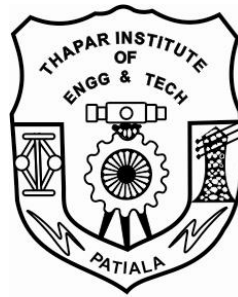


# **RETROFITTING OF SHEAR DEFICIENT RC BEAMS USING FERROCEMENT LAMINATES**

A Thesis

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

**Master of Engineering  
in  
Civil Engineering (Structures)**



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## CERTIFICATE

This is to certify that thesis entitled “**Retrofitting of Shear Deficient RC Beams Using Ferrocement Laminates**”, being submitted by Mr.Anshu Tomar, in partial fulfillment for award of the degree of **Master of Engineering in Civil Engineering (Structures) at Thapar Institute of Engineering and Technology (Deemed University), Patiala** is a bonafide work carried out by him under our guidance and supervision and that no part of this thesis has been submitted for the award of any other degree.

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I know that I will miss the staff from the Civil Engineering Department, T.I.E.T., Patiala for a long long time. They were always helpful and always greeted me with a smile.

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(ANSHU TOMAR)

## ABSTRACT

Reinforced concrete structural components are found to exhibit distress, even before their service period is over due to several causes. Such unserviceable structures require immediate attention, enquiry into the cause of distress and suitable remedial measures, so as to bring the structures back to their functional use again.

This strengthening and enhancement of the performance of such deficient structural elements in a structure or a structure as a whole is referred to as retrofitting. The all important issue to be addressed in retrofitting is life safety .What can be done to prevent collapse of the structure and prevent injury or death to occupants? Some retrofit requirements may try to address only the issue of life safety, while acknowledging that some structural damage may occur.

Ferrocement as a retrofitting material can be pretty useful because it can be applied quickly to the surface of the damaged element without the requirement of any special bonding material and also it requires less skilled labour ,as compared to other retrofitting solutions presently existing. The ferrocement construction has an edge over the conventional reinforced concrete material because of its lighter weight, ease of construction, low self weight, thinner section as compared to RCC & a high tensile strength which makes it a favourable material for prefabrication also.

In the present thesis shear deficient RC beams initially stressed to a prefixed percentage of the safe load are retrofitted using ferrocement to increase the strength of beam in both shear and flexure, the wiremesh is placed at an angle of  $45^0$  to the longitudinal axis of the beam. From the study it is seen that the safe load carrying capacity of rectangular RC elements retrofitted by ferrocement laminates is significantly increased with wiremesh oriented at  $45^0$ .

# CONTENTS

	PAGE NO.
<b>CERTIFICATE</b>	i
<b>ACKNOWLEDGEMENT</b>	ii
<b>ABSTRACT</b>	iii
<b>LIST OF TABLES</b>	vi
<b>LIST OF FIGURES</b>	vii
<b>LIST OF PLATES</b>	viii
<b>LIST OF NOTATIONS</b>	ix
<b>CHAPTER 1. INTRODUCTION</b>	<b>1</b>
1.1 General	1
1.2 Ferrocement	3
1.2.1 Historical Background	5
1.2.2 Specifications for ferrocement	6
1.2.3 Constituent Materials	7
1.2.4 Design Requirements for ferrocement	9
1.2.5 Substitute Materials	10
1.2.6 Potential Applications	11
1.2.7 Other Applications	12
1.2.8 Organization of Thesis	12
<b>CHAPTER 2. REVIEW OF LITERATURE</b>	<b>16</b>
2.1 General	16
2.2 Review of Previous Work	16
2.3 The Need for Present Study	21
<b>CHAPTER 3.0 EXPERIMENTAL PROGRAMME</b>	<b>22</b>
3.1 Introduction	22
3.2 Test Programme	22
3.3 Materials	23
3.3.1 Cement	23
3.3.2 Fine aggregates	23
3.3.3 Coarse aggregates	23

3.3.4 Water	23
3.3.5 Reinforcing Steel	23
3.3.6 Steel Mesh	24
3.3.7 Concrete Mix	24
3.3.8 Mortar Mix	24
3.4 RCC beam design	24
3.5 Casting of composite Beams	24
3.6 Testing Arrangement	25
3.7 Retrofitting of Beams	25
3.7.1 Process of Retrofitting	26
<b>CHAPTER 4.0 RESULTS AND DISCUSSION</b>	<b>32</b>
4.1 Introduction	32
4.2 Testing Methodology	32
4.3 Control Specimen	33
4.4 Comparison of Retrofitted and Control Beam	33
4.4.1 Effect of Ferrocement on beam (60% stressed-R1& R2)	33
4.4.2 Effect of Ferrocement on beam (75% stressed-R3& R4)	34
4.4.3 Effect of Ferrocement on beam (90% stressed-R5& R6)	34
4.4.4 Comparison of Retrofitted beam at Different Stress Level	35
4.4.5 Failure Pattern	35
4.4.6 Comparison of Experimental Crack Characteristics of Retrofitted beams to Control specimen	36
4.4.7 Comparison of Experimental Deflection Characteristics of Retrofitted beams to Control specimen	36
4.4.8 Comparison of Experimental Ultimate Loads.	36
<b>CHAPTER 5.0 CONCLUSIONS</b>	<b>56</b>
<b>References</b>	<b>57</b>

## LIST OF TABLES

<b>Table No.</b>	<b>Description</b>	<b>Page No.</b>
Table 1.1	Different types of Wiremeshes	13
Table 3.1	Physical Properties of Cement used	27
Table 3.2	Physical Properties of Fine Aggregates	27
Table 3.3	Sieve Analysis of Fine Aggregates	28
Table 3.4	Sieve Analysis of Coarse Aggregate (20mm)	28
Table 3.5	Sieve Analysis of Coarse Aggregate (10mm)	29
Table 3.6	Physical Properties Of Coarse Aggregates	29
Table 3.7	Physical Properties Of Steel Bars And Steel Mesh Wires	29
Table 4.1	Comparison of Experimental Crack Characteristics of Beam	41
Table 4.2	Comparison of Experimental Deflection Characteristics of Beam	41
Table 4.3	Comparison of Experimental Ultimate Loads of Beams tested in Shear ( $a/d = 5$ )	42
Table 4.4	Comparison between Beam (R1) and Control Specimen	43
Table 4.5	Comparison between Beam (R2) and Control Specimen	44
Table 4.6	Comparison between Beam (R3) and Control Specimen	45
Table 4.7	Comparison between Beam (R4) and Control Specimen	46
Table 4.8	Comparison between Beam (R5) and Control Specimen	47
Table 4.9	Comparison between Beam (R6) and Control Specimen	48
Table 4.10	Cost Analysis	49
Table 4.11	Ductility Ratio and Energy Absorption	50

## LIST OF FIGURES

Figure No.	Description	Page No.
Figure 2.1	Typical cross-section of Ferrocement laminate	14
Figure 2.2	Different types of wire meshes	15
Figure 3.1	Longitudinal and Cross-Section of Unretrofitted beam	30
Figure 3.2	Longitudinal and Cross-Section of Retrofitted Beam	30
Figure 3.3	Loading System of Beam	30
Figure 3.4	Orientation of Wiremesh at 45 <sup>0</sup>	31
Figure 4.1	20 mm F/C 60 % Stressed (R1)	37
Figure 4.2	20 mm F/C 60 % Stressed (R2)	37
Figure 4.3	20 mm F/C 75 % Stressed (R3)	37
Figure 4.4	20 mm F/C 75 % Stressed (R4)	38
Figure 4.5	20 mm F/C 90 % Stressed (R5)	38
Figure 4.6	20 mm F/C 90 % Stressed (R6)	38
Figure 4.7	20 mm F/C (Different Stress Levels)	39
Figure 4.8	Percentage Increase in Load for Permissible Deflection (15 mm	39
Figure 4.9	Percentage gain in Ultimate Load	40
Figure4.10	Percentage Reduction in Crack Spacing	40

## LIST OF PLATES

<b>Plate No.</b>	<b>Description</b>	<b>Page No.</b>
Plate 4.1	CONTROL BEAM	51
Plate 4.2	SPECIMEN R1 (60 % STRESSED- 20 mm THICK)	51
Plate 4.3	SPECIMEN R2 (60 % STRESSED- 20 mm THICK)	52
Plate 4.4	SPECIMEN R3 (75 % STRESSED- 20 mm THICK)	52
Plate 4.5	SPECIMEN R4 (60 % STRESSED- 20 mm THICK)	53
Plate 4.6	SPECIMEN R5 (60 % STRESSED- 20 mm THICK)	53
Plate 4.7	SPECIMEN R6 (60 % STRESSED- 20 mm THICK)	54

## LIST OF NOTATIONS USED

- a - Shear span, mm
- b - Width ,mm.
- d - Effective depth of beam, mm.
- E - Young's Modulus of elasticity, N/mm<sup>2</sup>
- $\epsilon$  - Strain
- $f_t$  - Mortar tensile strength
- L - Span of beam, mm.
- n - No of Wire mesh layers
- p - Stress, N/mm<sup>2</sup>
- P - Shear load for shear failure (kg)
- $\zeta_{cr}$  - Cracking shear stress
- $\zeta_{ult}$  - Ultimate shear stress
- $V_{fw}$  - Volume fraction of wire mesh
- $V_f$  - Volume fraction of reinforcement

# **CHAPTER 1**

## **INTRODUCTION**

### **1.1 GENERAL**

The strengthening and enhancement of the performance of deficient structural elements in a structure or a structure as a whole is referred to as retrofitting. Repair refers to partial improvement of the degraded strength of a building after an earthquake. In effect, it is only a cosmetic enhancement. Rehabilitation is a functional improvement, wherein the aim is to achieve the original strength of a building after an earthquake. Retrofitting means structural strengthening of a building to a Pre- defined performance level, whether or not an earthquake has occurred. The seismic performance of a retrofitted building is aimed higher than that of the original building.

A survey of existing residential buildings reveal that many buildings are not adequately designed to resist earthquakes. In the recent revision of the Indian earthquake code (IS 1893:2002), many regions of the country were placed in higher seismic zones. As a result many buildings designed prior to the revision of code may fail to perform adequately as per new code. It is therefore recommended that the existing buildings be retrofitted to improve their performance in the event of an earthquake and to avoid large scale damage to life and property.

Getting it right first time is always the aim of designer for a new building or site. However, for various reasons, it is often necessary to improve or restore the seismic resistance of an existing building. Considering the situation for code requirements for wind and earthquakes, an example may be made using the UBC. This model code has been regularly issued in a new edition –lately, every three years.

The 1997 edition followed editions published in 1994, 1991, 1998, 1985, 1982, 1979, 1976, 1973, and 1971 and so on. That means that a particular building constructed in 1972, for example, was probably built confirmation with the requirements of the 1971

UBC, if that was the code jurisdiction. In 1999, therefore the building cannot be expected to have and of the features that were required by 9 upgrades of the code since 1971.

Positioning a building in time with regard to code changes may be used to anticipate what might be required for a code retrofit. If a retrofit is required for a remodeling, as an example, the building must be made to comply to the present code. However, if upgrading is desired simply for its benefit, the needs may be more specifically identified by determining the construction and its design at the time that work was done.

Since codes are clearly documented, the significant changes for a particular building can be relatively easily determined. However, code changes are not the only picture; other factors may be of equal or even greater significances. Of course not all code changes have necessarily resulted in improved resistances to wind or earthquake. Adjustments in structural requirements also reflect evidence from research and changes in the availability and use of building materials and products overtime .Some examples follow;

1. The commonly used grade of steel prior to the World War II was A7 steel, shortly after the war the common grade become A36 steel. As a result, steel structure built after 1945 can be expected to have slightly bouncier floors, skinnier columns, and possible more sidesway (drift) from lateral loads.
2. Starting with the 1963 edition of the ACI code most reinforced concrete structures were increasingly designed by what is now called the strength method versus the older working stress method. In some ways, the strength method permits smaller concrete dimensions and the use of more steel and higher grades of steel. Thus as with steel structures, some increased flexibility is a possibility. But since concrete remains essentially a brittle material, some increased amount of cracking in structural actions can be expected.
3. In the 1960's increasing pressure from the building industry resulted in building code acknowledgements of the usable diaphragm capacities of wood frame surfacing materials other than plywood. Thus, gypsum drywall, plaster, stucco, and fiber panels were permitted for shear walls. This is now viewed as highly questionable by many engineers on the basis of performance of buildings in windstorms and earthquakes in the past two decades notably.

For long range survival we may eventually learn how to make stratifies and indeed whole buildings as well that can withstand the force of many earthquakes (and windstorms, firestorms, floods, etc )

And bounce back quickly with easy repairs or replacements of damaged parts. Like the demolished derby race car, they can be quickly and easily patched up and put back into the race , almost as good as new.

## **1.2 FERROCEMENT**

Ferrocement is a composite material consisting of rich cement mortar matrix uniformly reinforced with one or more layers of very thin wire mesh with or without supporting skeletal steel.

American Concrete Institute Committee 549 has defined ferrocement in broader sense as “a type of thin wall reinforced concrete commonly constructed of hydraulic cement mortar , reinforced with closely spaced layers of continuous & relatively small diameter mesh “.The mesh may be metallic or may be made of other suitable materials. Ferrocement possesses a degree of toughness, ductility, durability, strength & crack resistance which is considerably greater than that found in other forms of concrete construction .These properties are achieved in the structures with a thickness that is generally less than 25 mm, a dimension that is nearly unthinkable in other forms of construction & a clear improvement over conventional reinforced concrete .One can certainly call it a high technology material.

