflection values obtained for all fiber reinforced concrete beams and reinforced concrete beams are presented in chapter 4.



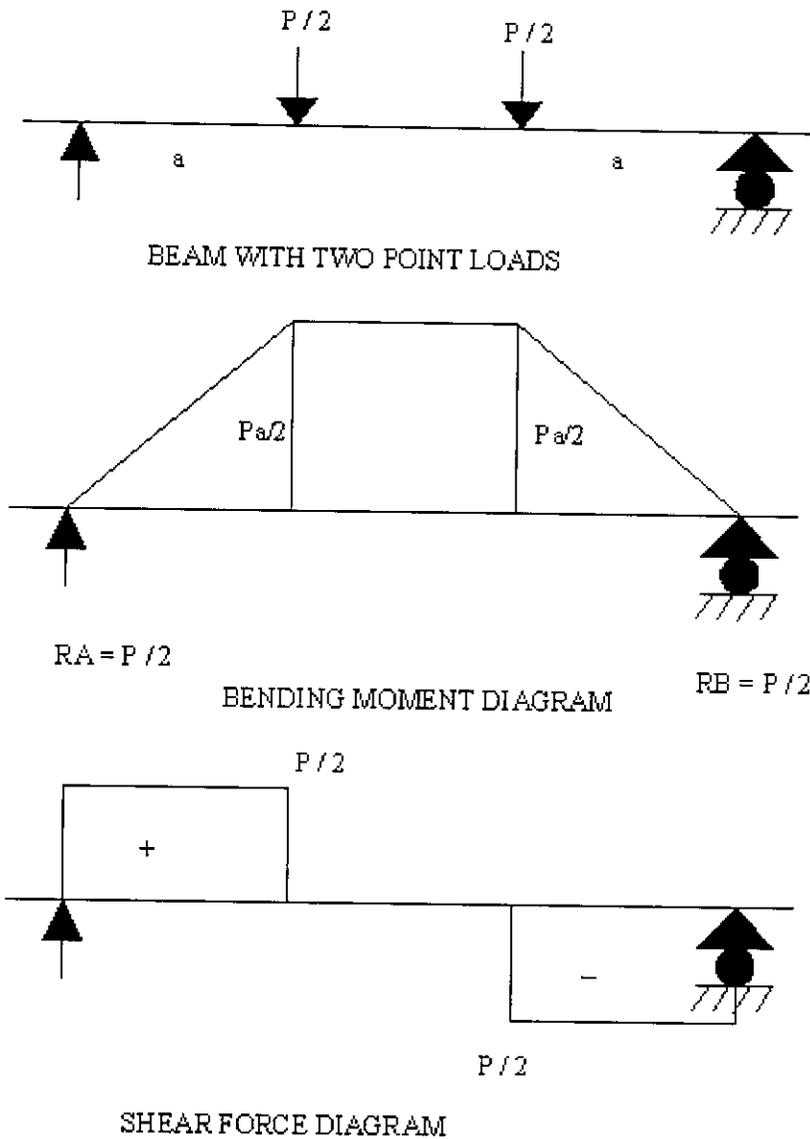
TEST SET-UP.

**Fig 3.11 Experimental set-up of beam (Two point loading).**



**Fig 3.12 Experimental set-up of beam with loading frame in laboratory.**

### 3.7 SHEAR FORCE AND BENDING MOMENT DIAGRAM



**Fig 3.13 Shear force and bending moment diagram for beam with two Point loads.**

## CHAPTER 4

### TEST RESULTS AND DISCUSSIONS

The structural-scale beams were tested under shear loading through the two-point load configuration, with a shear span to depth ratio equal to 2.67. The smaller anchorage length provided for the longitudinal reinforcement did not allow the development of arch effect, which caused failure by debonding of the flexural reinforcement in the case of beams without conventional shear reinforcement.

Fig.6 shows the typical load-deflection responses, where the shear ductility increase induced by fiber action is appreciable. In SFRC beams, the maximum load is higher by approximately 40% of that of RCC Beam with minimum stirrups, with increase the deflection at maximum load. An increase of ductility under shear loading can be clearly observed for all FRCs with respect to RCC Beam with minimum stirrups, indicated by the significant increase in the deformability of the elements at maximum load.

Nevertheless, the load carrying capacity of beams reinforced with fibers is always lower than that of the beams with the same volume of steel in the form of stirrups. This is attributed to the distributed nature of the fiber reinforcement where only some fibers are favorably oriented to resist cracking, whereas all the stirrups actively prevent crack opening.

