Retrofitting of RC Beams by Prestressing of Fibres

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CERTIFICATE

This is to certify that the thesis titled, "**Retrofitting of RC beams by prestressing Fibre**", being submitted by **Mr. Harkamaljeet Singh Gill** , in partial fulfillment of the requirement for the award of degree of **MASTER OF ENGINEERING (STRUCTURAL ENGINEERING)** in the **Department of Civil Engineering, Thapar University, Patiala,** is a bonafide work carried out by him under our guidance and supervision and that no part of this desertation report has been submitted for the award of any other degree.

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"It is difficult when it comes to thanks giving, but then it is the only way of expressing your gratitude towards the people who you are thankful to and who through a small encounter become your ideals".

A single person alone can never is credited for performing any extraordinary work successfully. It is only possible with the continuous and constant help and guidance that she/he receive from others.

I would like to thank my project guide Dr. Abhijeet Mukherjee (Director,Thapar University, Patiala) and Dr. Shweta Goyal (Lecturer Civil Engineering Department) who gave me the opportunity to work on this project and without their help this project is impossible.

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DATE: **Harkamaljeet Singh Gill**

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ABSTRACT

Now a days it is common observation that structures are unable to give service as much as they are expected as per design. This is because of deterioration of the concrete and reinforcements caused by environmental factors and the widespread application of deicing salts, or due to an increase in applied loads.

The Retrofitting can be used as a cost-effective alternative to the replacement of these structures and is often the only feasible solution. Fibre Reinforced Polymers (FRP) sheets or plates are well suited to this application because of their high strength-to-weight ratio, good fatigue properties, and excellent resistance to corrosion.

A lot of research has been done on the FRP as reinforcement in concrete beams. However, the amount of research conducted on FRP as a sheet is quite less and on retrofitting by prestressing is very less.

So in the thesis, we will study the effect of fibres on beams when they are pre stressed.

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CHAPTER 1

INTRODUCTION

1.1 GENERAL

Deterioration of concrete structures is one of the major problems in construction industry today. An increasing number of reinforced concrete structures have reached the end of their service life, either due to deterioration of the concrete and reinforcements caused by environmental factors and the widespread application of deicing salts, or due to an increase in applied loads. These deteriorated structures may be structurally deficient or functionally obsolete, and most are now in serious need of extensive rehabilitation or replacement. Also in many cases due to marginal design, poor construction, inferior materials etc structural members are rendered incapable of resisting the applied loads. Solutions in such cases include increasing the load carrying capacity of the affected members through strengthening. Strengthening, upgrading and retrofitting of existing structures are among the major challenges that modern civil engineering is currently facing. One of the most promising answer to these needs is the use of FRP sheets. They are well suited to this application because of their high strength-to-weight ratio, light weight, good resistance to chemicals as compared to steel, have non magnetic and non conductive properties, good fatigue properties, and excellent resistance to corrosion. Their application in civil engineering structures has been growing rapidly in recent years, and is becoming an effective and promising solution for strengthening deteriorated concrete members. Because FRPs are quickly and easily applied, their use minimizes traffic disruption and labor costs and can lead to significant savings in the overall costs of a project. CFRP sheets are widely used for external strengthening due to its high modulus and strength, and may be used more effectively by applying prestress force. The expected benefits of the prestressed CFRP sheets include the effective use of the material strength, restored prestressing, and improved serviceability, increased cracking loads, closure of existing cracks, and reduced deflections. An active load-carrying mechanism occurs which ensures that some of the dead load of the structure is carried by the prestressed CFRP sheets whereas, non prestressed sheets only support additional live loads. Despite these advantages, only limited research, regarding the application of prestressed CFRP sheets, has been reported to date.

1.2 FIBRE REINFORCED COMPOSITES (FRC)

Composite can be defined as ‘two or more dissimilar materials which when combined are stronger than the individual materials.’ Composites can be both natural and synthetic (or man-made) and as materials technology moves toward more sustainable solutions, the focus on the use of organic, or natural materials, especially as reinforcements, increases each year. Wood is a good example of a natural composite which is a combination of cellulose fiber and lignin. The cellulose fiber provides strength and the lignin is the "glue" that bonds and stabilizes the fiber. Reinforced concrete is another example of composite in which concrete and steel combines to create structures that are rigid and strong. This is a classic composite material where there is a synergy between materials. In this case, synergy means that the composite (or combination) of materials is stronger and performs better than the individual materials. Concrete is rigid and has good in compression ,while steel has high tensile strength. The result is a structure that is strong in both tension and compression. Composites are of two types, one is particle based and other is fiber based. Fiber based composites are used for civil engineering applications Fiber based composites are composed of fibers and resins. Two main types of polymer used for resins: thermosets and thermoplastics. The thermosetting polymers used in the construction industry are the polyesters and the epoxies. There are many thermoplastic resins used in composite manufacture: polyolefins, polyamides, vinylic polymers, polyacetals, polysulphones, polycarbonates, polyphenylenes and polyimides. Resin systems such as epoxies and polyesters have limited use for the manufacture of structures on their own, since their mechanical properties are not very high when compared to, for example, most metals however, they have desirable properties, most notably their ability to be easily formed into complex shapes. Fibers are added to increase the load-carrying capability of the composite material. The fibers may occupy anywhere from 40 percent to 70 recent (by volume) of the material. These fibers have relatively small diameters. For example, a typical graphite fiber diameter is on the order of 5 to 7 micrometers, while glass fibres are usually larger, on the order of 15 to 20 micrometers. A wide range of amorphous and crystalline materials can be used as the fiber. In the construction industry the most common fibre used is glass fiber (there are 4 types of glass fibers: E-glass, AR-glass, A-glass and high strength glass). Carbon fiber, of which there are 3 types (Type I, II, III) can be used separately or in conjunction with the glass fiber as a hybrid to increase the stiffness of a structural member or the area within a structure, so that the stiffness exceeds the value that can be obtained by using glass fiber. These materials have extremely high tensile and compressive strength but in 'solid form' these properties are not readily apparent. This is due to the fact that when stressed, random surface flaws will cause each material to crack and fail well below its theoretical 'breaking point'. To overcome this problem, the material is produced in fiber form, so that, although the same number of random flaws will occur, they will be restricted to a small number of fibers with the remainder exhibiting the material's theoretical strength. Therefore a bundle of fibers will reflect more accurately the optimum performance of the material. However, fibers alone can only exhibit tensile properties along the fiber’s length, in the same way as fibers in a rope. It is when the resin systems are combined with reinforcing fibers such as glass, carbon and aramid, those exceptional properties can be obtained. The resin matrix spreads the load applied to the composite between each of the individual fibers and also protects the fibers from damage caused by abrasion and impact. High strengths and stiffnesses, ease of moulding complex shapes, high environmental resistance all coupled with low densities, make the resultant composite superior to metals for many applications.

1.2.1 Mechanical properties of FRP composites

All three types of FRP composites, namely Glass fiber (GFRP), carbon fiber(CFRP) and Aramid (AFRP) have been used for strengthening RC structures in both practical application and research. **Table 1.1** illustrates the wide variety of strength and stiffness that FRC may possess. Regardless of the type of fibers forming method employed, all these thin FRP materials have the same stress-strain behavior: linear elastic up to brittle rupture when subjected to tension. This is very important property in terms of structural use of FRP composites.. Apart from illustrating typical strength differences between these materials, these curves give a clear contrast between the brittle behavior of FRP composites and the ductile behavior of steel. This has two major structural consequences. First, these materials do not possess the ductility that steel have and their brittleness may limit the ductile behavior of RC members strengthened with FRP composites. Nevertheless, when used to provide confinement for concrete, these materials can greatly enhance the strength and ductility of columns. The second implication of brittle behavior of FRP composites is that redistribution of stresses is restricted owing to this lack of ductility. Consequently, the design of structures bonded with FRP composites can not allow the use of existing methods meant for analysis for RC structures with FRPs simply treated as equivalent steel reinforcement. Instead existing design methods for RC structures need to be modified to take this brittleness into account based on extensive research. In addition, completely new problems arise which also require extensive research.