The construction of ferrocement can be divided into four phases:

1. Fabricating the skeletal framing system.
2. Applying rods & meshes.
3. Plastering.
4. Curing phase

Phase 1 & 3 require special skill while phase 2 is very labour intensive. The development of ferrocement evolved from the fundamental concept behind reinforced

concrete i.e. concrete can withstand large strains in the neighbourhood of the reinforcement & magnitude of the strains depends on the distribution & subdivision of the reinforcement throughout the mass of concrete. Ferrocement behaves as a composite because the properties of its brittle mortar matrix are improved due to the presence of ductile wire mesh reinforcement. Its closer spacing of wire meshes (distribution) in the rich cement sand mortar & the smaller spacing of wires in the mesh (subdivision) impart ductility & better crack arrest mechanism to the material..

Due to its small thickness, the self weight of ferrocement elements per unit area is quite small as compared to reinforced concrete elements. The thickness of ferrocement elements normally ranges from 10mm to 40mm whereas in reinforced concrete elements the minimum thickness used for shell or plate element is around 75mm. Low self weight and high tensile strength make ferrocement a favourable material for fabrication. With the distribution of small diameter wire mesh reinforcement over the entire surface, a very high resistance to cracking is obtained & other properties such as toughness, fatigue resistance, impermeability also get improved.

In the past 20 years there has been an increase in the field applications & the laboratory research with this type of construction. The major differences between a conventional reinforced concrete structural element & a ferrocement member can be enumerated as follows:

1. Ferrocement structural elements are normally consist of thin sections with thickness rarely exceeding 25mm. On the other hand conventional concrete members consist of relatively thick sections with thickness often exceeding 100 mm.
2. Matrix in ferrocement mainly consists of Portland cement instead regular concrete consist of coarse aggregate.
3. The reinforcement provided in the ferrocement consists of large amount of smaller dia wire or wire meshes instead of directly-placed reinforcing bars used in reinforced concrete. Moreover, ferrocement normally contains a greater percentage of reinforcement, distributed throughout the section.

4. In terms of structural behaviour, ferrocement exhibits a very high tensile strength & superior cracking performance.
5. In terms of construction, form work is very rarely needed for fabrication.

Metallic meshes are the most common type of reinforcement, Meshes of alkali resistant glass fibres & woven fabric, of vegetable fibres such as jute burlaps & bamboo have also been tried as reinforcement.

### **1.2.1 Historical Background**

The credit of using ferrocement in the present day goes to Joseph Louis Lambot who in 1848 constructed several rowing boats, plant pots, seats & other items from a material he called fercement. Lambot's construction consisted of a mesh or a grid reinforcement made of two layers of small diameter bars at right angle & plastered with cement mortar with a thin cover to reinforcement. Lambot's rowboats were 3.66 m long, 1.22 m wide & 25 mm to 38 mm thick. These were reinforced with grid & wire netting. One of the boats built by him, still in remarkably good condition, is on display in the museum at Brignoles, France.

There was very little application of true ferrocement construction between 1888 & 1942 when Pier Luigi Nervi began a series of experiments on ferrocement. He observed that reinforcing concrete with layers of wire mesh produced a material possessing the mechanical characteristics of an approximately homogenous material capable of resisting high impact. After the second world war Nervi demonstrated the utility of ferrocement as a boat building material.

In 1945, Nervi built the 165 ton Motor Yacht "Prune" on a supporting frame of 6.35 mm dia rods spaced 106 mm apart with 4 layers of wire mesh on each side of rods with total thickness of 35 mm. It weighed 5% less than a comparable wooden hull & cost 40% less at that time.

In 1947, Nervi built first terrestrial ferrocement structure, a storage warehouse of about 10.7 m × 21.3 m size. The strength of the structure was due to the corrugations of the wall & the roof which were 44.45 mm thick.

In 1948 Nervi used ferrocement in first public structure, the Tutrin Exhibition building .The central hall of the building which spans 91.4 m, was built of prefabricated elements Connected by reinforced concrete arches at the top & bottom. of the undulations.

In 1958, the first ferrocement structure – a vaulted roof over shopping centre was built in Leningrad in Soviet Union.

In 1970,a prototype prefabricated ferrocement home was constructed in U.S.A. The house was found much lighter in weight & higher in resistance to dynamic load than the conventionally built brick or block house.

In 1971 a ferrocement trowler named “Rosy in I” was built in Hong Kong.It had an overall length of 26 m & is claimed to be the worlds most longest ferrocement fishing boat.

In 1972 , the US National Academy of sciences through its board on sciences & technology for international Development established an adhoc panel on the utilization of ferrocement in developing countries.

In 1974 , the American Concrete Institute formed committee 549 on ferrocement.

In 1975, two ferrocement aqueducts were designed & built for rural irrigation in China.

In 1976 ,the International Ferrocement Information Centre (IFIC) was founded at Asian institute of Technology ,Bangkok,Thailand). The centre is financed by the United States Agency for International development, the Government of New Zealand & the Internatinal Development Reasearch Centre of Canada.

I n 1978 an elevated metrostation of 43.5 m × 1.6 m in size with continous ferrocement roofing was erected in Leningrad.

In 1979 RILEM (International Union of Testing & research Laboratries of materials & structures) established a Committee (48-FC) to evaluate testing methods for ferrocement.

In 1984 ,ferrocement was used in the construction of a shaking table of large scale earthquake simulation facility at the state university of New York at Buffalo.

Recently, it has been reported that the Chinese have been building ferrocement boats even before world war second.It is estimated that they have built 2000 boats . Most of these boats are 12 m to 15 m long & are mainly used in carrying goods.

## 1.2.2 Specifications For Ferrocement

### (A) Wiremesh Performance

- (1) Wire diameter  $-0.5 \text{ mm} \leq \text{Dia} \leq 2 \text{ mm}$
- (2) Size of mesh openings -  $-6 \text{ mm to } 25 \text{ mm}$
- (3) Volume fraction of reinforcement -Upto 8 % in both directions  
corresponding to up to 630 kg/m as steel per  
cubic meter of mortar.
- (4) Specific surface of reinforcement  $- \text{Upto } 4 \text{ cm}^2/\text{cm}^3$  in both directions.

### (B) Intermediate skeletal reinforcement( if used)

- (1) Type -Wires, Wire fabrics,rods,strands.
- (2) Diameter  $- 3\text{mm} \leq d \leq 10 \text{ mm.}$
- (3) Grid size  $-5 \text{ cm} \leq G \leq 10 \text{ cm.}$

### (C) Typical mortar composition

- (1) Portland cement - Any type depending on application.
- (2) Sand to cement ratio  $-1 \leq S/C \leq 2.5$  by weight.
- (3) Water to cement ratio  $-0.4 \leq W/C \leq 0.5$  by weight.
- (4) Recommendations - Fine sand all passing I.S sieve No 8 & having 5% by weight passing No 100 with a continuous grading curve in between.

### (D) Composite properties

- (1) Thickness  $-6 \text{ mm} \leq t \leq 50\text{mm.}$
- (2) Steel cover  $- 1.5 \text{ mm} \leq c \leq 5 \text{ mm.}$
- (3) Ultimate tensile strength  $-34.5 \text{ N / mm}^2$
- (4) Compressive strength  $-27.6 \text{ to } 68.9 \text{ N / mm}^2$
- (5) Allowable tensile strength  $-10.3 \text{ N/mm}^2$
- (6) Modulus of rupture  $-55.1 \text{ N/mm}^2$
- (7) Cube strength of mortar  $-29.9 \text{ N/mm}^2$

- (8) Young's modulus of wire mesh -  $2 \times 10^5 \text{ N/mm}^2$  for welded wire mesh .  
 -  $1.38 \times 10^5 \text{ N/mm}^2$  for woven mesh.
- (9) Yield strength of wire mesh -  $410 \text{ N/mm}^2$  for welded wire mesh  
 -  $385 \text{ N/mm}^2$  for woven mesh.

### 1.2.3 Constituent Materials

The constituent materials of ferrocement are:

- a) Reinforcing Mesh
- b) Cement
- c) Aggregates
- d) Mixing Water
- e) Admixtures.

#### (a) Reinforcing Mesh

One of the essential components of ferrocement is wire mesh different type of wire mesh shown in Fig. 1.2 are available almost everywhere .these generally consist of thin wires , either woven or welded into the mesh but main requirement is that it must be easily handled and if necessary , flexible enough to be bent around sharp corners.

The function of wire mesh and reinforcing rod is to provide the form and to support the mortar in its green state. In the hardened state, its function is to absorb the tensile stresses on the structure which the mortar on its own would not be able to withstand .

#### (b) Cement

The cement used should conform to IS specifications. There are several types of cements are available commercially in the market of which Portland cement is the most well known & available every where. Cement of Portland variety produced today is satisfactory enough to serve the purpose of ferrocement construction.

#### (c) Aggregates

The most common aggregate used in ferrocement is sand .Sand should comply with IS standard C33 for fine aggregate . Aggregate is the term given to the inert material & it

occupies 60 to 80 % of the volume of mortar. Aggregates to be used for the production of high quality mortar for ferrocement structure must be strong enough, impermeable & capable of producing a sufficiently workable mix with minimum water /cement ratio to achieve proper penetration of wire mesh.

**(d) Mixing water**

The quality of mixing water for mortar has a visual effect on the resulting hardened ferrocement. Impurities in water may interfere with setting of cement & will adversely effect the strength of cause staining of its surface & may also lead to its corrosion of ferrocement. Usually water that is piped from the public supplies is regarded as satisfactory .

**(e) Admixtures**

Admixtures are used to alter or improve one or more properties of cement mortar or concrete. Most of the admixtures are used to improve the workability, to lesson water demand & to prolong mortar setting . Admixtures can be classified into groups according to the effect they are expected to achieve. The commonly used admixtures are :

1. Accelerating admixtures
2. Retarding admixtures
3. Water reducing admixtures
4. Air entraining admixtures.

A new class of water reducing admixtures has emerged during last two decades, known as “super plasticizer”. There are the high range water reducers.

**1.2.4 Design Requirements For Ferrocement**

**(a) Reinforcement**

The total volume of reinforcement in ferrocement ranges 5-8 % by volume of structural elements. The reinforcement used in ferrocement is of two types- skeletal steel & wire mesh.

The skeletal steel comprises relatively large diameter (about 3 to 8 mm) steel rods spaced typically at 70 to 100 mm. Skeletal reinforcement consists of either welded mesh or mild steel bars. The skeletal steel steel frame is made conforming exactly to the shape & geometry of the structure & is used for holding the wire mesh in position & shape of the structure.

The wire meshes are usually 0.5 mm to 1.5 mm in diameter & spaced 5 mm to 25 mm apart & volume of mesh ranges from 0.2 % to 3 % of total volume of structural element. The wire mesh may be galvanized or ungalvanized. If the wire mesh is galvanized & used along with ungalvanized mild steel bars, then chromiumtrioxide at the rate of 100 to 300 parts per million by weight of water should be added in preparing the mortar. This effectively reduces the reaction between the galvanized mesh & the ungalvanized spacing between the layer of mesh .Wire mesh reinforcement should conform to the ASTM standard A-185.Welded wire mesh have a higher Youngs Modulus & hence higher stiffness & less cracking in early state of loading.The minimum yielding strength of wire should be  $450 \text{ N/mm}^2$  for smooth wires &  $480 \text{ N/mm}^2$  for deformed wires.

#### **(b) Matrix**

The matrix is a particulate composite consisting of fine aggregate bonded by the hydratad cement.Generally the matrix with cement to sand ratio of 1:1.5 or 1:2 is desirable for the ferrocement applications. Use of 1:3 ratio or even lean mixes is also reported.

#### **(c) Coatings**

To increase the durability of ferrocement ,it may be protected by surface coatings ,such As Acrylic, Latex,Polyster & cement based paints.

#### **(d) Cover**

Minimum cover to the outer most mesh layer is around 2 mm to 5 mm.

#### **(e) Water /Cement Ratio**

Water /Cement ratio typically ranges between 0.35 & 0.5 by weight.

### 1.2.5 Substitute Materials

#### (a) Bamboo

(*Chembi & Nimityongskol, 1969*) investigated the use of bamboo mesh to replace steel wire mesh in ferrocement water tank .A bamboo cement tank of 6 m<sup>3</sup> capacity was constructed in 1983 .After 5 years they found that the tank has not shown structural defects.Bamboo reinforcement 0.3 m from the top of tank was investigated & was found in good condition.

(*Venketshwarlu & Raj, 1989*) investigated the use of bamboo to replace the skeletal steel in ferrocement roofing elements. Slab reinforced with bamboo strips as skeletal steel & chicken wire mesh are subjected to monotonically increasing uniformly distributed load to study the load deflection behaviour.The investigation showed that by using the bamboo the cost of roofing elements comes to about 50 % of reinforced concrete & 70% of ferrocement elements.

#### (b) Rice Husk

(*Choeypont et al, 1988*) investigated the use of rice husk ash (RHA) cement for ferrocement. The RHA cement used constitute 50% RHA & 50% Portland Cement.The test results showed that the RHA cement mortar has better resistance to acidic attack than the Portland cement mortar.RHA in ferrocement improved its impact strength ,however its compressive, tensile, flexure strength decreased.