## 4.1 EXPERIMENTAL RESULTS

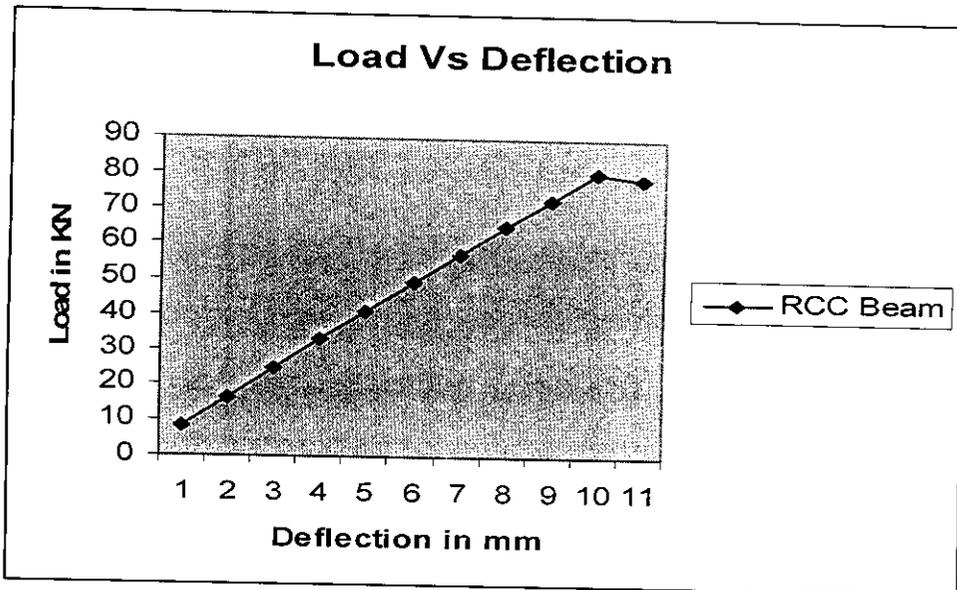
### 4.1.1 Load and Deflection for RCC Beam

The deflection was measured at three points using the dial gauge, one at the mid span and other two at one-third point from the support. The deflection increased according to the load increases. The maximum of 9 mm deflection was obtained for RCC Beam-1, which is for shear span to effective depth ratio of 2.67. The test result of RCC Beam-1 is presented in Table 4.1. The graph drawn for RCC Beam-1 is shown below.

Compared to all other beams the shear load carrying capacity of RCC beam with stirrups is maximum, up to the maximum load carrying capacity the Load Vs Displacement relationship is linear. After that the load and displacement is reducing gradually and then failure occur in the beam. The area under the Load Vs Displacement curve is maximum, so the toughness and the shear carrying capacity is maximum for RCC beam.

**Table 4.1 Load and Displacement values for Reinforced Concrete Beam.**

Sl.no.	Load in KN.	Displacement in mm (at point 1)	Displacement in mm (at centre)	Displacement in mm (at point 2)
1.	8.24	0.27	0.29	0.27
2.	16.45	0.64	0.65	0.60
3.	24.67	0.97	0.98	0.90
4.	32.94	1.40	1.46	1.31
5.	41.00	2.24	2.40	2.12
6.	49.30	3.0	3.24	2.86
7.	57.40	3.69	4.01	3.56
8.	65.15	4.72	5.10	4.55
9.	72.73	6.98	8.00	6.85
10.	80.301	7.50	8.62	7.20
11.	78.50	7.90	9.00	7.60



**Fig 4.1 Load Deflection diagram for the RCC beam.**

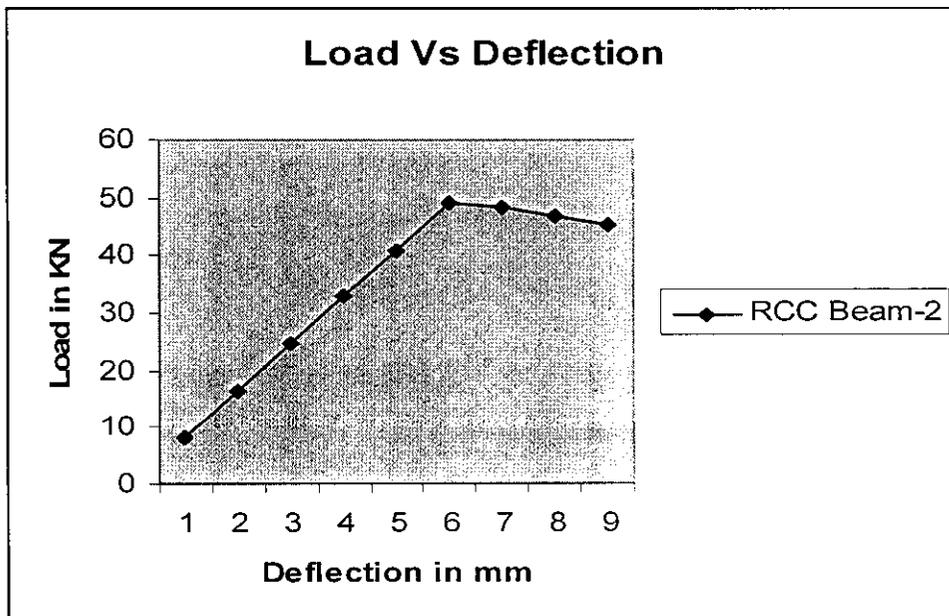
#### **4.1.2 Load and Deflection for RCC Beam with minimum stirrups**