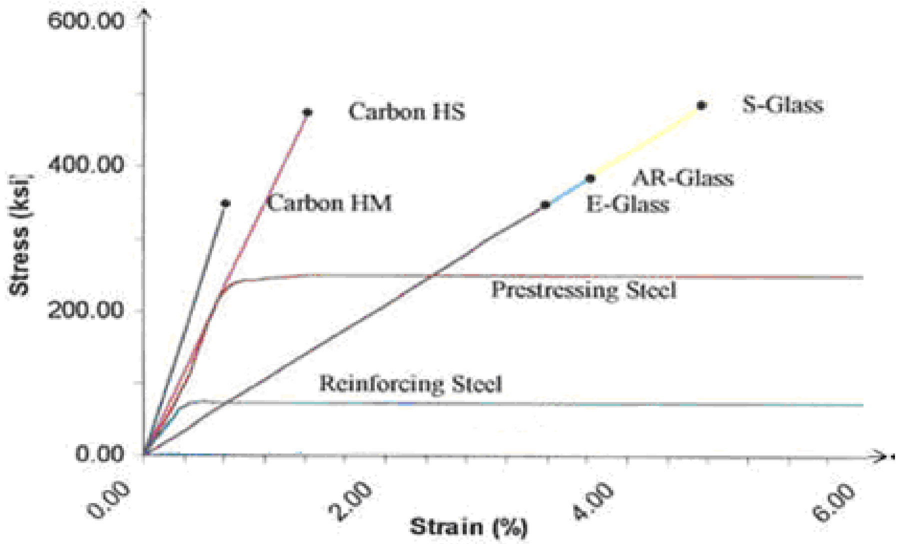


Fig 1.1 Stress strain curves for GFRP, CFRP and Mild steel

Table1.1 Typical mechanical properties of GFRP, CFRP &AFRP composites *(Head 1999)*

Unidirectional advanced composite materials	Fiber content (% BY WT.)	Density (KG/M^3)	Longitudinal tensile modulus	Tensile strength (MPA)
Glass fiber / Polyester GFRP laminate	50-80	1600-2000	20-55	400-1800
Carbon / Epoxy CFRP laminate	65-75	1600-1900	120-250	1200-2250
Aramid / Epoxy AFRP laminate	60-70	1050-1250	40-125	1000-1800

. 1.2.2 Advantages of FRP

The benefits of composite materials have fueled growth of new applications in markets such as transportation, construction, corrosion-resistance, marine, infrastructure, consumer products, electrical, aircraft and aerospace and appliances and business equipment. The benefits of using composite materials include:

High Strength – Composite materials can be designed to meet the specific strength requirements of an application. A distinct advantage of composites over other materials is the ability to use many combinations of resins and reinforcements, and therefore custom tailor the mechanical and physical properties of a structure.

Light Weight – Composites are materials that can be designed for both light weight and high strength. In fact, composites are used to produce the highest strength to weight ratio structures known to man.

Corrosion Resistance – Composites products provide long-term resistance to severe chemical and temperature environments. Composites are the material of choice for outdoor exposure, chemical handling applications, and severe environment service.

Design Flexibility – Composites have an advantage over other materials because they can be molded into complex shapes at relatively low cost. The flexibility of creating complex shapes offers designers a freedom that hallmarks composites achievement. Composites can be custom tailored to have strength in a specific direction. If a composite has to resist bending in one direction, most of the fiber can be oriented at 90^0 to the bending force. This creates a very stiff structure in one direction. What actually happens is that more of the material can be used where it counts. With metals, if greater strength is required in one direction, the material must be made thicker overall, which adds weight. Also *Tailorability* is the added advantage in FRP.

Durability – Composite structures have an exceedingly long life span. Coupled with low maintenance requirements, the longevity of composites is a benefit in critical applications. In a half-century of composites development, well-designed composite structures have yet to wear out.

1.2.3 Applications

It has many applications in aerospace and automotive fields, as well as in sailboats, and notably in modern bicycles and motorcycles, where these qualities are of importance. It is becoming increasingly common in small consumer goods as well, such as laptop computers, tripods, fishing rods, paintball equipment, racquet sports frames, stringed instrument bodies, classical guitar strings, and drum shells.

The main areas of application of FRP composites In Civil Engineering are:

- ***FRP Column wrapping***

FRP systems have been used extensively in seismic zones for confinement of concrete columns and walls. A number of FRP systems have been qualified for use for wrapping circular and rectangular bridge columns. Improvements in ductility factors of up to 10 fold have been achieved through the use of FRP column wrapping.

The majority of structural deficiencies in existing concrete columns can be attributed to lack of transverse reinforcement. This is especially true for columns in seismically active regions, designed prior to the enactment of modern seismic codes. Columns with insufficient transverse reinforcement suffer; i) premature shear failure, ii) brittle crushing of unconfined concrete and iii) reinforcement splice failure if the longitudinal reinforcement is spliced at or near a potential plastic hinge region. FRP sheets provide an excellent opportunity to enhance column resistance in all three areas of weakness. The jackets often consist of sheets wrapped around columns with fibers oriented in the transverse direction, though they may also be used in the longitudinal direction for flexural strength enhancement. Although surface bonded, the FRP sheets overlap when wrapped around a column, easily developing the required strength without the possibility of surface delamination.

One of the advantages of using FRP composites to seismically retrofit bridge columns is ease of application. Application can be done by hand, or for more rapid process, by an automated technique. One method of application constructs a hoop-wrapped jacket around a RC column using tows of continuous carbon fiber pre-impregnated with resin.

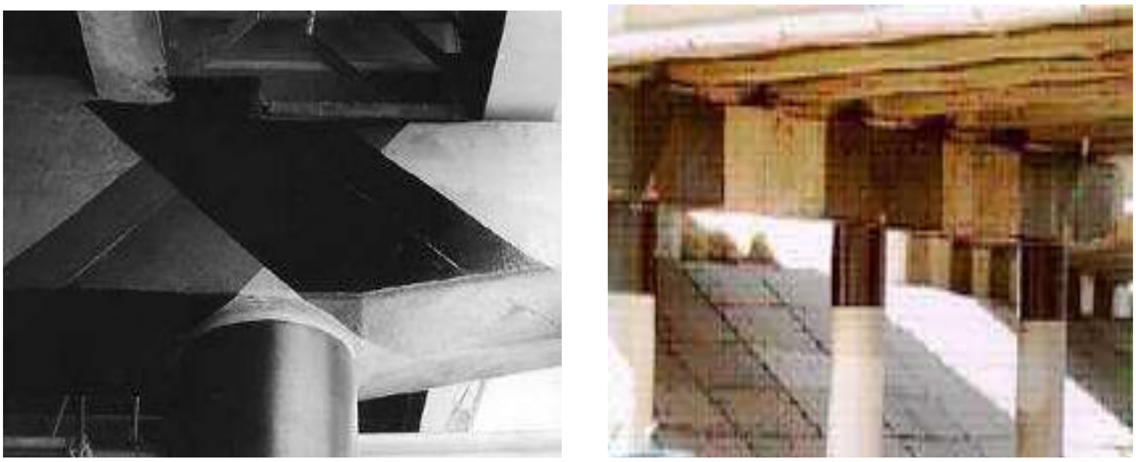


Fig 1.2 Bridge column and bent cap wrapped with FRP

Automated machines can wrap pre-preg tow to precise dimensions around highway and bridge columns. The machine rotates around the column while it moves up and or down, encasing the entire column with carbon fiber pre-preg .Because the carbon fiber is continuous, the wrap created provides uniform confinement of the concrete. This ensures there are no weak spots where the shear strength and flexural strength would be low. A radiant heat oven cures the resin at high temperatures. The resin also acts as an adhesive and bonds to the concrete forming a tight structure around the entire cross section of the column.**(Fig 1.3) .**