#### (c) Lime

(*Raj, 1989*) investigated the use of lime as an admixture in ferrocement.The amount of lime added was 15% by weight of the cement applied as fine powder .It was found that during the fresh state addition of lime in ferrocement mortar causes the mortar to become more plastic ,records higher slump & increases the workability. The optimum dose of lime was found to be between 15% to 20% by weight of cement & lime dose should be added either in the form of fine powder or putty.

### 1.2.6 Potential Applications

In the Last two decades ferrocement has been extensively used in different types of structure as follows :

### **Housing Applications**

Ferrocement has found wide spread applications in housing particularly in roofs ,flops,slabs, & walls . Some researches were also made on the use of ferrocement in beams & columns . Ferrocement roofs investigated included shell roofs , folded plates & the channel shaped roofs , box girders & secondary roofing.

*(Kaushik et al 1987)* investigated the behaviour of ferrocement cylindrical shell units as roofing elements and found that they can be used as roofing elements for low cost housing & satisfy Indian requirements of loading ,deflections & crack width with economy.

*(Jagdish & Radhakrishna, 1977)* investigated the suitability & effectiveness of using the ferrocement hyperbolic paraboloid shall roofing units for short spans of 4 m . They recommended that the ferrocement hyperbolic paraboloid shells with two layers of chicken mesh is quite adequate .

### **1.2.7 Other Applications**

Ferrocement applications to water resources structures are numerous . Ferrocement has been used for :-

1. Water tanks
2. Canal linings
3. Aqueducts
4. Pipes
5. Ferrocement gates
6. Culverts

Ferrocement has been widely accepted as a suitable building material for Biogas structures & marine applications such as boats ,ships ,bazges , pomtoons ,treatment plants, & floating docks etc.

In India Structural Engineering Research Centre (SERC) Ghaziabad is conducting extensive research on development of ferrocement for rural applications. This centre is concentrating efforts towards solving problems of farmers .viz grain storage & water storage structures by conducting research on how ferrocement could economically be used for the manufacture of bins ,silos & water tanks.

### **1.2.8 Organization of Thesis**

Many researchers put their theory for use of ferrocement laminates for retrofitting but very less work specifically regarding the behaviour of ferrocement laminates for strengthening of shear deficient beams has been done. So the aim of the study is to study the effect of retrofitting work done in strengthening of shear deficient beams initially stressed to a specified limits with ferrocement laminates. Organization of thesis has been organized in the following five chapters:

Chapter 1: This chapter general introduction and history of ferrocement

Chapter 2: This chapter deals with review of literature work and the need for the present study

Chapter 3: This chapter details the experimental programme. In this properties of several of various material used in work have been discussed.

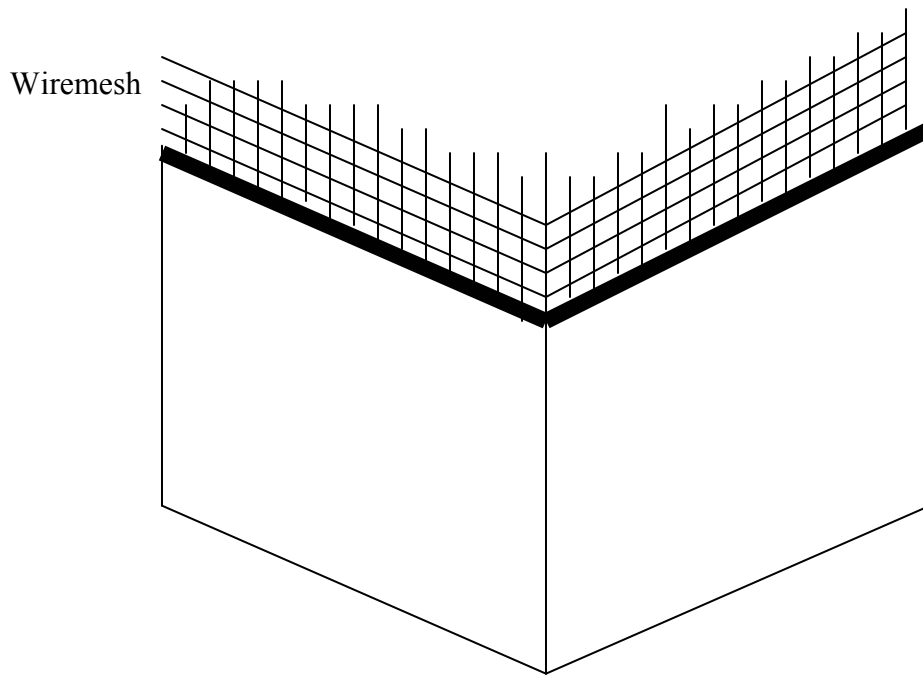
Chapter 4: This chapter is about the results and discussion of the work carried out.

Chapter 5: It details the conclusions of the work carried out

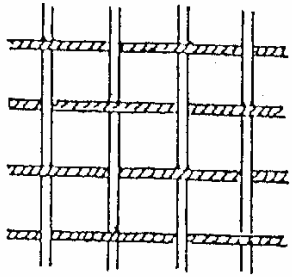
Chapter 6: It deals with the discussions and future scope of ferrocement research.

**Table 1.1 Different Types of Wire Meshes and their Characteristics**

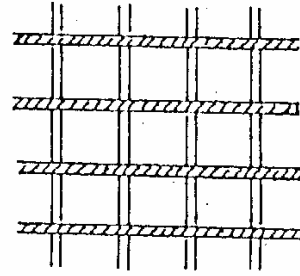
Type of Mesh	Wt of one layer per unit area(kg/m <sup>2</sup> )	Mesh thickness (mm)	Steel content (kg/m <sup>2</sup> )	Steel surface per unit Vol(mm <sup>2</sup> /mm <sup>3</sup> )
Hexagonal wire mesh 12mm * 22 gauge	0.58	1.4	410	0.275
Square welded mesh 12 mm *12 mm *19 gauge	1.08	2	540	0.248
Expanded metal expamet T.C.G.269	1.22	2.5	490	0.245
Wastan mesh	3.55	5.5	650	0.236



**FIGURE 1.1 Typical Cross Section Of Ferrocement Laminate**



WOVEN MESH  
Fig 1.2 (a)



WELDED WIRE FABRIC  
Fig 1.2 (b)

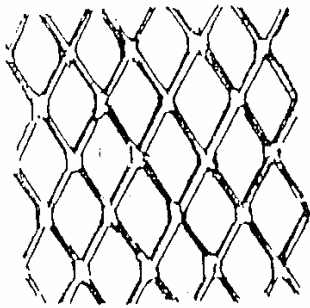


Fig 12 (C) EXPANDED METAL

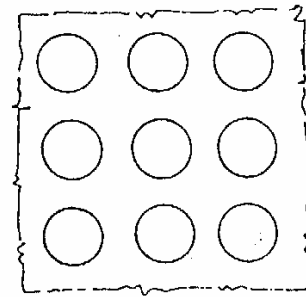


Fig 12 (d) PERFORATED SHEET

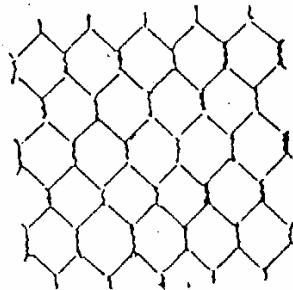


Fig 12 (e) CHICKEN WIRE MESH

DIFFERENT SHAPES OF WIRE MESHES

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## **CHAPTER : 2**

### **REVIEW OF LITERATURE**

#### **2.1 GENERAL**

Shear is that force which makes the specimen diagonally strong if any specimen is strong diagonally then it is strong in shear & thus specimen will not fail in shear .In the ferrocement because the section of element is very thin so researches have been carried out for evaluation of shear but for making the element structurally strong there should be more research for calculating the shear strength of ferrocement . But new developments of ferrocement warrant the study of its shear behaviour.

Several test methods can be used for shear testing .In shear testing the specimen fails when shear crack appears on the surface of the specimen . The shear crack appears diagonally .

There are a variety of tests to study the shear behaviour including the two point loading test ,four point loading test & axial compression test etc. These tests are suitable for different shape specimens i.e. a two point loading test is more suitable for the rectangular shape specimens. In two point loading we apply load symmetrically i.e. by dividing the span into three equal parts .

For trough shape specimen we generally apply axial compression test for testing in direct shear.

#### **2.2 REVIEW OF PREVIOUS WORK.**

Very little research work has been done to evaluate the shear strength of ferrocement laminates One of the major reasons of little research has been reported in the literature on behaviour of ferrocement in shear is due to the fact that in shear it is mainly used in thin shell elements where shear stresses are not a critical design consideration.

Most of the applications of ferrocement is in civil engineering structures where high tensile strength or small crack width is the governing criteria . Ferrocement panels used

for roofing purposes ,normally have large span/ depth ratio in flexure which the shear as a failure criteria.

**(S.K.Kaushik and V.K.Garg, 1994)** They tested reinforced concrete beams to study the effectiveness of externally bonded precast ferrocement plates in strengthening beams showing shear distress. The relative efficacy of the bonding media ( C-S mortar ,epoxy ) used in bonding the precast F.C Plates to the sides of beams were studied . Ferrocement was considered attractive for this application due to its thig tensile strength ,low weight economy in cost ,long life of treatment and precise assessment of the additional strength gained by its use .

Cement sand mortar bonding medium was found less effective than epoxy repaired beams, which showed a 20.5 % increase ultimate strength over original beams when subjected to identical loading . This specimen showed 25 % lower deflections than the original beams at the ultimate stage. The studies showed that the technique can be advantageously used for rehabilitation of RC beams failing in shear.

They had concluded from their studies that Ultimate strength characteristics of RC beams can be significantly increased with ferrocement laminates , the increase being 3.5 % corresponding to cement sand mortar and its 20.5 % corresponding to Epoxy bonded specimens. Reduction in deflection was less at working loads strengthened in shear, however 2 % and 25 % reduction was noted corresponding to beams strengthened cement sand mortar and Epoxy respectively. In the beams strengthened with ferrocement crack width got reduced significantly upto 55 and 30.8 % corresponding to cement sand mortar and epoxy respectively.

**(S.F.A. Rafeeqi , S..H.Lodi and Z.R. Wadalawala, 1998)** Shear mode of failure in beams is undesired mainly being a brittle failure . Therefore an attempt has been made by these people to explore the potentials of ferrocement in transferring the brittle mode to ductile mode .

Ferrocement wrap and equally spaced strips with one or two layers of woven square mesh are presented and compared with RC beam designed as shear deficient ( in both) . These researchers from their studies had concluded that the strengthened beam showed a marked improvement in performance at service load, greatly improved ductility at

ultimate with either a ductile shear failure or seemingly a transition from shear to flexure mode of failure. Moreover ferrocement wraps are more effective than ferrocement strips. Another thing of importance deduced is that the enhancement in load carrying capacity is not substantial, however is present. Service range had been able to increase the stiffness of strengthened beams and also reduces the crack width and deflection in comparison with un-strengthened beam .

(*M.A.Al-Kubaisy and P.J.Nedwel, 1999*) presents the study on the behaviour and strength of ferrocement beams under shear . The results of thirty simply supported beams tested under single concentrated load are presented . The influence of the following variables ;shear span to depth ratio ( $a/h$ ), volume fraction of reinforcement ( $V_f$ ), and the strength of mortar ( $f_c'$ ) on crack patterns ,modes of failure and the cracking shear strength were examined . The results indicated that the cracking shear strength of ferrocement beams increases as the  $a/h$  ratio is decreased and as  $f_c'$  and  $V_f$  are increased .

An empirical equation is proposed to predict the cracking shear strength. This equation takes into account the effect of variables covered in this study . The proposed equation for computing the cracking shear strength is compared with other test results and also with the ACI Code provisions which are shown to be very conservative.

They had concluded that the mode of shear failure cant only be predicted on the basis of  $a/h$  and  $V_f$  alone ,only we can get some guide regarding it further beams failing in shear mode had more reserve capacity beyond cracking than beams failing in diagonal mode. The shear force at failure cannot be relied upon to exceeding the cracking shear. Accordingly the shear force at critical cracking must be considered as the useful shear capacity of beam , moreover we can say that the

cracking shear force may be predicted by the lower bound line given by equation

$$V_{cr} / bh = 0.1 f_c'^{1/2} + 55.5 V_f .h / X_c$$

$$\text{Where } X_c = 198 (a/h) / f_c'^{1/2} - 14$$

(*Callan & Kirwan,1960*) tested ferrocement specimens reinforced with woven mesh & skeletal bars in bending at a shear span/depth ratio of 0.4 .The shear strength of

ferrocement elements was equal to about 35 % of their flexural strength over a fairly wide range of steel contents (288-480 kg/m<sup>3</sup>).

**(Mansur & Ong, 1987)** found that the shear strength of ferrocement depends on the strength of mortar volume fraction & strength of wire mesh . Shear strength of ferrocement beams with welded wire mesh was found to be more than the shear strength of ferrocement beams reinforced with woven or hexagonal wire mesh. They investigated the behaviour & strength of ferrocement in transverse shear by conducting flexural tests on beams under two symmetrical point loads. The beams were reinforced with only welded wire mesh , with the various layers of mesh lumped together in layers at the top & bottom. Test results indicate that diagonal cracking strength ferrocement increases as the span to depth ratio is decreased & volume fraction of reinforcement, strength of mortar & amount of reinforcement near compression face is increased. Ferrocement beams are susceptible to shear failure at small span to depth ratios when volume fraction of reinforcement & strength of mortar are relatively high .