The deflection was measured at three points using the dial gauge, one at the mid span and other two at one-third point from the support. The deflection increased according to the load increases. The maximum of 6.71 mm deflection was obtained for RCC Beam-2, which is for shear span to effective depth ratio of 2.67. The test result of RCC Beam-2 is presented in Table 4.2. The graph drawn for RCC Beam-1 is shown below.

Compared to all other beams the shear load carrying capacity of RCC beam with minimum stirrups is minimum, up to the maximum load carrying capacity the Load Vs Displacement relationship is linear. After that the load and displacement is reducing gradually and than shear failure occur in the beam, which is shown in the Fig 4.3

**Table 4.2 Load and Displacement values for Reinforced Concrete Beam with minimum stirrups.**

Sl.no.	Load in KN	Displacement in mm (at point 1)	Displacement in mm (at centre)	Displacement in mm (at point 2)
1.	8.24	0.43	0.43	0.35
2.	16.45	0.77	0.78	0.65
3.	24.67	1.20	1.26	1.06
4.	32.94	1.75	1.93	1.66
5.	41.00	2.70	3.05	2.60
6.	49.30	3.46	3.95	3.33
7.	48.2	4.25	4.75	4.10
8.	47.0	5.34	6.00	5.25
9.	45.5	5.90	6.71	6.08



**Fig 4.2 Load Deflection diagram for RCC beam-2**



**Fig 4.3 Shear crack on Reinforced Concrete Beam with minimum stirrups**

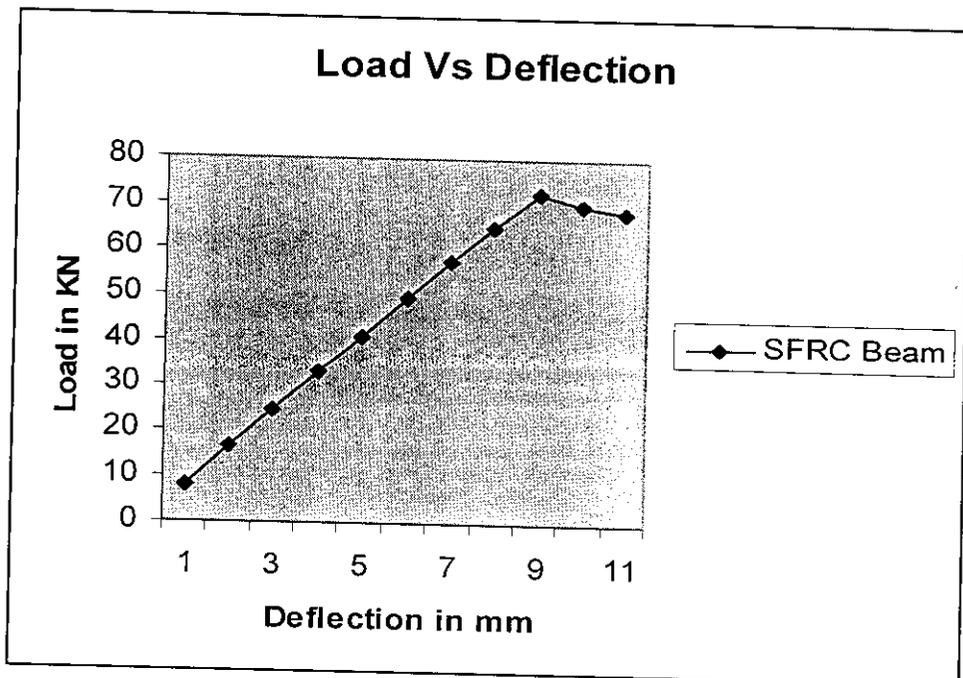
#### **4.1.3 Load and Deflection for SFRC Beam**

The deflection was measured at three points using the dial gauge, one at the mid span and other two at one-third point from the support. The deflection increased according to the load increases. The maximum of 10.25 mm deflection was obtained for SFRC, which is for shear span to effective depth ratio of 2.67. The test result of SFRC is presented in Table 4.3. The graph drawn for SFRC is shown below.