- ***Bridges***

Tavakkolizadeh et al. (2002) conducted a study on behavior of damaged steel-concrete composite girders repaired with CFRP sheets under static loading. A total three large scale composite girders were prepared and tested. Two parameters were varied in the experimental program. First is the number of CFRP layers and second is the specimen with percentage loss

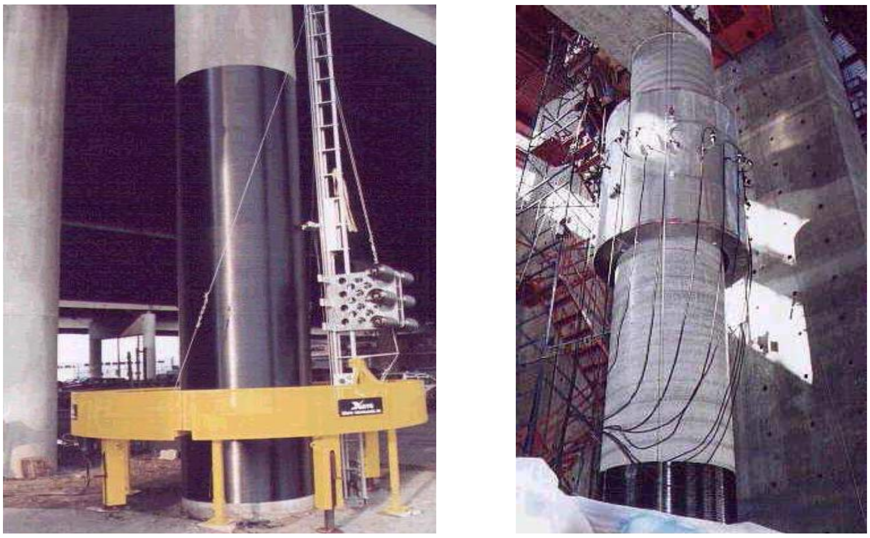


Fig 1.3 Automated column wrapping system and Curing the FRP wrap

of the cross sectional area of their tension flange. The test result showed that epoxy bonded CFRP sheet could restore the ultimate load carrying capacity and stiffness of damaged steel-concrete composite girders. CFRP parallel wire bundles are used as a stay cables in cable stayed bridges. CFRP stay cables each with a load capacity of 12 MN were installed in 124 m span two lane Stock bridge at Winterthur railway station. (Meier (2002)).

• ***Strengthening of historic buildings***

Garden et al. (2002) strengthened the beams of the Boots building, high street, Nottingham, constructed in 1903. Beams had suffered a loss of flange and web section of approximate 30% due to electrochemical corrosion. The laminates were installed using vacuum bag technology, using a low temperature moulding (LMT) prepeg, in which material was held under vacuum pressure against the beams profile and local temperature was raised to promote full cure of epoxy resin matrix. From the study it can be said that steel and cast iron structures can easily be rehabilitated using composite materials, irrespective of the geometry and complexity of shape.

• ***Foundation piles***

Composite piles were first used in the late 1980s as replacements for timber fender piles at the Port of Los Angeles, USA. The first composite pile prototype was driven in 1987 and consisted of a composite steel pipe encased by recycled plastic. Since the first composite pile application in 1987, several other projects have used composite piles.

- ***Seismic retrofitting***

Structural parts like beams, columns and beam- column joints damaged by seismic forces can be retrofitted by CFRP.

- ***Cladding and facades***

FRPs are successfully working as a cladding and facades in modern buildings.
Other applications are

  - Fencing
  - Masts
  - Pipes
  - Roofing
  - Tanks (Swimming pools)
  - Towers & Domes.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

A lot of research has been done on the FRP as reinforcement in concrete beams. However, the amount of research conducted on FRP as a sheet is quite less and the application of prestressing to the CFRP sheet is still lesser. Use of prestressed FRP composites for flexural strengthening of concrete and metallic beams and slabs has developed over recent years. Main reasons for using prestressed system for strengthening structural elements are:

- To increase live load capacity
- To reduce dead load deflections
- To reduce crack widths in concrete elements and to delay onset of cracking
- To improve the behavior of the structural elements at servicebality
- To improve the fatigue strength by minimizing the absolute tensile stress.

This study deals with the literature review of the work already done on the use of FRP in strengthening structural members. The major emphasis is laid on prestressed FRPs.

2.2 RETROFITTING OF RC BEAMS BY USING FIBRES

2.2.1 Retrofitting of Beams without Prestressing FRP

***Shahawy et al (*1995)** studied the effect of CFRP laminates on reinforced concrete rectangular beams . In their study Flexural behavior of reinforced concrete rectangular beams with epoxy bonded carbon fiber reinforced plastic (CFRP) laminate was experimentally investigated. Four reinforced concrete rectangular beams of size 2.744 × 0.203×0.305 m were casted. Firstly beam was tested and was failed ,then CFRP laminates were applied to the beam. Experiment was conducted by varying the number of laminates .On first beam only one laminate was applied and consequently on others beams no of laminates applied were increased. It was observed that cracking moment for the laminated beams was significantly higher than that of the control beam. The percentage increase in the measured cracking moment was 12%, 61% and 105% for the beam with one, two and three CFRP laminate layers, and the ultimate capacity increased was 13% for beam with one layer of CFRP, 66% and 92 % for the beams with two and three layers of CFRP. Also the moment capacity increased significantly with the increase in number of CFRP laminates which means CFRP laminates resulted in a significant increase in beam stiffness.

Esfahani et al (2007) investigated the Flexural behaviour of reinforced concrete beams strengthened by CFRP sheets. In this research they found the effect of reinforcing bar ratio on the flexural strength of the strengthened beams. Reinforcing bar ratio was the main factor that was considered in the research. Twelve concrete beam specimens were casted having dimensions of 150 mm width, 200 mm height, and 2000 mm length .Three different reinforcing ratios were used in these beam sections .Nine specimens were strengthened in flexure by CFRP sheets and the other three specimens were considered as control specimens. The width, length and number of layers of CFRP sheets were varied in different specimens. Bars of size 8,10,12,16 and 20 mm were used in the specimens. CFRP sheets were used for strengthening the beams and adhesive used was hand mixed epoxy. Reinforcing bar ratio in the beams were 30%,60%,80% of the tensile reinforcement. At the top two 10 mm dia deformed bars were used in all the specimens and plain bars of 8 mm dia were used in transverse reinforcement. Number of layers of CFRP and width of CFRP layers were varied in each specimen. All the specimens were tested under the four point load system by using hydraulic Jack. Displacements measured were the mid span displacements. It was found that as the diameter of bar is increased load carrying capacity of control beams increased. Also when the beams are strengthened by CFRP they exhibit large stiffness as compared to control beams. After yielding of reinforced bars the strength and stiffness of strengthened specimen was larger as compared to control beams. After failure of CFRP, the load–displacement curve of most of the strengthened specimens dropped and almost corresponded to those of the control specimens.