**(Venkata Krishna & Basa Gouda, 1989)** performed testing of beams with different volume fractions of reinforcement in transverse shear . It was found that the shear strength depends on strength of mortar ,strength of wire mesh , volume fraction & shear span .Theoretical expressions were developed for predicting the shear strength of first crack & collapse of ferrocement beams with different types of wire meshes namely hexagonal ,woven & welded. The correlation between the experimental & predicted value was quite satisfactory .

**(Lua & Vanwanrioj, 1988)** studied the effect of galvanized welded wire mesh as shear reinforcement when it was used with shotcrete for strengthening of reinforced concrete beams. Tests results showed that wire mesh acting as reinforcement was fully effective.The failure mechanism of beams changed from shear failure to flexure when strengthened with ferrocement layer.

**(Al – Sulaimani, 1977)** The behaviour of hollow box beams under transverse shear was studied by Al Sulaimani conducting flexural tests on 15 nos .beam specimens . The major parameters used were amounts of wire mesh reinforcements in web & in flanges of the beam & shear span to depth ratio. The test results indicate that the cracking & ultimate shear forces increases as wire mesh in web is increased. Placing

wire mesh in flanges in web also increases the shear resistance through arresting of the tension cracks & causing them to be finer.

Hence, the shear behaviour is studied with relation to the total volume of wire mesh reinforcement which includes wire mesh in both webs & flanges. The cracking & ultimate shear strength also increases as shear span to depth ratio is decreased. ACI equation for shear strength for conventional reinforced concrete beams without web reinforcement underestimates the cracking shear strength of ferrocement box beams.

The behaviour of ferrocement under direct shear was investigated by Al-Sulaimani by conducting axial compression test on z-shaped specimens reinforced with woven wire mesh producing pure shear on the shear lane. The major study parameters were the volume fraction of wire mesh reinforcement, the shear span & mortar strength. Conclusions were made from the studies of the behaviour of ferrocement under direct shear as two stages of behaviour namely cracked & uncracked, while ferrocement under flexure exhibits a third stage i.e. ultimate or plastic stage in addition to the uncracked & cracked stages. Hence ferrocement is less ductile under shear than flexure. The presence of cracked stage in ferrocement behaviour under direct shear increases with increasing amount of wire mesh reinforcement.

They have also found out that the cracking & ultimate shear stresses of ferrocement increases with increasing mortar strength & wire mesh reinforcement, and can be predicted by the following empirical formulae:

$$\tau_{cr} = f_t + 450 V_f$$

$$\tau_{Ult} = f_t + 900 V_f$$

$f_t$  = Mortar tensile strength

$V_f$  = Volume fraction of wire mesh

$\tau_{cr}$  = Cracking shear stresses.

$\tau_{ult}$  = Ultimate shear stress.

The shear stiffness in the uncracked stage is not significantly affected by the amount of wire mesh when it is significantly affected in the cracked stage. However, the shear stiffness in both the stages are affected by the mortar strength. Regarding ductility of ferrocement in shear they had said that with increasing wire mesh reinforcement ultimate shear displacement increases while with the increase in mortar strength ductility reduces but toughness represented by the area under the shear load & shear displacement curve is not significantly effected. Toughness however increases with increasing wire mesh reinforcement.

Recently, 89 ferrocement trough section elements in shear have been tested in shear at the Indian Institute of Sciences, Bangalore variable of study are the no of layers of wire meshes, volume fraction of longitudinal steel bars in tension flange, spacing of shear reinforcement bars & shear span to depth ratio.

Test results indicate that the shear strengths of ferrocement in flexural cracking, web shear cracking & web shear failure increased as the shear span to depth ratio was decreased & the volume fraction of wire mesh & volume fraction of longitudinal steel bars in tension flange increased.

### **2.3 THE NEED FOR PRESENT STUDY**

The foregoing review reveals that shear strength of ferrocement elements is greatly affected by various parameters such as, variation in thickness of ferrocement plastering, contribution of various stress levels of safe load.

The enhancement in mechanical properties suggest that the use of welded wire mesh in ferrocement elements offer high resistance to shear because welded wire mesh has high modulus of elasticity.

As not much information is available on the behaviour of ferrocement under shear, therefore, it is necessary to undertake more studies on the shear behaviour of ferrocement.

**Therefore the objective of present study is to carry out an experimental study on the behaviour of shear deficient RC beams retrofitted using ferrocement laminates.**

## **CHAPTER-3**

### **EXPERIMENTAL PROGRAMME**

#### **3.1 INTRODUCTION**

In the existing scenario there are a number of laminates like CFRP(Carbon fibre reinforced polymer),GFRP(Glass fibre reinforced polymer), Ferrocement etc are being used for retrofitting of structures. Among all the materials ferrocement due to its inherent properties like lighter weight, ease of construction, low self weight, thinner section etc is gaining popularity. Few researchers had used ferrocement laminates for enhancing either the flexural or shear strength of the beam but in actual the beam may need strengthening in both shear and flexure.

Thus in the present study shear deficient beams are cast and subsequently stressed to 60%, 75%, 90% of the safe load and are retrofitted with ferrocement laminates bonded to beam with cement slurry and having wire mesh at an angle of  $45^{\circ}$  to the longitudinal axis of the beam.

For the proposed work eight real size beams (127 x 227 x 4100 mm) beams were cast. Out of these two are controlled beams tested to find out safe load carrying capacity of

beams and subsequently two each of the rest of the six beams are stressed to 60%, 75%, and 90% of the safe load and then retrofitted with 20 mm thick ferrocement laminate.

### **3.2 TEST PROGRAMME**

The test programme is so devised so as to find out the properties of materials to be used for casting of beams and then the behavior of retrofitted beams. The test programme consist of:

1. Determination of basic properties of constituent materials namely cement, sand, coarse aggregates and steel bars as per relevant Indian standard specifications.
2. Casting of eight real size beams (127 x 227 x 4100mm) using M 20 grade concrete, the mix of which is designed with evaluated properties.
3. Computation of the ultimate failure load of the beams and subsequently the safe load from deflection criteria.
4. The beams are stressed to 60 %, 75 % and 90 % of safe load and then retrofitted with ferrocement laminates of thicknesses 20 mm using cement slurry as bonding agent having mesh at an angle 45 degree to the longitudinal axis of beam

### **3.3 MATERIALS**

Cement, fine aggregates, coarse aggregates, reinforcing bars are used in designing and casting of beams and MS welded wire mesh, cement slurry are used for retrofitting of these beams. The specifications and properties of these materials are as under:

#### **3.3.1 Cement**

Portland pozzolana 43 grade cement from a single lot is used for the study. The physical properties of cement as obtained from various tests are listed in Table 3.1. All the tests are carried out in accordance with procedure laid down in IS: 8112-1989.

#### **3.2 Fine Aggregates**

Locally available sand is used as fine aggregate in the cement mortar and concrete mix. The physical properties and sieve analysis of results of sand are shown in Table 3.2 and Table 3.3.

#### **3.3.3 Coarse Aggregates**

Crushed stone aggregate (locally available) of 20mm and 10mm are used through out the experimental study. The physical properties and sieve analysis of results of both coarse aggregate are given in Table 3.4 ,Table 3.5 and Table 3.6.

#### **3.3.4 Water**

Fresh and clean water is used for casting and curing the specimens. The water is relatively free from organic matter, silt, oil, sugar, chloride and acidic material as per requirements of Indian standard.

#### **3.3.5 Reinforcing Steel**

HYSD steel of grade Fe-415 of 10mm,8mm and 6mm diameters were used as longitudinal steel. 10mm dia bars are used as tension reinforcement and 8mm bars are used as compression steel. 6mm diameter bars are used as shear stirrups. The properties of these steel bars are shown in Table3.7

#### **3.3.6 Steel Mesh**

MS welded steel wire mesh of 2.4mm diameter with square grids was used in ferrocement jacket. The grid size of mesh was 40X40 mm. The salient properties of mesh wire used are given in Table.3.7

#### **3.3.7 Concrete Mix**

M20 grade concrete mix is designed as per standard design procedure using the properties of materials as discussed and provided in Table 3.1 to 3.6 The water-cement ratio used in the design is 0.5. The mix proportion of material comes out to be 1:1.45:3.123 (cement: sand: aggregate) and compressive strength of materials after 7 days and 28 days is 21.5 MPa & 29 MPa respectively.

#### **3.3.8 Mortar Mix**

The range of mix proportion recommended for common ferrocement application are between 1:1.5 to 1:2.5 (cement:sand) by weight, but not greater than 1:3 and water cement ratio by weight, 0.35 to 0.5. The higher the sand content higher is the required water contents to maintain same workability. Fineness modulus of the sand, water cement ratio and sand-cement ratio should be determined from trial batches to ensure a mix that

can infiltrate the mesh and develop a strong and dense matrix. In the present study the proportion of cement –sand mortar used for the ferrocement sheets is 1:2 (cement: sand and the water-cement ratio for mortar taken as 0.40.

### **3.4 RCC BEAM DESIGN**

In the present study RCC beam is designed using M20 grade concrete and Fe 415 steel. The RCC beam is designed using limit state method considering it to be an under-reinforced section. The beam is designed having 2 steel bars of 8mm dia at compression face and 2 bars of 10mm dia at tension face. The stirrups used are of 6mm diameter and at the spacing of 300 mm which is more than the minimum required spacing, so that the beam should behave as a shear deficient beam. The dimension of the beam are fixed to 127 x 227mm overall. Longitudinal section and cross-section of beam is shown Figure 3.1 and Figure 3.2

### **3.5 CASTING OF COMPOSITE BEAMS**

The casting of beams is done in a single stage. The beams are casted in a mould of size 127 x 227 x 4100 mm. First of all the entire beam mould is oiled. so that the beam can be easily removed from the mould after the desired period. Spacers of size 25 mm are used to provide uniform cover to the reinforcement. When the bars have been placed in position as per the design, concrete mix is poured in the mould and vibrations are given with the help of needle vibrator. The vibration is done until the mould is completely filled and there is no gap left. The beams are then removed from the mould after 48 hours. After demoulding the beams are cured for 28days using jute bags.

### **3.6 TESTING ARRANGEMENT**

All the eight beams are tested under simply supported end conditions. Two point loading is adopted for testing and spacing between two concentrated loads is so selected that  $a/d$  ratio comes out in range of 5 for the beam to be failing in combined shear and flexure. The testing of beams is done with the help of hydraulic operated jack connected to load cell. The load is applied to the beam with the help of hydraulic jack and the data is recorded from the data acquisition system, which is attached with the load cell. Three dial

gauges are placed at quarter positions, one dial gauge is placed at the center and other two dial gauges are placed at a distance of  $\text{span}/4$  from each end. The value of deflection is obtained from these dial gauges. Out of these eight beams 2 are control beam, which are tested after 28 days of curing to find out the safe load which is taken as load corresponding to deflection of  $L/250$  i.e. 15 mm. Two each of the remaining six beams are stressed up to 60%, 75%, and 90% of the safe load.

### **3.7 RETROFITTING OF BEAMS**

The beams are stressed up to a specified limit as above and then retrofitted by applying steel wiremesh at an orientation of  $45^\circ$  as shown in fig no 3.4 and then plastering it with cement mortar up to the thickness of 20mm for all six beams. Therefore final cross-section of beam with ferrocement laminate will become 167 x 247 x 4100 mm. Effect of three different stress levels of 60%, 75%, and 90% has been studied to see their effect on the strength of retrofitted beams with steel wiremesh at an angle of 45 degree, placing it over the three surfaces of beam. An overlap of 3 inches at the place of joint between wire mesh is introduced.

#### **3.7.1 Process of Retrofitting**

First of all surface of beam is cleaned and after cleaning the surface, the cement slurry is applied on beam for bonding between ferrocement laminate and beam. All 6 beams are retrofitted with wiremesh at an orientation of  $45^\circ$  as shown in fig no 3.4. After that 20 mm plaster in the form of 1:2 cement mortar ( $w/c=0.4$ ) is applied on three faces of beams. After this the beam is cured for 7 days. Then with the same procedure as of control beam, testing of beams is done under two point loading in order to calculate ultimate load and corresponding deflections.

The following chapter contains the observations of the experimental programme along with the results and their discussion.