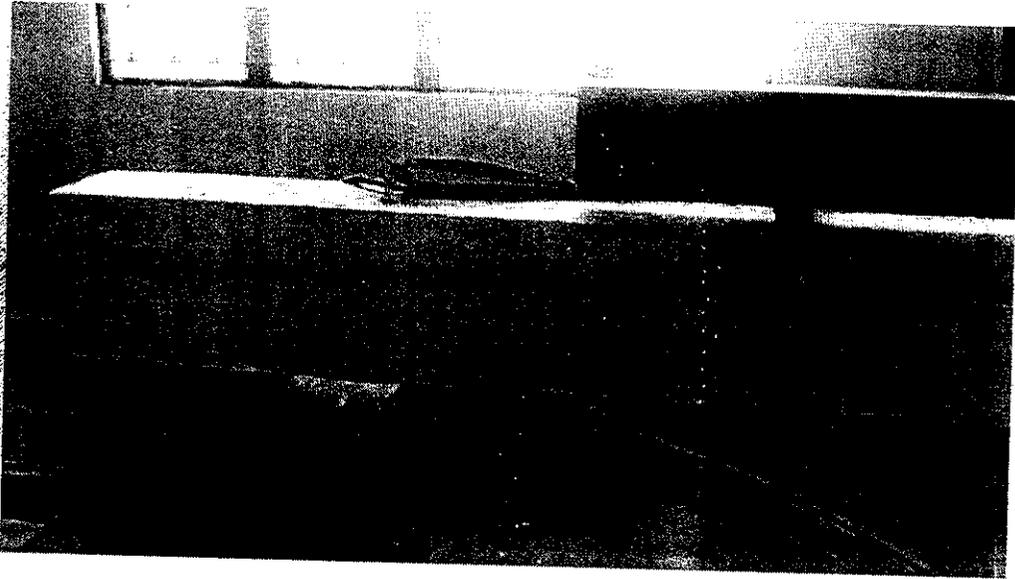
Compared to PFRC & RCC Beam-2 the shear load carrying capacity of SFRC beam is maximum, up to the maximum load carrying capacity the Load Vs Displacement relationship is linear. After that the load and displacement is reducing gradually and than failure occur in the beam. Due to incorporation of steel fibers in concrete the length and width of the crack is reduced. The cracking pattern of SFRC beam is shown in Fig 4.5

Table 4.3 Load and Displacement values for SFRC beam.

Sl.no.	Load in KN	Displacement in mm (at point 1)	Displacement in mm (at centre)	Displacement in mm (at point 2)
1.	8.24	0.36	0.37	0.36
2.	16.45	0.82	0.84	0.78
3.	24.67	1.07	1.12	1.03
4.	32.94	1.75	1.90	1.70
5.	41.00	2.29	2.50	2.22
6.	49.30	2.94	3.20	2.85
7.	57.40	3.81	4.25	3.68
8.	65.15	4.55	5.07	4.39
9.	72.73	5.55	6.20	5.40
10.	70.00	6.28	7.10	6.25
11.	68.50	8.00	10.25	8.25



**Fig 4.4 Load Deflection diagram for SFRC beam**



**Fig 4.5 Shear Crack on SFRC Beam**

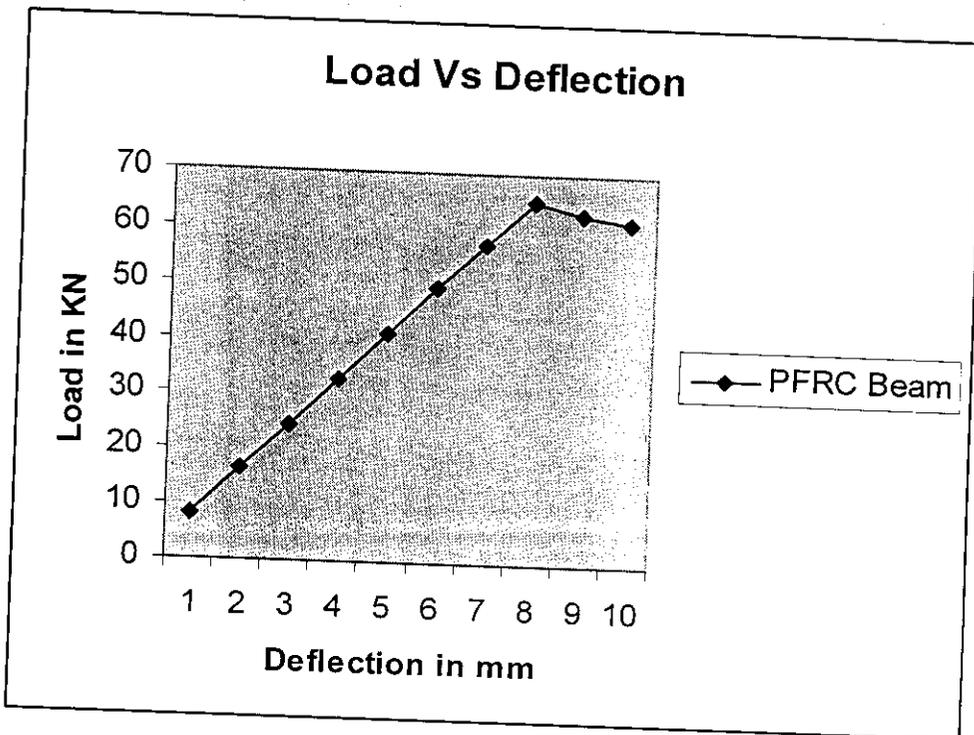
#### **4.1.4 Load and Deflection for PFRC Beam**