Hadi (2003) studied the load carrying capacity of beams failed in shear by retrofitting them by Carbon fibers. In his research he studied two objectives, first he investigated the effectiveness of two types of wrapping material in enhancing the shear capacity of reinforced concrete beams, and the second objective was to investigate the increase in the strength and ductility of reinforced concrete beams, where their compressive zone is confined by helical reinforcement. For this work total of sixteen reinforced concrete beam specimens of dimension 1.2m ×100×150 mm were casted. The specimens were designed into four distinct groups, depending on the beams reinforcement arrangement. Group 1 consisted of beams reinforced with 2N16 (16 mm diameter bars of 500 MPa tensile strength and normal ductility) longitudinal bars. Group 2 consisted of beams reinforced with 2N16 bars and helices within the compressive zone. While Groups 3 and 4 consisted of 2N20 and 2N24 longitudinal bars respectively each with helical reinforcement within the beams in compressive region. 8 beams were retrofitted with CFRP and 8 beams with E glass fiber. Under reinforced beams had two wrapping layers of CFRP and E glass and Over reinforced beams and balanced reinforced beams had three layers of CFRP and E glass fiber. All testing specimens were subjected to four-point loading. The tested beam specimens which were deficient in shear strength and did not achieve their ultimate flexural strength under loading and was to retrofit in shear strength, the strengthening materials were only applied on the beam at pure shear span as shown in **Fig2.1.**

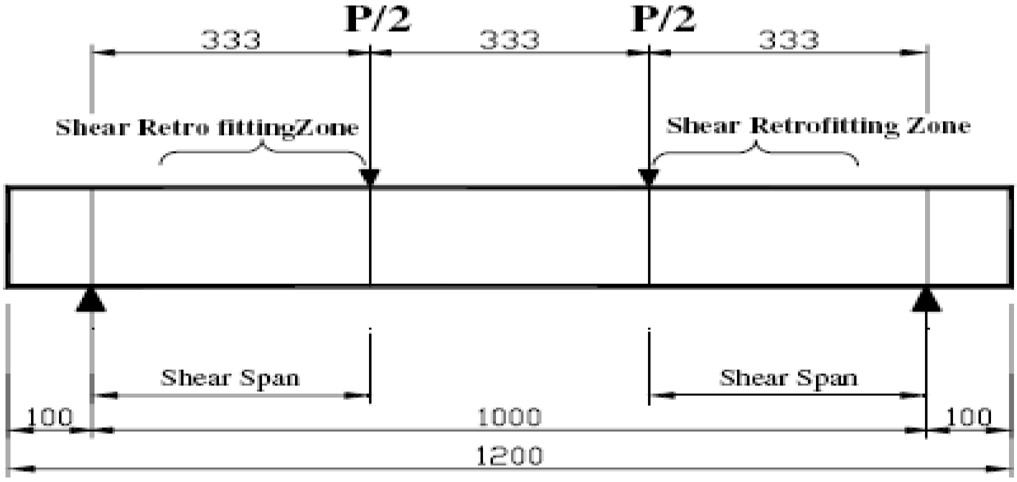


Fig 2.1 Shear Strengthening Configuration (Hadi, 2003)

After all the testing, he concluded that the inclusion of the helical reinforcement proved to increase the performance of the beam in both load carrying capacity and flexural strength, a 3% of strength increment was observed from the test results for beam that failed in bending . FRP materials contributed to beams load carrying capacity and flexural strength, it is indicated that retrofitting with FRP provides a feasible rehabilitation technique for repair as well as strengthening and beams strengthened with CFRP display an increase in the beams maximum flexural strength of up to 31% higher compared to that of beams strengthened with

E-glass . Balanced and over reinforced beams which were strengthened with CFRP, generally failed in bending, which shows that the CFRP material out performs the Eglass material in terms of external shear reinforcement.

Rajamohan et al (2009) carried out the research by strengthening the RC beams deficient in shear by using GFRP sheets. In this study the response of RC beams strengthened in shear using bi-directional GFRP fabrics was found out. The retrofitting was done by two ways

1. Using inclined side GFRP strips
2. By providing inclined U-strips of GFRP.

This experiment was aimed at understanding the best wrapping style for retrofitting the deficient beams. In his study five control beams were taken having cross-sectional dimensions of 100 mm ×150 mm and 1000 mm length. From these five beams one beam was fully strengthened. But the other four beams were so designed such that they were shear deficient. The experimental program aimed at raising the strength of the shear deficient beams to that of the fully strengthened beams by externally bonding inclined GFRP strips to the beams. These beams were then raised to the strength of that of the fully strengthened beams by externally bonding the beams with GFRP strips on sides as well as using U-wrap fashion. For the testing of these beams two point loading was adopted. For testing three sets of beams were casted in which one set was of control beam, second set was of beams which were externally bonded with inclined GFRP strips on the sides of shear span and third set of beams were those which was given inclined U-wrap of GFRP strips in the shear span as shown in **Fig 2.3.**

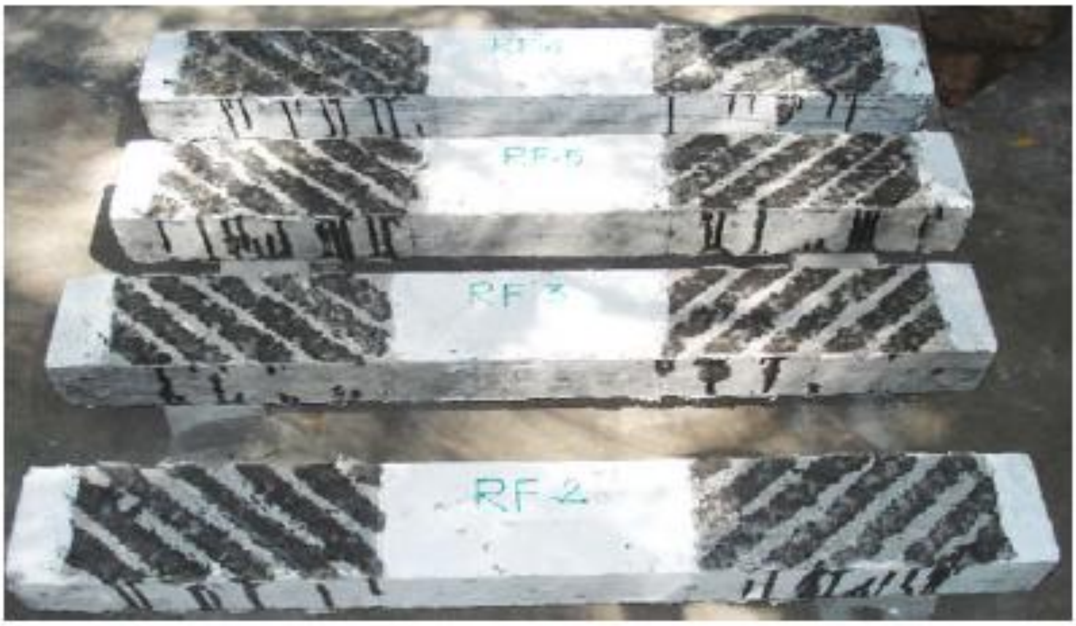


Fig 2.2 GFRP inclined strips of various widths.