**Table 3.1: Physical Properties of Cement used**

<b>Sr. No</b>	<b>Characteristics</b>	<b>Value obtained Experimentally</b>	<b>Value specified by IS: 8112-1989</b>
1	Standard consistency	34	-
2	Fineness of cement as retained on 90 micron sieve	0.5	<10%
3	Setting time 1. Initial 2. Final	35 mins 5 hours	>30 mins <10 hours
4	Specific gravity	3.07	-
5	Compressive strength (N/mm <sup>2</sup> ) 1. 7days	33.5	33

	2. 28 days	43.5	43
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**Table 3.2: Physical Properties of Fine Aggregates**

Sr. No.	Characteristics	Value
1.	Specific gravity	2.56
2.	Bulk density loose (kg/lt)	1.48
3.	Fineness modulus	2.51
4.	Water Absorption	2.06%
5.	Grading Zone	Zone III

**Table 3.3: Sieve Analysis of Fine Aggregates**

Total weight taken = 1000gm

Sr. No.	Sieve Size	Mass Retained (gm)	Percentage Retained	Cumulative Percentage Retained	Percent Passing
1.	4.75 mm	95.0	9.5	9.5	90.5
2.	2.36 mm	42.5	4.25	13.75	86.25
3.	1.18 mm	110.5	11.05	24.8	75.2
4.	600 $\mu$ m	128.5	12.85	37.65	62.35
5.	300 $\mu$ m	308.0	30.8	68.45	31.55
6	150 $\mu$ m	281.0	28.1	96.55	3.45
7.	Pan	34.5	3.45		
				$\Sigma = 250.70$	Fineness modulus =2.507

**Table 3.4: Sieve Analysis of Coarse Aggregate (20mm)**

Total weight taken = 3 kg

<b>Sr. No.</b>	<b>Sieve Size</b>	<b>Mass Retained (kg)</b>	<b>Percentage Retained</b>	<b>Cumulative Percentage Retained</b>	<b>Percent Passing</b>
1.	20 mm	0	0	0	100
2.	12.5 mm	2.1865	72.883	72.883	22.117
3.	10 mm	0.6745	22.483	95.366	4.634
4.	4.75mm	0.1300	4.33	99.69	0.31
5.	Pan	0.009	0.3		0
				$\Sigma = 267.93$	Fineness modulus=7.68

**Table 3.5: Sieve Analysis of Coarse Aggregate (10mm)**

Total weight taken = 3 kg

<b>Sr. No.</b>	<b>Sieve Size</b>	<b>Mass Retained (kg)</b>	<b>Percentage Retained</b>	<b>Percent Passing</b>	<b>Cumulative Percentage Retained</b>
1.	12.5 mm	0.555	18.5	81.5	18.5
2.	10 mm	0.8905	29.68	51.82	48.18
3.	4.75mm	0.9565	31.88	19.94	80.06
5.	Pan	0.5970	19.90		
					$\Sigma = 146.74$ Fineness modulus=6.47

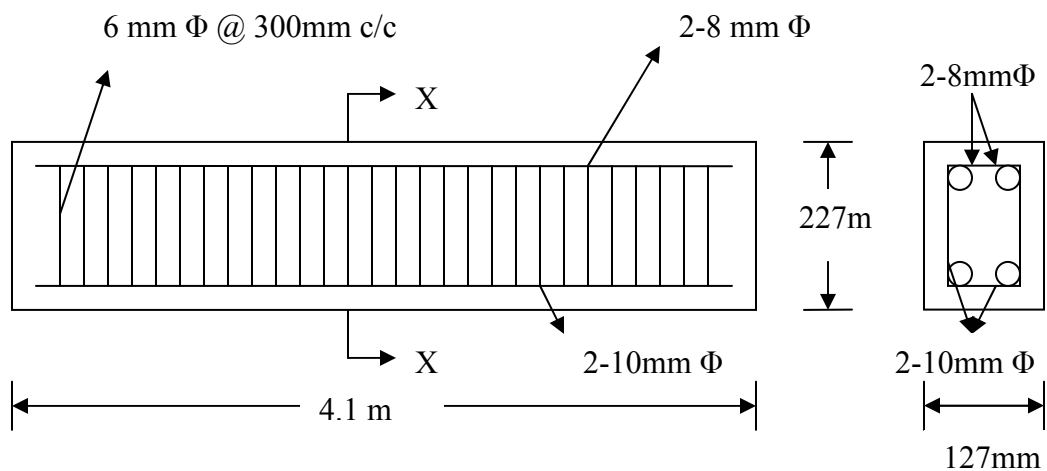
**Table 3.6: Physical Properties of Coarse Aggregates**

Sr. No.	Characteristics	Value	
		20mm	10mm
1.	Type	Crushed	Crushed
2.	Specific gravity	2.655	2.704
3.	Water absorption	3.645	1.643
4.	Fineness modulus	7.68	6.47

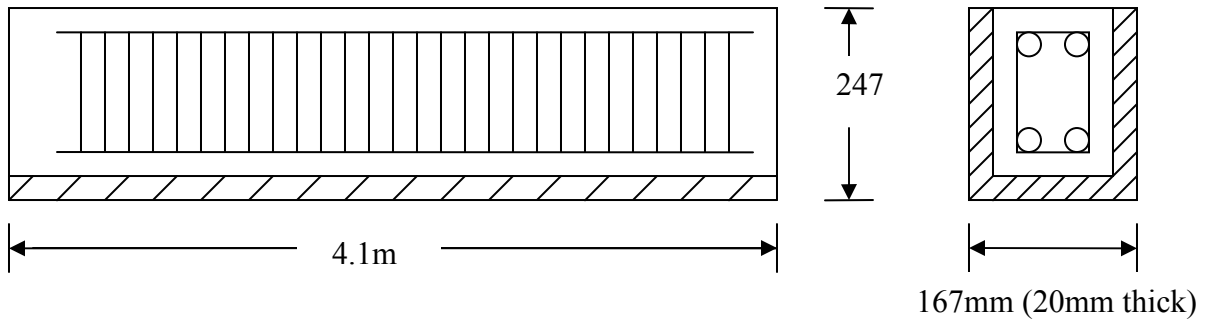
**Table 3.7  
Physical Properties of Steel Bars**

**and Steel Mesh Wires**

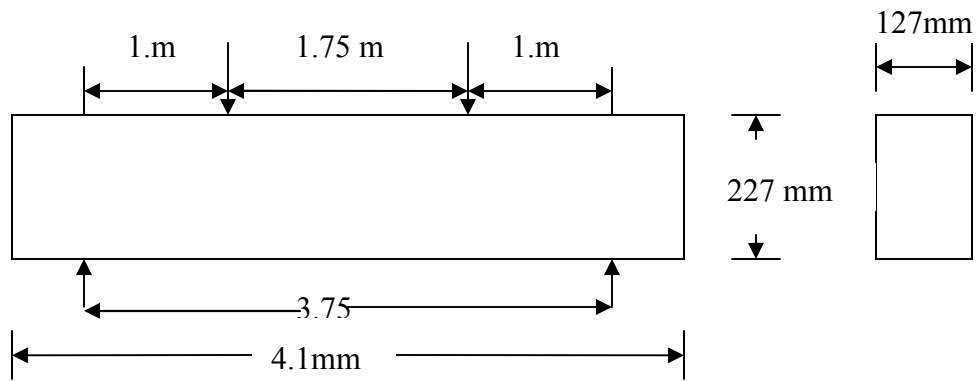
Sr. No.	Diameter of bars/ mesh wire	Yield-strength (N/mm <sup>2</sup> )	Ultimate strength	Percentage Elongation
1.	10mm	445.55	509.2	15.5
2.	8mm	559.5	634.13	20.3
3.	6mm	442.42	612.7	32.9
4.	2.4mm	400	511.36	2.52



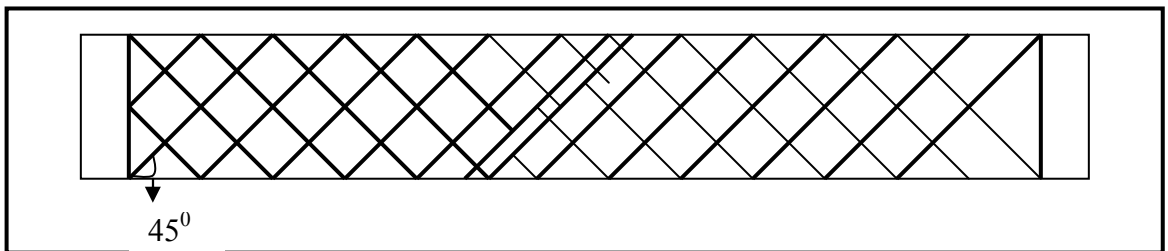
**Figure 3.1: Longitudinal and Cross-Section of Unretrofitted**



**Figure 3.2: Longitudinal and Cross-Section of Retrofitted Beam**



**Figure 3.3: Loading System of Beam**





**Figure 3.4 Orientation of Wiremesh at 45**

## **CHAPTER 4**

### **RESULTS AND DISCUSSION**

#### **4.1 INTRODUCTION**

In the present study, the effect of stress level on the strength of shear deficient retrofitted beam is studied. For the purpose the beams are firstly stressed to predecided stress level of 60%, 75% and 90% of the safe load and then the beams are retrofitted beams using 20 mm thick ferrocement laminate having single layer welded wire mesh at an angle of  $45^{\circ}$  to the longitudinal axis of beam. A comparative study on the strength variation of retrofitted beams and control beam is done with above parameters and is presented in the subsequent section.

## **4.2 TESTING METHODOLOGY**

Firstly control beams are tested to failure and the data corresponding to it is recorded through data acquisition system.

Then two each of the rest of the six beams are stressed upto 60%, 75% and 90% of the safe load of control beam. The safe load is calculated from the load deflection curve for allowable deflection of 15mm and the load so read from the curve is 13 KN and the corresponding loads at 60%, 75%, 90% stress level are 7.8KN, 9.75KN, 11.7KN respectively. Then the retrofitting of the beams is done with cement mortar of thickness 20mm along with wire mesh bonded on three sides for all six beams. After one week of curing the beams are tested again with the same method as the control beam was tested initially and the corresponding results are recorded and shown in the sections to follow.

The beam designations as used are as follows:-

Control Specimen - Control beam (C1)

Retrofitted Beam 1 - R1 ( 60% stressed)

Retrofitted Beam 2 - R2 ( 60% stressed)

Retrofitted Beam 3 - R3 ( 75% stressed)

Retrofitted Beam 4 - R4 ( 75% stressed)

Retrofitted Beam 5 - R5 ( 90% stressed)

Retrofitted Beam 6 - R6 ( 90% stressed)

## **4.3 CONTROL BEAMS**

Two beams are tested as a control beam under two point concentrated loading system. As the beams are deficient in shear, so the distance between two loads is so selected that  $a/d$  ratio is around 5 for the beam to be failing either in shear or flexure or in both. The load is increased in intervals and deflection is noted at  $L/2$  and  $L/4$  as shown IN plate 4.1 .The results of load and corresponding deflection so obtained are presented in Table 4.4 to Table 4.9. In the beginning the deflection in middle increases almost linearly with the increase in load but after load of 20 KN it increases at much higher rate till the ultimate load of 21.8 KN as shown in fig no 4.1. The first crack in the beam is observed is at a load of 8 KN thereafter number of cracks increased and spread over the entire length of the beam. Pattern of cracks as shown in plate 4.1 indicates the combined shear and flexure failure of the beam.

#### **4.4 COMPARISONS OF RETROFITTED AND CONTROLLED BEAM**

##### **4.4.1 Effect of Ferrocement laminate on the Beam (stressed to 60% -R1 & R2) as compared to Control Specimen.**

The effect on strength of retrofitted shear deficient RC beam stressed to 60 % level is shown in Figs 4.1 & 4.2. The Table 4.4 and Table 4.5 shows the load deflection data for control beam & 60 % stressed retrofitted beam. Plates 4.2 & 4.3 show the loading arrangement & crack pattern for the retrofitted beam.

It is observed from the experimental data and the corresponding graph that retrofitting leads to increase in the ultimate load carrying capacity from 21.8 KN (Control Beam) to 35.3 KN whereas the deflection corresponding to ultimate load of 35.3 KN is 38.5 mm as compared to 44.85 mm for the control beam at 21.8 KN. Also there is a considerable increase in the allowable load corresponding to permissible deflection of 15mm from 13KN (Control Beam) to 21.5 KN for the retrofitted beam.

For the R2 specimen exactly similar trend is observed and increase in load is also of almost of the same order i.e from 21.8 KN (Control Beam ) to 36.2 KN with deflection of about 55 mm. The load corresponding to the permissible deflection also increases from 13 KN (Control Beam) to 20.5 KN.

Thus on an average for 60% stress level shear deficient beams, on retrofitting the ultimate load increase is of the order of 64 % and safe load corresponding to a deflection of 15 mm increased by 46 %.

#### **4.4.2 Effect of Ferrocement Laminate on the Beam (stressed 75%- R3 & R4 ) as compared to Control Specimen.**

The effect on strength of retrofitted shear deficient RC beam stressed to 75 % level is shown in Figs 4.3 & 4.4 The Table 4.6 and Table 4.7 shows the load deflection data for control beam & 75 % stressed retrofitted beam. Plates 4.4 & 4.5 show the loading arrangement & crack pattern for the retrofitted beam.

It is observed from the experimental data and the corresponding graph that retrofitting leads to increase in the ultimate load carrying capacity from 21.8 KN (Control Beam) to 33 KN whereas the deflection corresponding to ultimate load of 33 KN is 45 mm as compared to 44.85 mm for the control beam at 21.8 KN. Also there is a considerable increase in the allowable load corresponding to permissible deflection of 15mm from 13KN (Control Beam) to 15.5 KN for the retrofitted beam.

For the R4 specimen exactly similar trend is observed and increase in load is also of almost of the same order i.e from 21.8 KN (Control Beam ) to 34 KN with deflection of about 52.5mm. The load corresponding to the permissible deflection also increases from 13 KN (Control Beam) to 15.5 KN.

Thus on an average for 75 % stress level shear deficient beams, on retrofitting the ultimate load increase is of the order of 53.6 % and safe load corresponding to a deflection of 15 mm increased by 47.6 %.