The deflection was measured at three points using the dial gauge, one at the mid span and other two at one-third point from the support. The deflection increased according to the load increases. The maximum of 9.80 mm deflection was obtained for PFRC, which is for shear span to effective depth ratio of 2.67. The test result of PFRC is presented in Table 4.4. The graph drawn for SFRC is shown below.

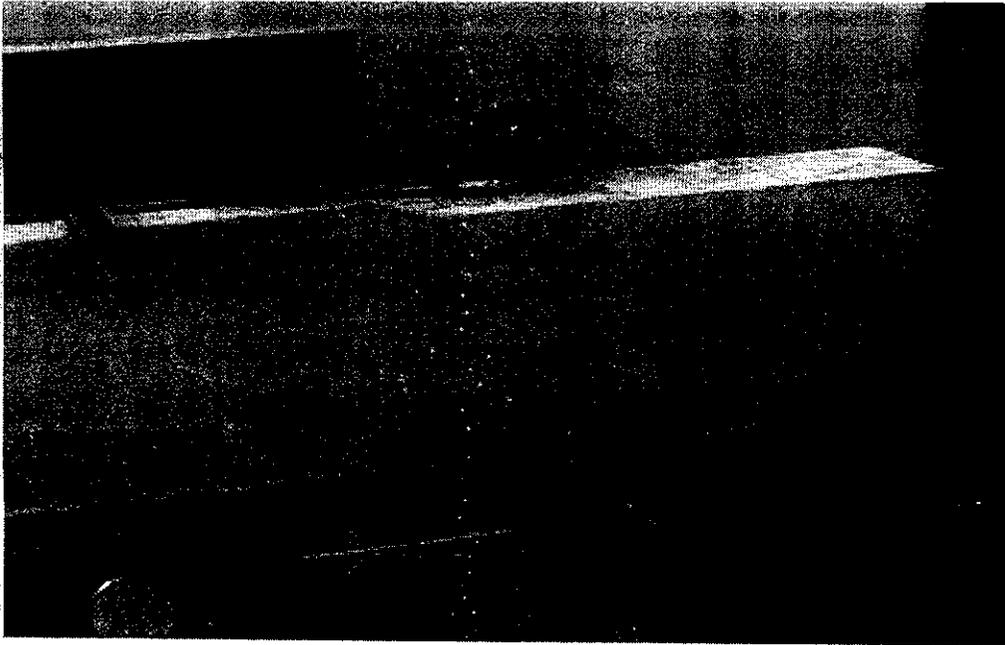
Compared RCC Beam-2 the shear load carrying capacity of PFRC beam is maximum; up to the maximum load carrying capacity the Load Vs Displacement relationship is linear. After that the load and displacement is reducing gradually and than failure occur in the beam. Due to incorporation of polypropylene fibers in concrete the length and width of the crack is reduced. The cracking pattern of PFRC beam is shown in Fig 4.6

**Table 4.4 Load and Displacement values for PFRC beam**

Sl.no.	Load in KN	Displacement in mm (at point 1)	Displacement in mm (at centre)	Displacement in mm (at point 2)
1.	8.24	0.45	0.54	0.39
2.	16.45	0.86	0.82	0.72
3.	24.67	1.09	1.09	1.04
4.	32.94	1.89	2.0	1.76
5.	41.00	2.23	2.43	2.11
6.	49.30	2.60	2.90	2.51
7.	57.40	3.60	4.04	3.51
8.	65.15	4.32	4.87	4.24
9.	63.00	5.20	6.70	5.60
10.	61.50	8.60	9.80	8.80