Fig 2.3 GFRP inclined U strips of various widths

Fig 2.4 Experimental Setup

After testing failure mode and load carrying capacities were observed. Following type of failures were observed, shear failure due to GFRP rupture,shear failure without GFRP rupture, crushing of concrete at the top and flexure failure. Flexural kind of failure was prominent when retrofitting was done using both the wrapping schemes with inclined GFRP strips. Beams retrofitted with GFRP inclined U strips had flexure cracks caused due to rupture of FRP. The retrofitted beams when tested for their ultimate loads were found to have greater load carrying capacity than their corresponding control beams. It was noted that the all the retrofitted beams had ultimate load carrying capacity similar to that of the fully strengthened beam. This is due to the use of GFRP strips. Maximum percentage of increase in ultimate strength of 50% was observed in the beams retrofitted with GFRP Inclined strips For the beams retrofitted with GFRP Inclined U strips 50% increase in strength was observed. Initial cracks were delayed in shear deficient beams retrofitted with GFRP strips as compared to their respective control beams. It showed that use of GFRP strips are more effective in the case of strengthening of structures in shear. The ultimate strength of beams can be increased by the use of GFRP inclined strips. The ultimate loads of beams retrofitted with U-wrapping were greater than the beams retrofitted by bonding the GFRP strips on the sides alone as shown in **Fig2.5.** The load carrying capacity of the retrofitted beams were found to be greater than that of the control beams, thus the externally bonded FRPs were able to help in taking more load.

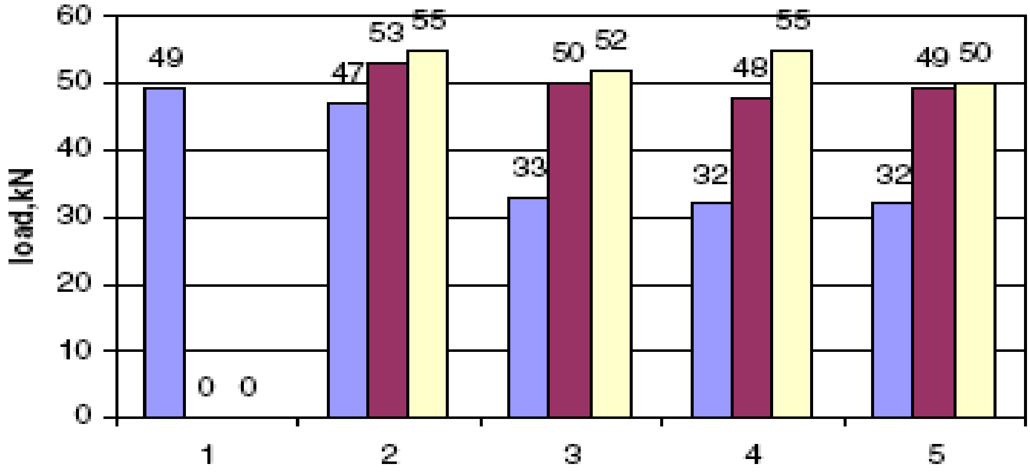


Fig 2.5 Ultimate Loads taken by control beams and retrofitted beams

Blue Graph -Control beams
Red Line –Beams retrofitted with GFRP Inclined Strips
Yellow Line-Beams Retrofitted with GFRP U Inclined Strips.

Naaman et al. (2001) studied parameters influencing flexural response of RC beams strengthened using CFRP sheets. The experimental program comprised of 14 RC T beams as shown in **Fig 2.6**. The test parameters included two levels of steel reinforcement ratio before strengthening and up to four levels. It was observed that beams strengthened with CFRP interfacial shear failure that occurs within concrete, instead of tensile failure of CFRP sheet or plate. Ultimate load capacity was increased and deflection was reduced. If cover is less for a strengthened beam then also there is not much need of consideration. Also preloaded and precracked beam beyond reinforcement yielding had no serious influence on strengthening effect, So CFRP bonding technique can be applied to seriously damaged beams. Although numerous factors can affect extent to which a RC beam can be strengthened for bending using CFRP laminates, it seems to be safe to design for increments of bending strength not exceeding about 20 % of nominal bending resistance of the beam calculated assuming reinforcement ratio equal to ρ_{max}, where ρ_{max} is maximum reinforcement ratio as defined in ACI building code. This limitation arises from the concerned that failure should occur at the interface, it will not significantly damage the concrete cover.

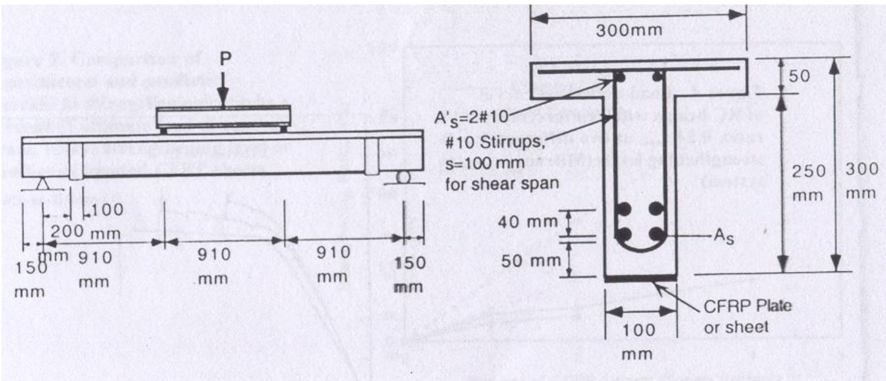


Fig 2.6 Typical cross section of beam and testing set up

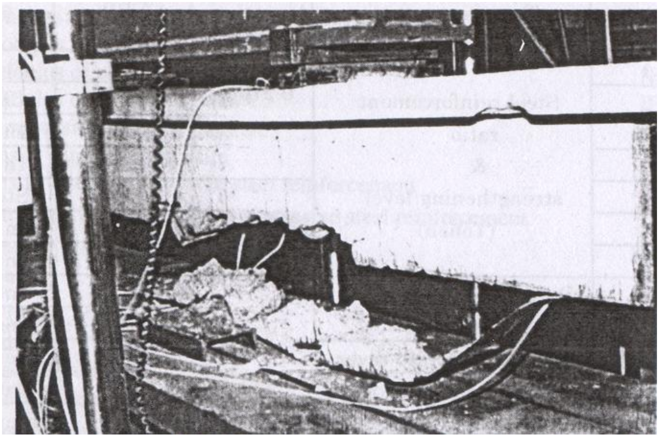


Fig 2.7 Interfacial shear failure of concrete and spalling of concrete cover (Tooth type failure) in beam.

In analyzing or designing a strengthened RC beam, design criteria should not only include equilibrium, strain compatibility and stress strain relations of component materials, but also criteria related to interlaminar shear resistance of the concrete both in direction of bending and shear.

Duthinh et al. (2002) studied strength & ductility of RC beams wrapped with CFRP. Refering to work carried out by ***Naaman et al. (2001)*** study was carried out. In the tests seven concrete beams reinforced internally with varying amounts of steel & externally with precured CFRP plates after the concrete had cracked under service loads. Curvature was computed in constant moment region by measuring strains.

From the test results, it was observed that Carbon FRP plates are very effective for flexural strengthening of RC beams, provided proper anchorage of FRP is ensured. As the amount of steel reinforcement increases, additional strength provided by CFRP reinforcement, decreases. Compared to a beam reinforced heavily with steel only, beams reinforced with both steel and carbon have adequate deformation capacity, in spite of their brittle mode of failure. Clamping or wrapping of the ends of the precured FRP plate enhances the capacity of adhesively bonded FRP anchorage.