#### **4.4.3 Effect of Ferrocement Laminate on the Beam ( stressed 90 %-R5 &R6 ) as compared to Control Specimen.**

The effect on strength of retrofitted shear deficient RC beam stressed to 90 % level is shown in Figs 4.5 & 4.6 The Table 4.8 and Table 4.9 show the load deflection data for control beam & 90 % stressed retrofitted beam. Plates 4.6 & 4.7 show the loading arrangement & crack pattern for the retrofitted beam.

It is observed from the experimental data and the corresponding graph that retrofitting leads to increase in the ultimate load carrying capacity from 21.8 KN (Control Beam) to 31.8 KN whereas the deflection corresponding to ultimate load of 33 KN is 45 mm as compared to 44.85 mm for the control beam at 21.8 KN. Also there is a considerable increase in the allowable load corresponding to permissible deflection of 15mm from 13KN (Control Beam) to 19 KN for the retrofitted beam.

For the R6 specimen exactly similar trend is observed and increase in load is also of almost of the same order i.e from 21.8 KN (Control Beam ) to 34 KN with deflection to about 52.5mm. The load corresponding to the permissible deflection also increases from 13 KN (Control Beam) to 17 KN.

Thus on an average for 90 % stress level shear deficient beams, on retrofitting the ultimate load increase is of the order of 46.75 % and safe load corresponding to a deflection of 15 mm increased by 42 %.

#### **4.4.4 Comparison of Retrofitted beams at different stress levels**

From a comparative point of view it is observed from Fig 4.7 that the beams which were initially stressed to a different levels of safe load show different behaviour, beams stressed to 60 % show maximum improvement in their ultimate load from 21.8 KN (control Beam) to 36.2 KN without much increase in the deflection if we are going to consider the performance as such. The results corresponding to the 75 % stress level show nearness to the earlier one and the only difference is that the ultimate load increases to a slightly lower value that is 34 KN again keeping the deflection in control as compared to the control beam. The results corresponding to the 90 % stress level also show significant improvement in their ultimate load and also in controlling the deflection as the ultimate load increases to about 32.2 KN but controlling the deflection to a maximum limit of 36.75 mm. Again the load corresponding to the permissible deflection (15 mm) increased to 18 KN.

#### **4.4.5 Failure Pattern**

Our objective of this study is to observe the behaviour of beam failing in shear by keeping distance between two point loads as 1.75 m and making shear span (  $a$  ) equal to 1 metre so that  $a/d$  ratio becomes 5, because a beam to be failing in combined shear and flexure it is necessary for  $a/d$  ratio to lie near to value 5.

For the control specimen(as shown in plate 4.1) it is observed from the cracks visible at the end of beam that it fails in shear but not pure shear because our  $a/d$  value is 5 , thus meaning that failure is combined shear and flexure . So the theoretical prediction of mode of failure are in agreement with experimental results .Then the beams are strengthened by ferrocement laminates making the beams more resistant towards any kind of failure, that is why all retrofitted beams of series R1, R2, R3, R4, R5, R6 show flexural cracks than shear cracks resulting in justification of strengthening meaning thereby that shear deficient beams get strengthened in shear also through the retrofitting procedure. However, the % age improvement in shear capacity can be considered as a part of further work.

#### **4.4.6 Comparison of Experimental crack characteristics of Retrofitted beams to Control specimen**

From a comparative point of view it is observed from Fig 4.10 and Table 4.1 that the retrofitting process is justified and leads to decrease in the crack spacing that means more equally distributed stress levels . A decrease of 28 %, 32 % and 13.2 % for series R1 R2 R3 R4 R5 and R6 respectively.

#### **4.4.7 Comparison of Experimental Deflection Characteristics of Retrofitted beams to Control specimen**

From a comparative point of view it is observed from Fig 4.8 and Table 4.2 that the % increase for permissible deflection is satisfactory in all the beams as the % increase they are showing is huge keeping in mind the retrofitting efficiency. They have been able to increase that by 61.55 %, 19.2 % and 42 % for Series R1& R2, R3&R4, R5 and R6 respectively.

#### 4.4.8 Comparison of Experimental Ultimate loads after Retrofitting

From a comparative point of view it is observed from Fig 4.9 and Table 4.3 that % increase in the ultimate loads of the retrofitted beams has been able to justify the thesis work till date because the results are in lieu to the economy considerations, all the beams have been able to perform very efficiently increasing the ultimate loads to a percentages as high as 63.97%, 53.6%, 46.75% for 60%,75% and 90% stress level respectively.

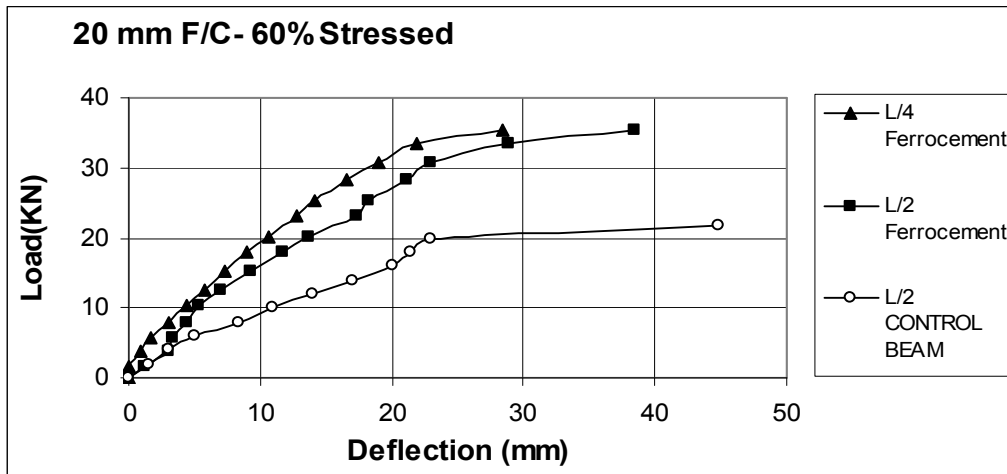


fig no 4.1 Load Deflection Curve for Specimen (R1)

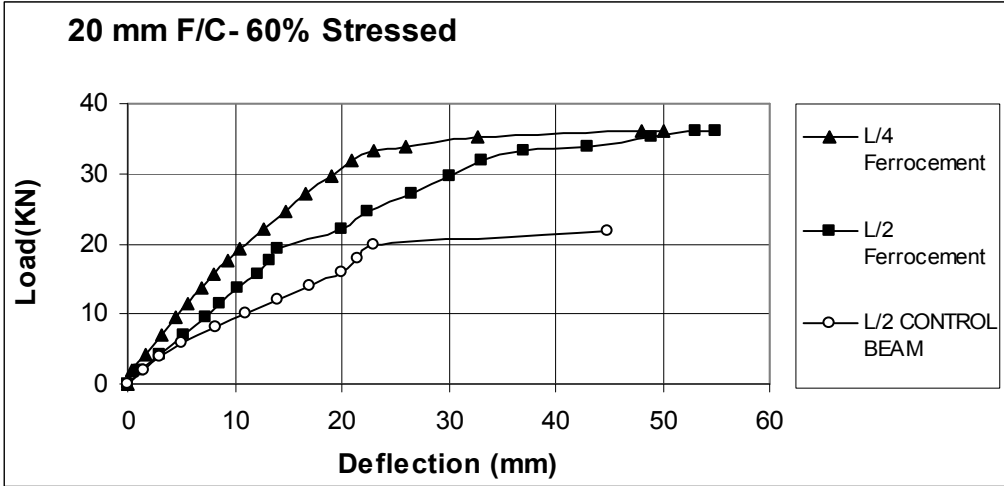


fig no 4.2 Load Deflection Curve for Specimen (R2)

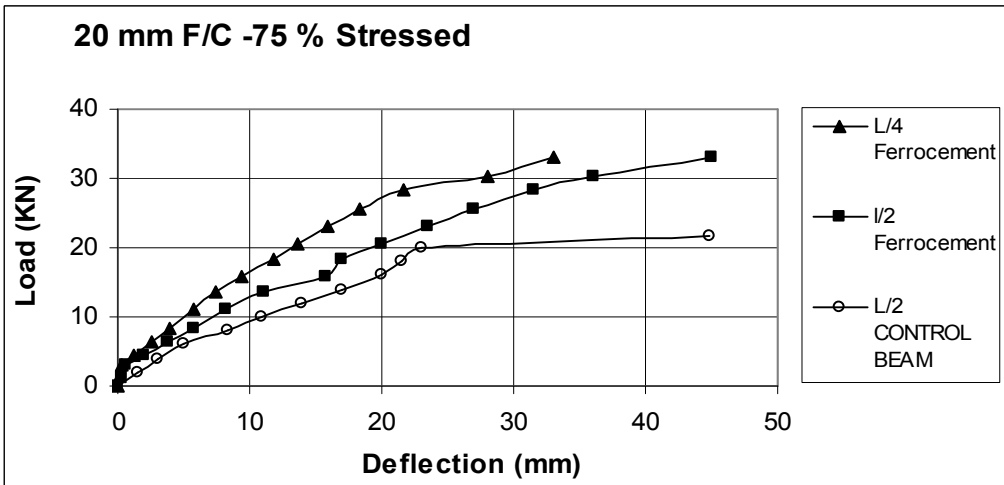


fig no 4.3 Load Deflection Curve for Specimen (R3)

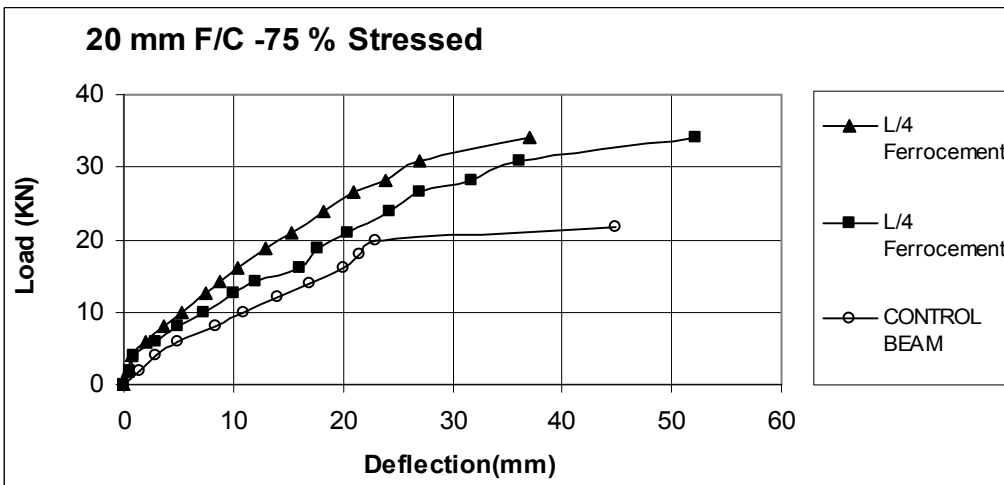


fig no 4.4 Load Deflection Curve for Specimen (R4)

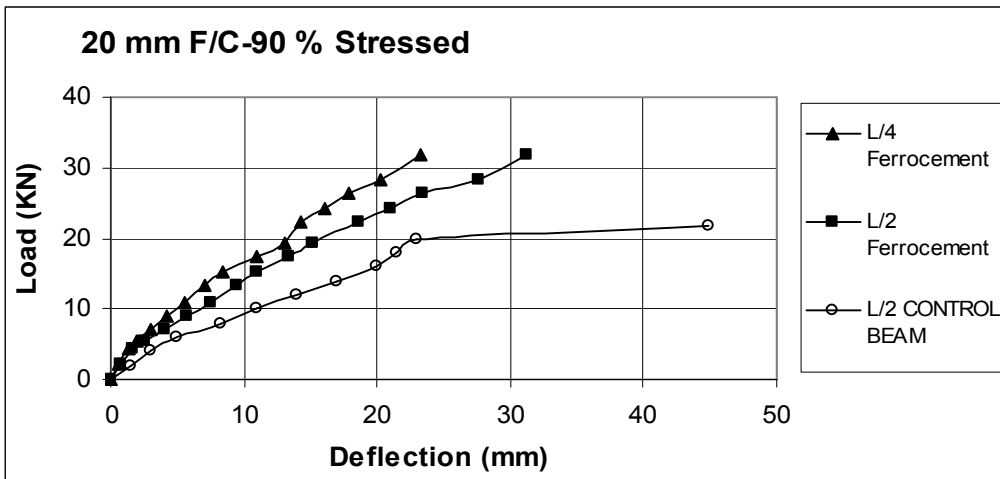


fig no 4.5 Load Deflection Curve for Specimen (R5)