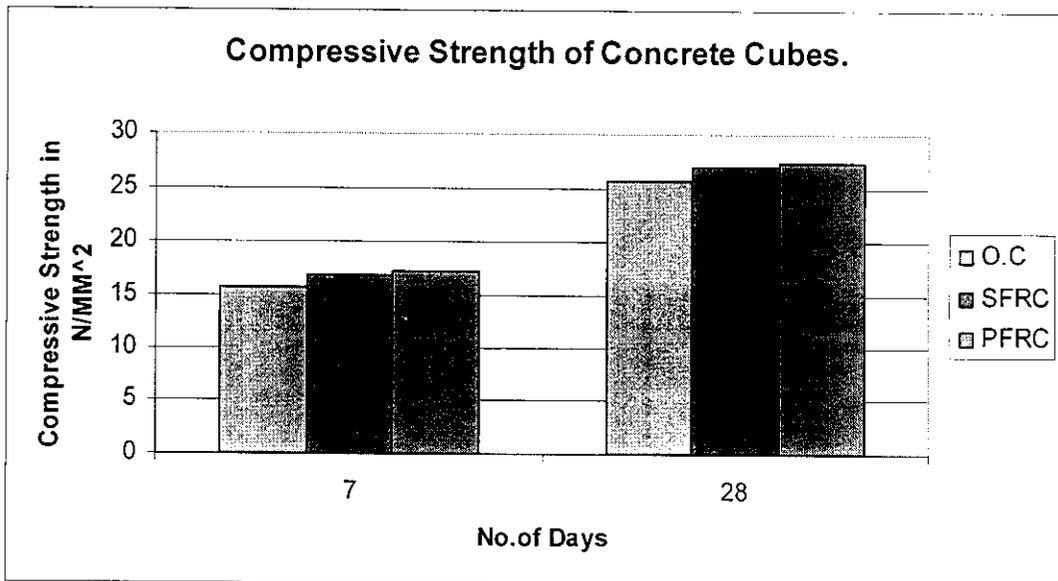
**Fig 4.6 Load Deflection diagram for PFRC Beam**



**Fig 4.7 Shear Crack on PFRC Beam.**

#### 4.1.5 Test Results of Cubes, Cylinders, Prisms and Beams.

Compressive strength of the concrete is marginally improved due to the incorporation of fibers in the concrete. Compressive strength is maximum for SFRC than PFRC.



**Fig 4.8 Compressive Strength of Concrete Cubes**

Both flexural and split tensile strength is maximum for SFRC than compared to normal and PFRC. From the Fig 4.9 & 4.10 we can clearly see that the flexural and tensile strength is maximum for SFRC.