Niu et al.(2006) studied the effect of interface bond properties on the performance of FRP-strengthened reinforced concrete (RC) beams in terms of concrete cracking, interface stress transfer. FRP bonding technology highly depends on bond integrity between concrete & FRP.

A CFRP strengthened RC beam subjected to three-point bending ***Wu and Kurokawa (2002)*** is analyzed using the commercial finite element program DIANA. Deflections were measured against loads. From the experiments it was observed that relatively low stiffness may be helpful to distribute more uniform stresses in both steel and FRP sheets, which may help to relieve local stress concentrations and reduce the likelihood of debonding in practice. Interfacial bond strength influences the yield load and to a less extent, the ultimate load-carrying capacity. High bond strength may be helpful to distribute cracks and thus increase the effectiveness of FRP strengthening.

Heffernan & Erki(2004) investigated fatigue behavior of reinforced concrete beams Post strengthened with CFRP laminates. For this twenty reinforced concrete beams, 150 X 330 X 3000 mm, were casted. The CFRP sheets were cut to 125 X 2650 mm and applied in accordance with the specifications of the manufacturer. The fatigue life of a CFRP strengthened reinforced concrete beam appeared to be at least as long as for an equivalent strength conventionally reinforced concrete beam subjected to the same loads, where that fatigue life is largely dependent on the stress range applied to the steel reinforcement. The results indicated that concrete softening due to repeated loads leads to an increase in the stresses in the tensile steel reinforcement. These increases in steel stresses were not as severe for the CFRP strengthened beams as for the beams without CFRP sheets. No significant degradation in the CFRP sheets or the CFRP to concrete interface occurred due to cyclic loading, and the basic assumptions for monotonic behavior remained valid for beams loaded cyclically.

Li et al.(2005) performed experimental and numerical analysis to predict the loading carrying capacity of reinforced concrete beams strengthened with carbon fiber reinforced plastics

(CFRP) composites. Four-point bending test was carried out for rectangular beams in a large testing frame of 2000 KN capacity. Dimensions of the beams were b × h = 120 × 200mm, length = 2000mm, clear span = 1800mm, which were designed as under reinforced. From the tests, it was concluded that CFRP can effectively increase initial cracking loads, ultimate loads, stiffness and ductility of concrete beams and improve crack patterns. The distance from the end of fiber to the support point is the main influencing parameter on debonding failure when a single layer fiber is used for strengthening. When the two-layer fibers are used for strengthening, the effect of increase of the length of the second layer of the fiber on performance of beams approaches a constant value if the length of the second layer reaches some limit, CFRP strengthening will have a low ratio of performance to cost under this condition. Debonding failure of concrete beams strengthened with CFRP occurs before the normal ultimate load, and the high strength property of CFRP cannot be fully utilized. Debonding failure has greater influence on initial cracking loads than on stiffness, ductility and ultimate loads of concrete beams and it has a lesser influence on crack patterns, but it does affect these behaviour significantly. They concluded that it will greatly influence the performance of strengthened concrete beams and it must be considered sufficiently during the design process. Construction procedures and anchorage design procedures may not avoid debonding failure completely.

Xiong et al.(2004) conducted the test program including six beams and two strengthening systems, namely hybrid carbon fiber glass fiber-reinforced polymer strengthening and CF-reinforced polymer strengthening were used. The beams were 125 X 200 mm in cross section and 2,300 mm in length. The process of applying a fiber sheet to concrete involved surface preparation, priming, resin undercoating, fiber sheet application, and resin over coating according to ACI 2000. It was required that the end anchorage failure would not occur in this research. The fiber sheets, therefore, were extended upward at the ends of beams. Authors concluded that test results indicated that H-CF/GF-RP strengthening can obtain both a significant increase of ductility and a remarkable decrease of strengthening cost with slight varieties in load carrying capacity and stiffness of strengthened beams. Under similar failure loads the deflection ductility, stiffness, and strengthening cost of H-CF/GF-RP strengthening beams were 89.7% higher, 10% and 38% lower than those of the CFRP strengthening beam.

Silva & biscaia (2007) studied the degradation of bond between FRP & RC beam. The effects of cycles of salt fog, temperature and moisture as well as immersion in salt water on the bending response of beams externally reinforced with GFRP or CFRP, especially on bond between FRP reinforcement and concrete was considered. Temperature cycles (-10^{o}C to 10^{o}C) and moisture cycles were associated with failure in the concrete substrate, while salt fog cycles originated failure at the interface of concrete–adhesive. Immersion in salt water and salt fog caused considerable degradation of bond between the GFRP strips and concrete. However, immersion did not lower the load carrying capacity of beams, unlike temperature Cycles that caused considerable loss. No significant differences were detected on the behavior of the systems strengthened with GFRP and CFRP, perhaps because the design of the tests impeded failure of the fibers.

Benjeddou et al.(2006) studied the damaged reinforced concrete beams repaired by external bonding of carbon fiber reinforced polymer (CFRP) composite laminates to the tensile face of the beam. Two sets of beams were tested in this study: control beams (without CFRP laminates) and damaged and then repaired beams with different amounts of CFRP laminates by varying different parameters (damage degree, CFRP laminate width, concrete strength class). All beams were tested in four-point bending over a span of 1800 mm. The beams were 120 mm wide, 150 mm high and 2000 mm long. The span of the beam (1800 mm) is limited by the testing machine configuration. After testing these beams were repaired using unidirectional carbon fibers laminates‘‘SIKA CARBODUR LAMELLE’’. Five beams were repaired with S1210 having 100 mm width and one beam was repaired with S1205 having 50 mm width.

a

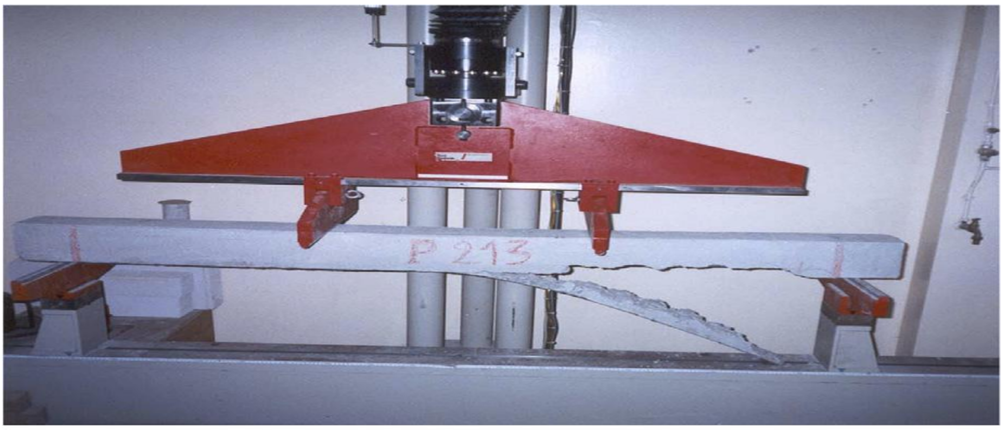


Fig. 2.8 Repaired beam failure by peeling off.(Benjeddou et al, 2006)