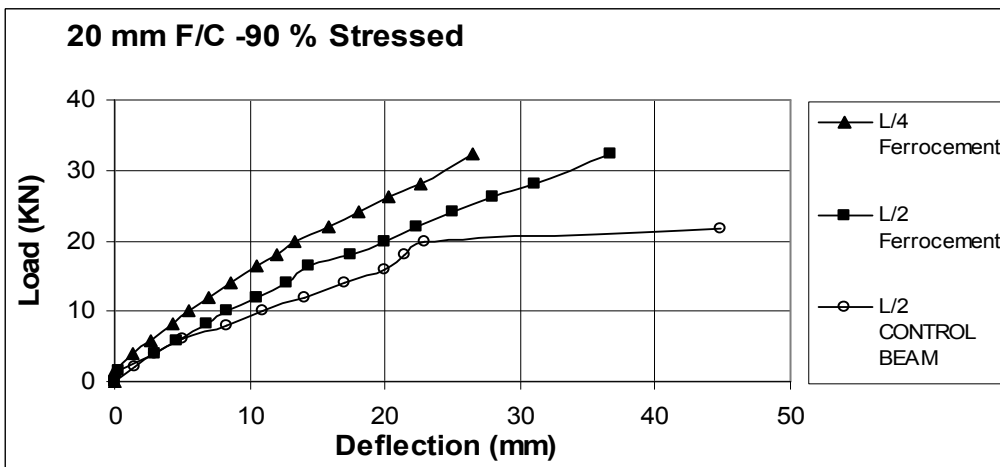
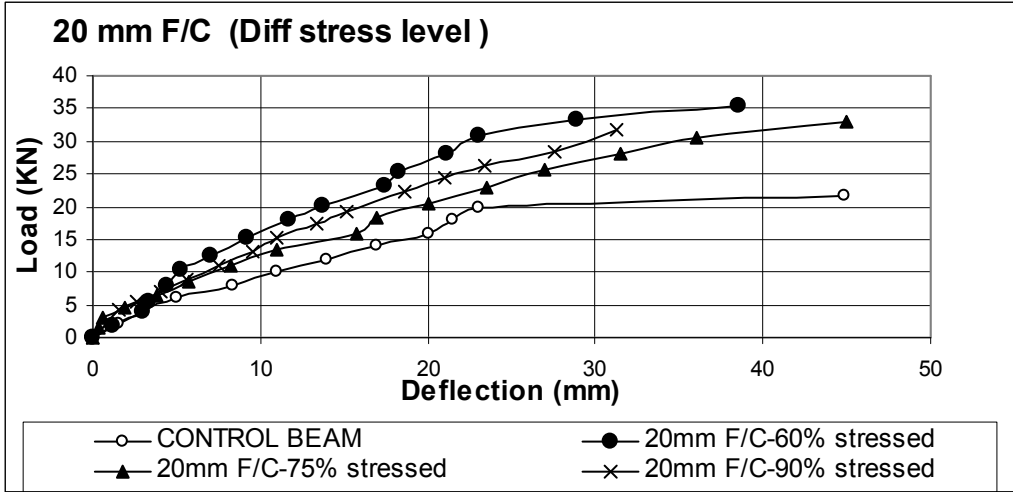


fig no 4.6 Load Deflection Curve for Specimen (R6)



**fig no 4.7 Comparison at Different stress level with control beam**

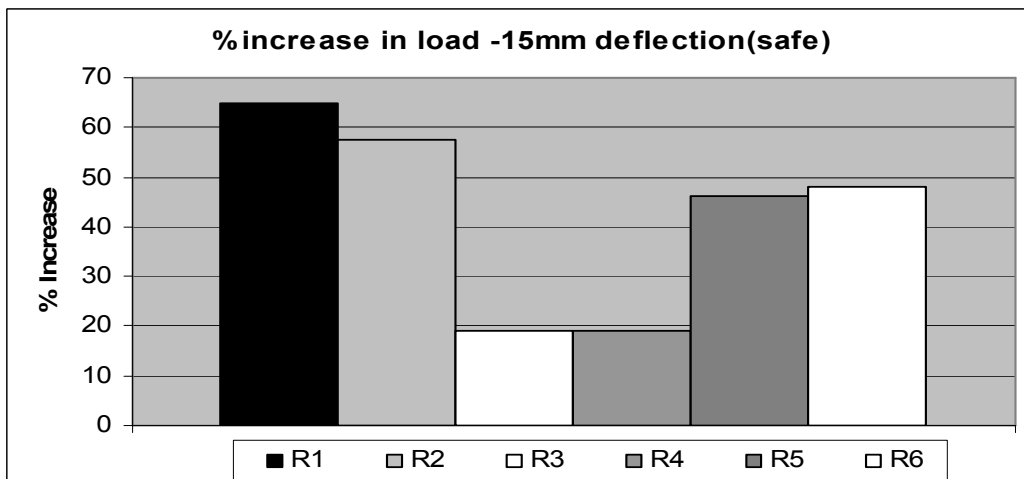


fig no 4.8 % Increase in Load -15 mm deflection(safe)

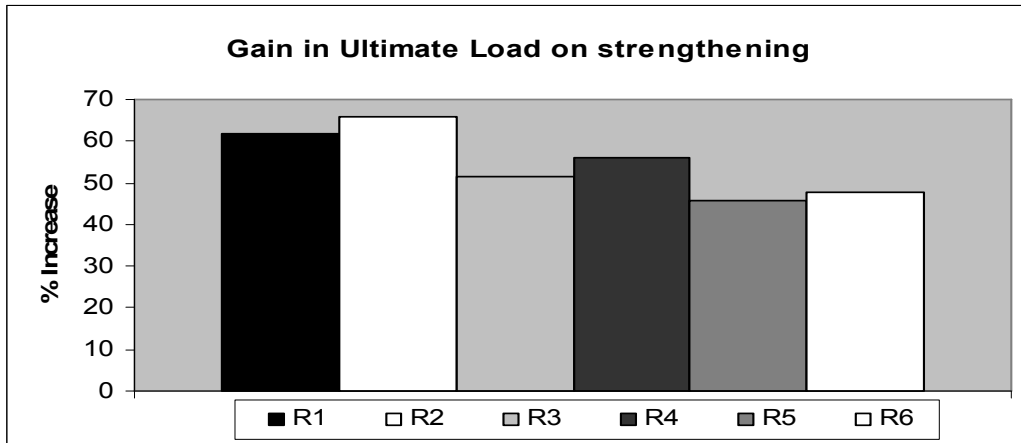


fig no 4.9 % Gain in Ultimate Load

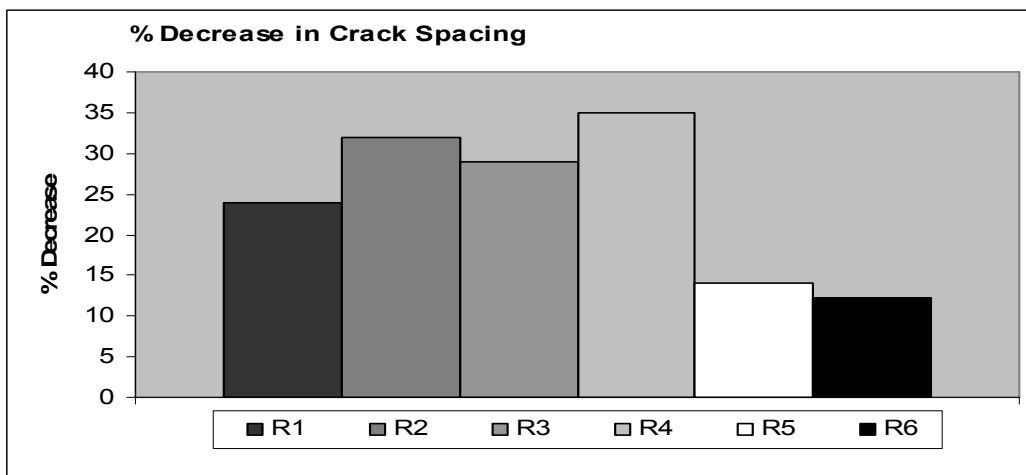


fig no 4.10 % Decrease in crack Spacing

**Table 4.1 Comparison of Experimental Crack Characteristics of Beam**

Beam Designation	Treatment (Plaster thickness)	No of Cracks	Mean Value of Crack Spacing(cm)	Max Value of Crack Spacing (cm)	% Decrease in crack spacing	Total Crack Distance from Centre (cm)	
						Left	Right
Control Beam		28	10.13	16.5		60	64
R1	20 mm	31	7.67	14	24 %	110	128
R2	20 mm	30	6.86	16	32 %	104	102
R3	20 mm	29	7.2	17	29 %	105	105
R4	20 mm	29	6.58	17.5	35 %	95	96
R5	20 mm	30	8.7	16	14.11 %	120	118
R6	20 mm	27	8.88	17	12.3 %	120	120

**Table 4.2 Comparison of Experimental Deflection Characteristics of Beam**

Beam Designation	Treatment (Plaster Thickness)	Experimental Deflection(mm) of midspan at Ultimate Load	Load corresponding to (L/250) i.e 15mm Deflection(KN)	% Increase in load for 15mm deflection (permissible)
Control Beam		44.85	13	
R1	20 mm	38.5	21.5	65 %
R2	20 mm	55	20.5	57.7 %
R3	20 mm	45	15.5	19.2 %
R4	20 mm	52.2	15.5	19.2 %
R5	20 mm	31.3	19	46 %
R6	20 mm	36.75	18	38 %

**Table 4.3 Comparison of Experimental Ultimate Loads of Beams tested in Shear**

Beam Designation	Exp Load Ult (KN)	Gain in load Ult on strengthening ( KN)		Predicted Failure Mode (Theoretical)	Actual mode of Failure
		Value	% increase		
Control Beam	21.8	-		Combined Shear&Flexure	Combined Shear& Flexure
R1	35.3	13.5	61.9 %	Combined Shear&Flexure	Combined Shear&Flexure
R2	36.2	14.4	66 %	Combined Shear&Flexure	Combined Shear&Flexure
R3	33	11.2	51.3 %	Combined Shear&Flexure	Combined Shear&Flexure
R4	34	12.2	55.9 %	Combined Shear&Flexure	Combined Shear&Flexure
R5	31.8	10	45.8 %	Combined Shear&Flexure	Combined Shear&Flexure
R6	32.2	10.4	47.7 %	Combined Shear&Flexure	Combined Shear&Flexure

**Table 4.4 Comparison between Beam**

**(R1) and Control Specimen**

Sr No	Control Beam			60 % STRESSED		
	Load (KN)	Deflection (mm)		Load (KN)	Deflection (mm)	
		L/4	L/2		L/4	L/2
1	2	0.8	1.5	2	0.1	1.4
2	4	1.82	3	4	0.85	3
3	6	3.02	5	6	1.75	4.2
4	8	5	8.3	8	3	5.3
5	10	7	10.98	10	4	6.4
6	12	9.22	14	12	5	7.2
7	14	11.2	17	14	6.5	8.2
8	16	13.5	20	16	7.6	10.2
9	18	14	21.5	18	9	11.5
10	20	16	23	20	10.4	13.2
11	21.8	33.4	44.85	22	11.7	16
12				24	12.1	17
13				26	14.7	19
14				28	16.4	21
15				30	18.4	22.1
16				32	20.9	27.5
17				34	22.3	29.7
18				35.3 KN	28.3	38.7

**Table 4.5 Comparison between Beam (R2) and Control Specimen**

Sr No	Control Beam			60 % STRESSED		
	Load (KN)	Deflection (mm)		Load (KN)	Deflection (mm)	
		L/4	L/2		L/4	L/2
1	2	0.8	1.5	2	.3	1.2
2	4	1.82	3	4	1.2	2.7
3	6	3.02	5	6	2.6	4.6
4	8	5	8.3	8	4	5.8
5	10	7	10.98	10	4.8	7.5
6	12	9.22	14	12	5.9	8.8
7	14	11.2	17	14	7.2	10.5
8	16	13.5	20	16	8.4	12.5
9	18	14	21.5	18	9.7	13.5
10	20	16	23	20	11.1	14.9
11	21.8	33.4	44.85	22	12.6	19.5
12				24	14.3	22.2
13				26	16	26
14				28	17.4	29
15				30	19.3	30.3
16				32	23	33
17				34	39	49
18				36	48	53
19				36.2 KN	50	55

Sr No	Control Beam			75 % STRESSED		
	Load (KN)	Deflection (mm)		Load (KN)	Deflection (mm)	
		L/4	L/2		L/4	L/2
1	2	0.8	1.5	2	0.5	0.6
2	4	1.82	3	4	0.9	1.2
3	6	3.02	5	6	2.3	2.8
4	8	5	8.3	8	3.6	5.4
5	10	7	10.98	10	5.1	7.6
6	12	9.22	14	12	6.4	10
7	14	11.2	17	14	7.9	14
8	16	13.5	20	16	9.6	16
9	18	14	21.5	18	11.4	16.8
10	20	16	23	20	13.3	19.3
11	21.8	33.4	44.85	22	15.3	22.8
12				24	16.5	24.2
13				26	18.7	27.6
14				28	21.5	31.2
15				30	27.8	35.8
16				32	32.4	42
17				33 KN	33	45

**Table 4.6 Comparison between Beam (R3) and Control Specimen**

**Table 4.7 Comparison between Beam (R4) and Control Specimen**

Sr No	Control Beam			75 % STRESSED		
	Load (KN)	Deflection (mm)		Load (KN)	Deflection (mm)	
		L/4	L/2		L/4	L/2
1	2	0.8	1.5	2	0.3	0.5
2	4	1.82	3	4	0.7	1.1
3	6	3.02	5	6	2	3
4	8	5	8.3	8	3.7	5.1
5	10	7	10.98	10	5.3	7.3
6	12	9.22	14	12	6.8	9.6
7	14	11.2	17	14	8.7	11.8
8	16	13.5	20	16	10.4	16
9	18	14	21.5	18	12.4	17
10	20	16	23	20	14.6	19.7
11	21.8	33.4	44.85	22	15.9	21
12				24	18.3	24.2
13				26	20.7	26.7
14				28	23.8	31.8
15				30	26.6	35.6
16				32	27.6	36.8
17				34 KN	37	52.2