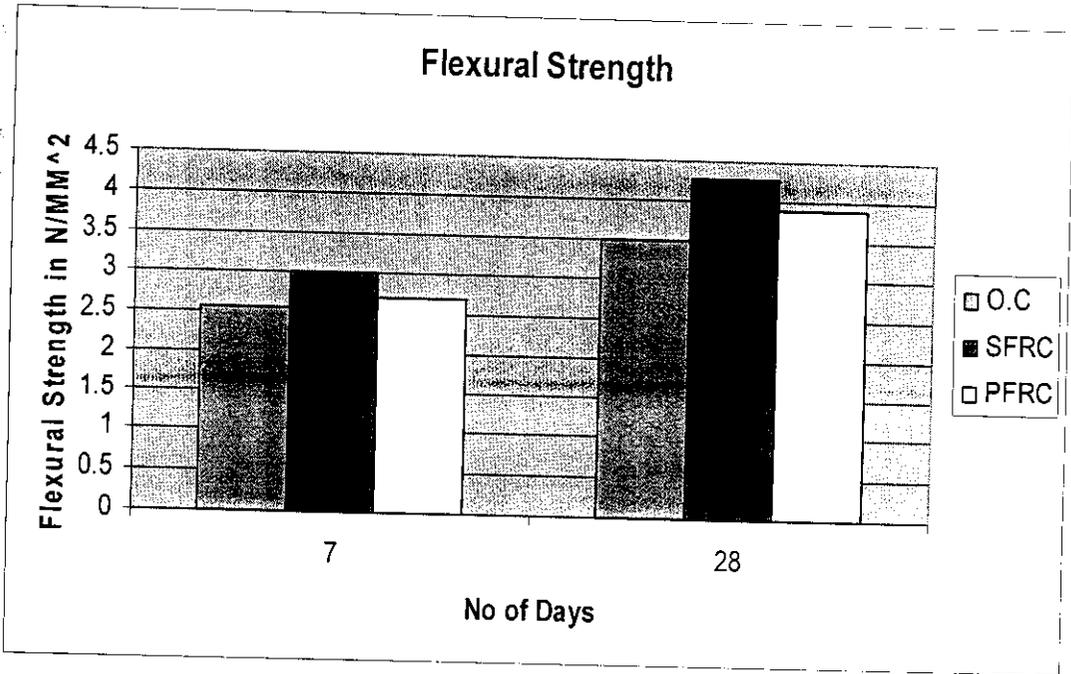


Fig 4.9 Flexural strength of concrete

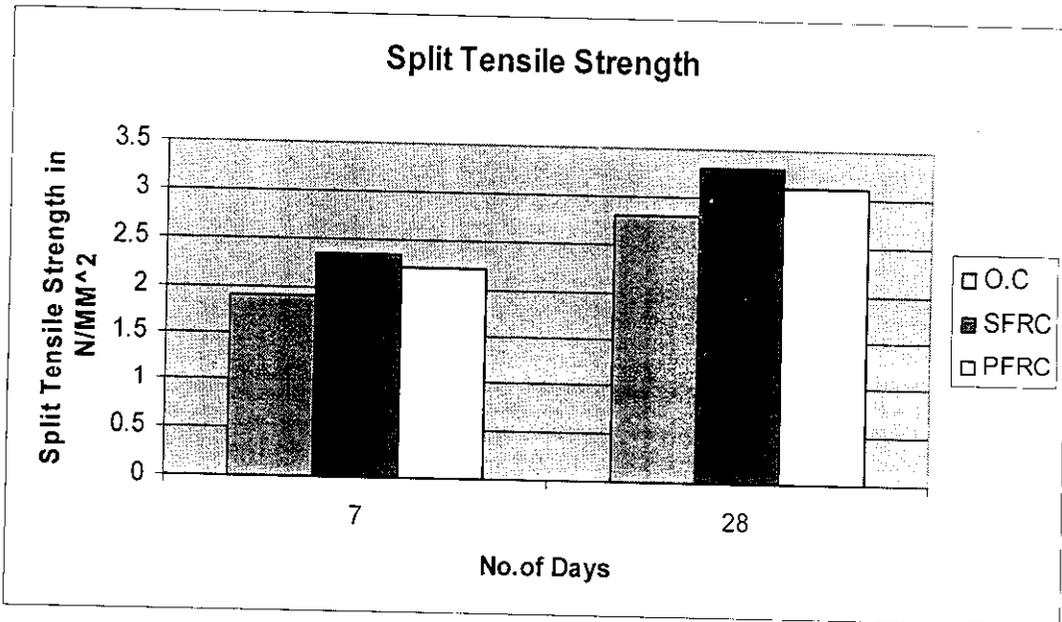
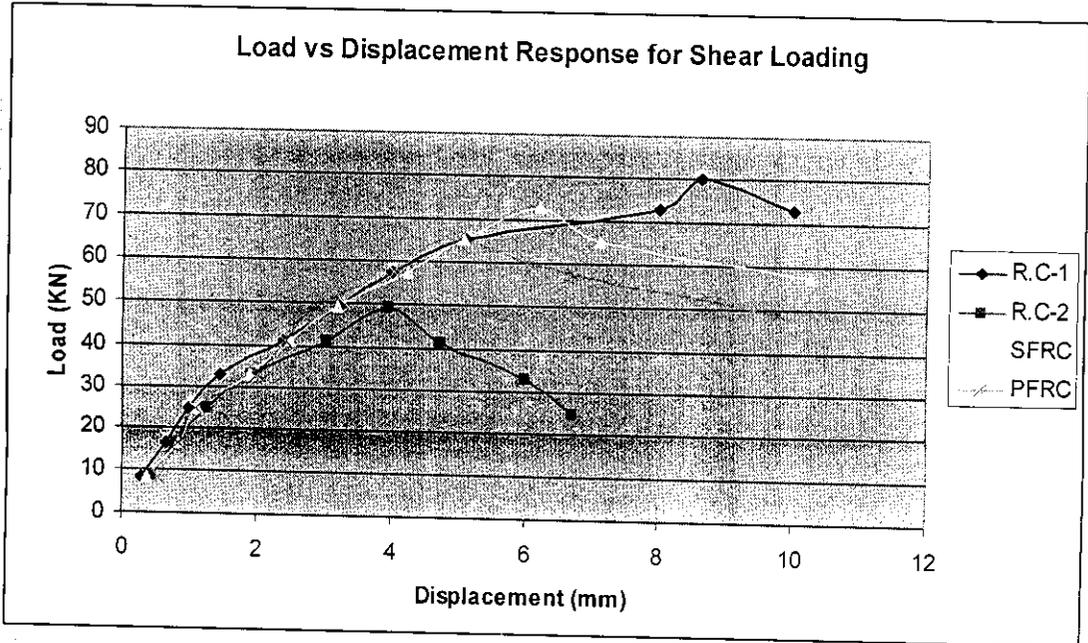


Fig 4.10 Split Tensile Strength of Concrete



**Fig 4.11 Load-Displacement Response under Shear Loading**

From the Fig 4.11 we can clearly note that the area under the curve is maximum for RCC Beam -1. Compared to RCC Beam with minimum stirrups, the area under the curve is maximum for both SFRC & PFRC beams. SFRC beam is having more shear carrying capacity than compared to PFRC beam.