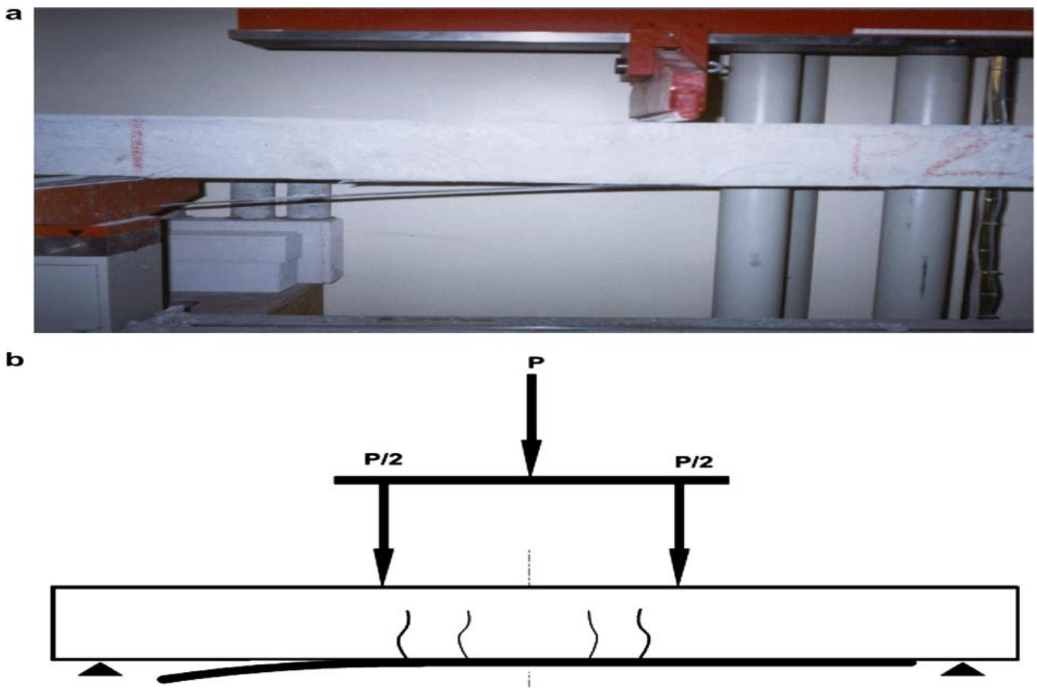


Fig.2.9 A repaired beam failure by interfacial debonding and (b) schematic drawing (*Benjeddou et al. 2006)*

Authors concluded that the mechanical performance of the repaired RC beams is highly increased by using the CFRP laminates. Therefore, this technique is effective to at least restore the mechanical performance of cracked or damaged RC beams. The laminate width affects the failure modes of the repaired beams. These failure modes change from interfacial debonding to the peeling-off when the width increases from 50 mm to 100 mm **Fig 2.8 & Fig 2.9**. Also for a load capacity improvement, reinforcement with a CFRP having about a half width of the beam is satisfactory, even when interfacial debonding occurs.

Bank and Arora (2006) have done the experimental work in which FRP strips, reinforced with a combination of carbon and E-glass unidirectional fibers and continuous strand mats, were fastened to the concrete beams with steel powder-actuated (PA) fasteners and expansion anchors (EA) and were tested to different failure mode. The strengthened RC beams were designed to fail in a ductile manner. Test results implies that the strengthened beams showed increases in yield and ultimate moments of up to 25% and 58%, respectively over an unstrengthened beam. All strengthened beams failed, as intended, in a ductile manner with the ultimate failure mode due to concrete compression failure at large deflections with the FRP strip still firmly attached.

2.2.2 Strengthening of Slabs by using FRP composites

Smith et al (2009) investigated the effects of FRP on slabs. For this research they made Six simply-supported one-way spanning RC slabs . All slabs were prismatic, rectangular in cross-section and were 3400 mm long, 160 mm deep with a clear span (distance between supports) of 3200 mm. Two of the slabs were control specimens and two of the slabs were without cutouts. Slabs were strengthened by bonding high-strength, light-weight, non-corrosive fibre reinforced polymer (FRP) composites to the tension face of the slab. When load was applied to the slabs it was observed that the three control slabs were loaded until the deflection increased without a substantial increase of load. At this stage the tension steel reinforcement had yielded and cracking had stabilised. The test was stopped when the mid-span deflection was at most 100 mm due to the maximum stroke of the actuator being reached. Slabs strengthened with FRP failed after debonding of FRP from the concrete surface after this the behaviour of the slabs approximately resorted to that of plain unstrengthened slabs and the application of load stopped when the mid-span deflection reached at most 100 mm. The maximum average load of control Slabs was 50.6 kN, 45.6 KN and 49.3 KN and the FRP-strengthened Slabs debonded at an average load of 76.4, 70.5 and 80.8 kN, so they concluded that all FRP-strengthened slabs failed by IC debonding, and the extent of debonding and the ability of the slab to sustain load post-initiation of debonding was dependent on the position of the load.

2.2.3 Retrofitting of Concrete beams by prestressing FRP

Mukherjee & Rai (2009) studied the performance of reinforced concrete beams externally prestressed with carbon fiber reinforced polymers (CFRP) laminates which included the effect of variation in prestressing force on CFRP laminates bonded to the RC beam is investigated in terms of the flexural strength, deflections, cracking behavior and failure modes. All the reinforced concrete (RC) beam specimens were loaded under four point bend test setup. The beams have been loaded with equal force on the two load points until the beams deformed did not take any further load. The loading was discontinued when the load deflection curve was flat and no increase load was observed due to the increase in deflection. It is noticed that the flexure performance of the rehabilitated beams were far superior to that of the fresh RC beams. The beams had higher failure loads and lower deflections. They remained in the elastic zone for a much higher applied load. The recovery from the deformation increased with the increase in the prestressing force. As a result, the area under the load-deflection curve was much higher for the highly prestressed beams. However, the ultimate load and the maximum deflection did not go up significantly with higher levels of prestress. They concluded that a rehabilitation one must decide the amount of CFRP based on the requirement of the ultimate capacities. By prestressing one would be able to achieve a linear load-deflection curve for higher levels of loading. Thus, the operating levels of the beam can be extended by prestressing. The results indicate that rehabilitation of significantly cracked beams by bonding CFRP laminates is structurally efficient.

Bang-yun et al (2008) used pre-stressed carbon fiber plates to strengthen reinforced concrete beams. In their study they casted five beams of dimensions 150 mm × 250 mm × 3300 mm. One beam was made controlled and other four beams were strengthened with Carbon fibres Plates. Second beam was simply strengthened by reinforcing CFP, Third beam was strengthened by reinforcing CFP and then pressed it .Fourth beam was strengthened by Prestressed CFP plates. Fifth beam was reinforced with PCFP with an initial load. Load was applied with the help of hydraulic jack. After applying load it was found that beam which was directly reinforced by CFP had higher cracking load then control beam. The beam reinforced and prestressed with CFP has same yielding load as of control beam but ultimate and cracking load was higher then control beam. One more beam which was pressed by CFP and prestressed , had increased the cracking load capacity increased by 47% and the yield load increases by 22% and the ultimate load increases by 48%, showing that the pre-stress and CFP have great effect on the cracking and ultimate loads, but has little effect on the yield load. It was observed that every reinforced method increases the ultimate bearing capacity, indicating that CFP plays a good role in reinforcing concrete. Compared with the non-reinforced beam, the beam reinforced by CFP had a much higher ultimate bearing capacity. The ultimate bearing capacity of pre-stressed beams was found to be the same as that of control beams, showing that pre-stress has nothing to do with the ultimate bearing capacity. Compared with the beam pressed only once, the yield load and ultimate load of the beam with an initial load is a little lower because the beam pressed only once has the CFP and rebar working together and the rebar of the twice-pressed beam has some strain before being reinforced. The CFP plays a better role in beams pressed only once and the beam pressed twice has a lower carrying capacity for strain of CFP lagging behind the strain of the rebar. The crack appears later and the cracking load is higher in the concrete beam reinforced with CFP than the non-reinforced beam since the CFP participates in bearing load and restricts the concrete beam. **Table 2.1** shows the carrying capacities of beams strengthened with different conditions