**Table 4.8 Comparison between Beam (R5) and Control Specimen**

Sr No	Control Beam			90 % STRESSED		
	Load (KN)	Deflection (mm)		Load (KN)	Deflection (mm)	
		L/4	L/2		L/4	L/2
1	2	0.8	1.5	2	0.5	0.6
2	4	1.82	3	4	1	1.3
3	6	3.02	5	6	2.4	3.4
4	8	5	8.3	8	3.6	4.6
5	10	7	10.98	10	4.8	6.9
6	12	9.22	14	12	6.1	8.9
7	14	11.2	17	14	7.7	10.3
8	16	13.5	20	16	8.9	12.7
9	18	14	21.5	18	11.4	14.5
10	20	16	23	20	13.4	15.8
11	21.8	33.4	44.85	22	14.3	18.6
12				24	16	21
13				26	17.5	23
14				28	20	27.2
15				30	22	30
16				31.8 KN	23.3	31.3

**Table 4.9 Comparison between Beam (R6) and Control Specimen**

Sr No	Control Beam			90 % STRESSED		
	Load (KN)	Deflection (mm)		Load (KN)	Deflection (mm)	
		L/4	L/2		L/4	L/2
1	2	0.8	1.5	2	0.3	0.6
2	4	1.82	3	4	1.35	3
3	6	3.02	5	6	2.7	4.7
4	8	5	8.3	8	4.1	6.6
5	10	7	10.98	10	5.5	8.3
6	12	9.22	14	12	7	10.5
7	14	11.2	17	14	8.6	12.7
8	16	13.5	20	16	10.2	14.1
9	18	14	21.5	18	12	17.4
10	20	16	23	20	13.3	19.8
11	21.8	33.4	44.85	22	15.85	22.3
12				24	18	25
13				26	20	28
14				28	22.7	31
15				30	25.1	35.3
16				32.2 KN	26.5	36.7



**Table 4.10 Cost Analysis**

Material	Rate (Rs.)	Cost (Rs.) of Beam type						
		C1	R1	R2	R3	R4	R5	R6
<b>Concrete Ingredients</b>								
Cement (kg)	215	215						
Rebars (kg)								
10mm	30.10	148.7						
8mm	30.75	97.14						
6mm	33.75	1115						
Coarse Aggregates (cft)	14.0	50.89						
Fine aggregates (cft)	17.0	29.56						
Labour for control beams		200						
Cost of ingredients		852.8	852.8	852.8	852.8	852.8	852.8	852.8
<b>Retrofitting material</b>								
Welded Wire mesh	Lump sum	-	420	420	420	420	420	420
Cement, Fineagg	Lump sum	-	143	143	143	143	143	143
Labour	Lump sum	-	192	192	192	192	192	192
Retrofitting Cost		-	755	755	755	755	755	755
<b>Total Amount</b>	<b>Lump sum</b>	<b>852.8</b>	<b>1607.8</b>	<b>1607.8</b>	<b>1607.8</b>	<b>1607.8</b>	<b>1607.8</b>	<b>1607.8</b>
Increase in cost (%)		-	88.53	88.53	88.53	88.53	88.53	88.53
Increase in strength (%)		-	61.9	66	51.3	55.9	45.8	47.7
Strength/Cost Ratio			0.69 liii	0.74	0.57	0.63	0.51	0.53

**Table 4.11 Ductility Ratio and Energy Absorption**

Sr No.	Beam Designation	P <sub>Ult</sub> (KN)	Max Deflection (mm)	Ductility Ratio*	Energy Absorption* (KN-m)
1	Control Beam	21.8	44.85		691.5
2	R1	35.3	38.7	0.86	930
3	R2	37.1	55	1.22	1287
4	R3	33	45	1.003	891
5	R4	34	52.2	1.16	1125
6	R5	31.8	31.3	0.69	512
7	R6	32.2	36.7	0.81	611

\* *Ductility ratio is defined as ratio of deflections of retrofitted and un-retrofitted beam at ultimate load*

\*\* *Area under the load deflection curve*

EXPERIMENTAL RESULT PLATES

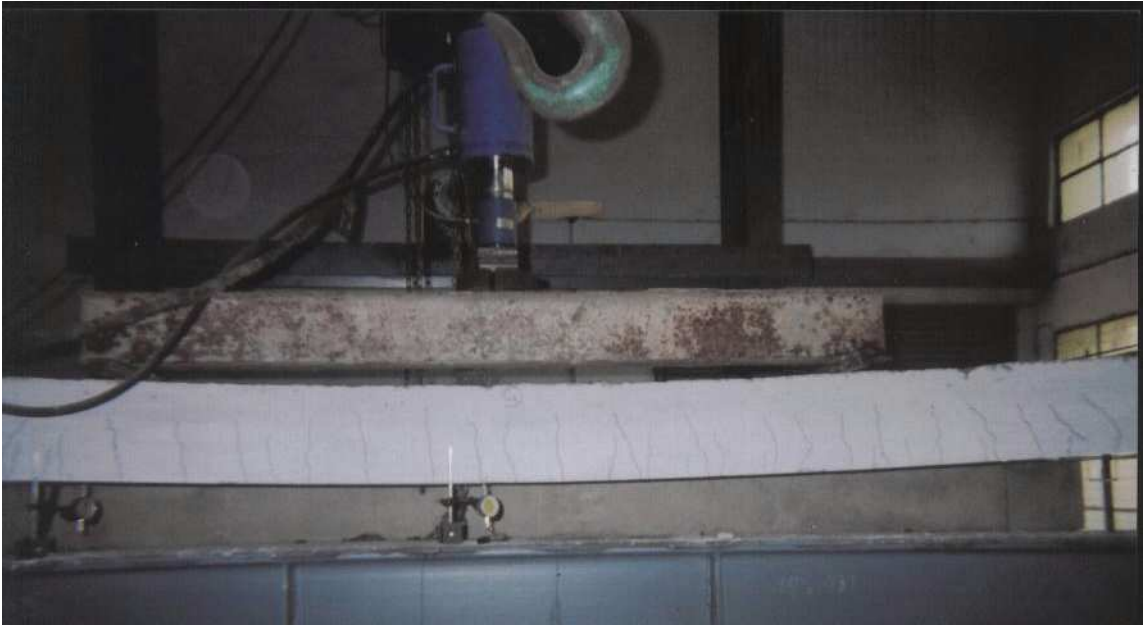


PLATE 4.1 CONTROL SPECIMEN

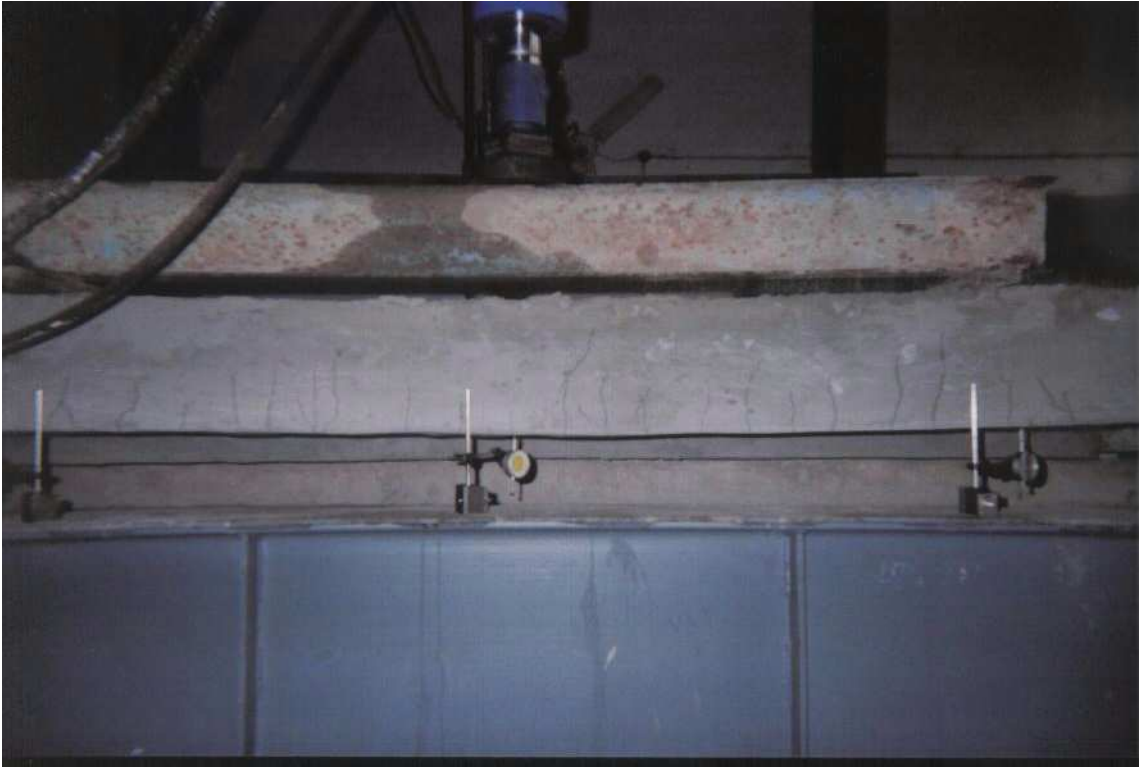


PLATE 4.2 SPECIMEN R1 (60 % STRESSED)



PLATE 4.3 SPECIMEN R2 (60 % STRESSED)

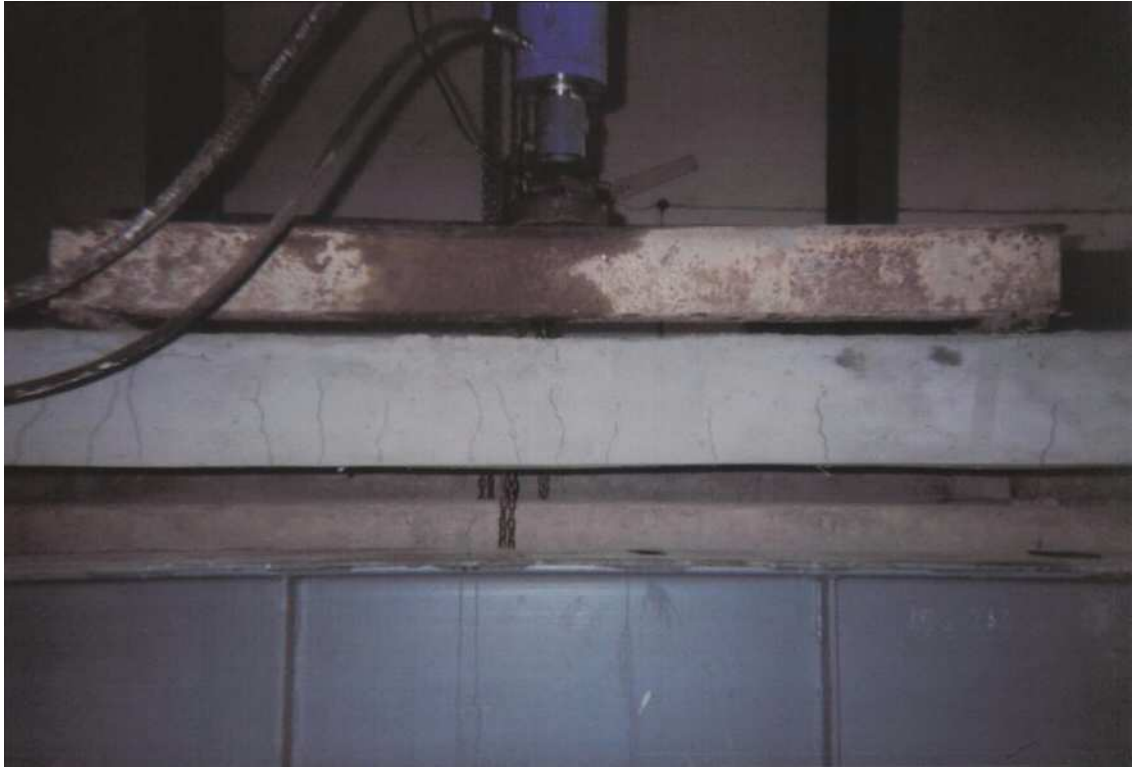


PLATE 4.4 SPECIMEN R3 (75 % STRESSED)



PLATE 4.5 SPECIMEN R4 (75 % STRESSED)



PLATE 4.6 SPECIMEN R5 (90 % STRESSED)



PLATE 4.7 SPECIMEN R6 (90 % STRESSED)

## **CHAPTER 5**

### **CONCLUSIONS**

Based upon the results of experimental study carried out the following conclusions can be drawn :-

1. The beams retrofitted with wiremesh for different stress levels do not debond when loaded to failure.
2. The failure of composite is characterized by development of shear and flexural cracks over the tension zone. The spacing of cracks is reduced for beams retrofitted with wiremesh at  $45^{\circ}$  for different stress levels indicating better distribution of stress.

3. Retrofitted beam corresponding to stress level of 60% has the highest load carrying capacity as compared to other specimen depicting that increasing stress levels contributes to strengthening in decreasing order.
4. After retrofitting all the specimen showed reduced crack width ,large deflection at the ultimate load,a significant change in Ductility ratio and considerable increase in energy absorption as well making the different specimen useful for application in componenets subjected to dynamic loads.
5. Beams retrofitted corresponding to stress level of 60% showed the maximum strength/cost ratio depicting that the specimen R1 and R2 are the most efficient as they are having the maximum strength/cost ratio of 0.69, 0.74 respectively.

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