## CHAPTER 5

### CONCLUSIONS

The following conclusions were drawn within the limitations of experimental investigation

- It has been observed that the incorporation of fibers to the mix increases the material toughness both in tension and compression, as represented by the toughness indexes of the ASTM and JSCE standards. The toughness increases results in higher shear strength of the concrete and better deformability, i.e. the deflection at maximum load is significantly higher for FRC beams than plain concrete specimens.
- The compressive strength increase only marginally due to fiber incorporation in concrete.
- First crack occurs earlier in PFRC when compared to SFRC.
- In SFRC beams, the maximum load increased by approximately 20% of the plain concrete.
- The length and width of the crack is reduced due to the incorporation of fibers in the concrete

## **CHAPTER 6**

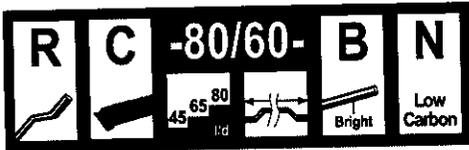
### **SCOPE FOR FUTURE WORK**

- Fiber dosage can be varied and studied for shear test
- Hybrid fiber can be used for the study.
- Dimensions of the beam can be varied and studied.
- Orientation of fibers can be studied

## REFERENCES

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# Dramix®



**Description:** Dramix® fibres are filaments of wire, deformed and cut to lengths, for reinforcement of concrete, mortar and other composite materials. Dramix® RC-80/60-BN is a cold drawn wire fibre, with hooked ends, and glued in bundles.

**Applications:**

- jointless floors
- suspended ground slabs
- jointless floors on vibrocompacted piles
- industrial floors
- slabs on vibro-compacted piles
- liquid tight floors
- overlays
- pavements
- segmental linings
- compression layers
- cellar walls
- precast

**Geometry:**

**Length (l)**  
60 mm

**Diameter (d)**  
0,75 mm

**80** Performance class: 80

**45 65 80 l/d** Aspect ratio (= l/d): 80

4600 fibres/kg

**Tensile strength:**

- on the wire: minimum 1050 N/mm<sup>2</sup>
- low carbon conforms to EN 10016-2 - C9D

**Coating:** None

**Approvals:**

Conforms to **ASTM A820**

Product **Belgium**  
**ATG 04/1857**

The Netherlands  
**22702**

Turkey  
**TS 10513**

Czech Republic  
**C.070-021415**

**Quality System in**  
Belgian, Brazilian, Czech,  
Turkish and Chinese plants

**ISO 9001**  
Dramix

Product **Poland**  
**AT-15-2117/2001**

**Romania**  
**007-01/068-2003**

**Germany**  
**Z-3.71-1745**

**Slovak Republic**  
**1402A/02/0771/1/C/C04**

**Technical data:**

## Recommendations - mixing

**1. General**

- ✓ preferably use a central batching plant mixer
- ✓ recommended maximum dosage:

Max. aggregate size (mm)	Dosage (kg/m <sup>3</sup> )	
	pour	pump
8	60	45
16	50	35
32	35	30

- ✓ a continuous grading is preferred
- ✓ mix until all glued fibres are separated into individual fibres. Fibres don't increase mixing time significantly.
- ✓ if special cements or admixtures are used, a preliminary test is recommended

**2. Fibre addition**

Bags are non-degradable and may not be thrown into the concrete.



**2.1. In batching plant mixer**

- ✓ never add fibres as first component in the mixer
- ✓ fibres can be introduced together with sand and aggregates, or can be added in freshly mixed concrete

**2.2. Truckmixer**

- ✓ run mixer at drum speed: 12-18 rpm
- ✓ adjust slump to a min. of 12 cm (preferably with water reducing agents or high water reducing agents)
- ✓ add fibres with maximum speed of 40 kg/min
- ✓ optional equipment: belt-hoist elevator
- ✓ after adding the fibres, continue mixing at highest speed for 4-5 min. (± 70 rotations)

**2.3. Automatic dosing**

- ✓ Fibres can be dosed from bulk at rates from 0 up to 3,5 kg/sec with a specially developed dosing equipment

## Recommendations - storage

Protect the pallets against rain

Do not stack the pallets on top of each other

Delivered in

non water-soluble bags of 20 kg on pallet 1200 kg      big bag 1100 kg

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Unless an indication to the contrary, all details describe our products in general form.