Table 2.1 Carrying Capacities of tested Beams .(*Bang-yun et al 2008)*

S.No	Type of Beam	Cracking Load (KN)	Yield Load (KN)	Ultimate Load(KN)
1	Control beam (L-1)	7.5	30	33
2	Beam reinforced by CFP only (L-2	8	33	41
3	Beam reinforced by CFP and pressed (L-3	8	32	41
4	Beam pressed by PCFP and (L-4	11	39	49
5	Beam reinforced by PCFP with initial load(L-5)	8.5	37	47

It was concluded for the study that after the CFP was stuck to the surface of the beams, the loading capacity of concrete beams clearly increased and cracking load, the yield load and the ultimate load also increased to a certain extent. This increase is especially evident after they are pre-stressed and the rigidity of the concrete beam increased to some extent and the deflection decreased. He concluded that Reinforcement with CFP can clearly reduce the deflection of the beam, with or without the pre-stressed load as in graph. CFP with pre-stress can offer more assistance to the deflection of beams than the one without pre-stress, which means that CFP without pre-stress has less effect on deflection before the rebar yields than after the yielding of the rebar. While the PCFP can have an obvious effect on the deflection of the beam before the rebar yield and the CFP has smaller effect on the deflection of the beam with the secondary loads than the one without initial loads as shown in **fig 2.10.**

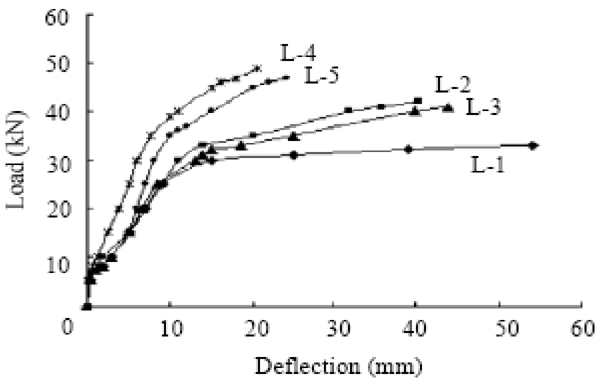


Fig 2.10 Load and deflection curve of tested beams.*(Bang .et.al,2008)*

R G wight et al (2001) investigated the effect when Prestressed FRP sheets are applied to poststrengthened RC beams for strengthening. In their study they investigated the behavior of reinforced concrete beams strengthened with prestressed FRP sheets, and compared the effectiveness of FRP prestressed sheets with nonprestressed sheets. In their study they casted four beams of size 5000×300×575 mm. One beam was made control beam ,one beam was strengthened with CFRP sheet and two beams were strengthened with prestressed CFRP sheet. CFRP sheets were applied on the tension face of the beam. All beams were strengthened with five layers of CFRP sheets. Mechanical prestressing and anchorage system was used for prestressing the beam as shown in **fig 2.11.** The mechanical anchorage system consisted of steel roller anchors bonded to the sheets and steel anchor assemblies fixed to the beam. The roller anchors that gripped the sheet consisted of two stainless-steel rollers bonded to each end of the sheet To prestress the sheets, the roller at one end of the FRP sheet was fixed to the beam (dead end) and the roller at the other end was movable (jacking end).

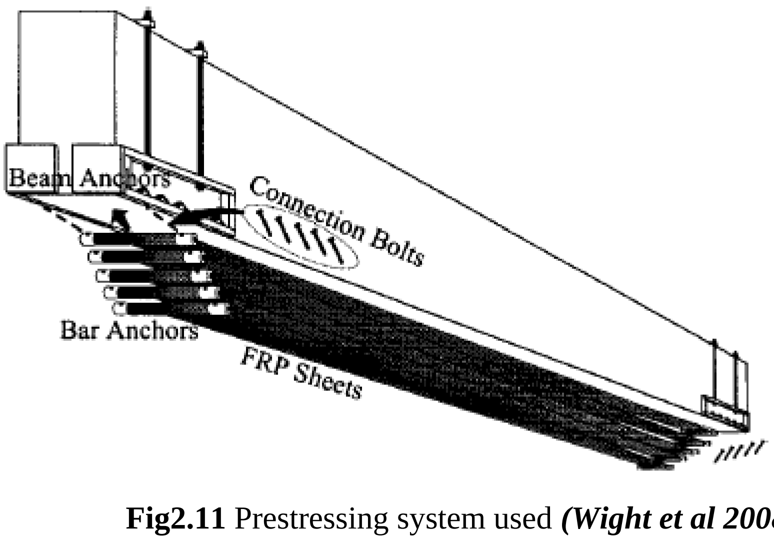


Fig2.11 Prestressing system used ***(Wight et al 2008)***

During prestressing process CFRP sheets tended to separate itself from the beam due to imperfections in the tension face of the beam. To make this contact, force was applied perpendicularly to CFRP sheets. Epoxy was used for making bond between CFRP sheet and beam surface, after applying epoxy it was cured for two weeks before the beams were tested to failure. Load was applied to the beam using a 500 KN capacity actuator. After applying loads it was observed that stiffness of the beams strengthened with non prestressed and prestressed CFRP sheets were the same as the control beam. Strengthening the beam with CFRP sheet slightly increased the first crack load, but when the sheets were prestressed, the cracking load increased significantly. Cracking of prestressed beams was delayed to applied loads by 45 to 55 KNm above the cracking load of the control beam which represented an increase in cracking load over the control beam by 150%. CFRP sheets contributed to the load-bearing capacity of the beams and the stresses carried by steel reinforcement were redistributed to the CFRP. This led to increased magnitudes of moment when the steel yielded. The beam strengthened with nonprestressed sheets, had yielding of the steel occurred at a load 20% higher than that of the control beam. When prestressed CFRP sheets were used for beams, the internal steel reinforcement was relieved of tensile stresses and placed slightly into compression. Because of the prestress, a greater portion of the tensile stresses were transferred from the steel reinforcement to the CFRP. When prestressed sheets were bonded to the concrete beams, yielding occurred at loads that were 35 to 40% higher than that of the control beams. Increase of 35 % in ultimate load was observed in when beam strengthened with CFRP and 45% was observed when strengthened with prestressed sheets. Displacement and curvature at any load were significantly less with the addition of the nonprestressed and prestressed CFRP sheets. Unstrengthened beams have wide cracks then strengthened beams. Displacements and curvatures were further reduced when the sheets were applied with a prestress because of the initial camber in the system and relatively larger tensile force was carried by the sheets which reduces crack widths. It was concluded from the study that Prestressed FRP sheets can significantly improve the serviceability of a reinforced concrete structure and reduces crack widths and delays the onset of cracking, can dramatically decrease the strains in the reinforcing steel and delay its yielding .Because cracking is reduced, the beams with prestressed sheets have smaller deflections and curvatures at failure. Although FRP sheets can significantly increase the ultimate strength of concrete beams, prestressed sheets are slightly more effective at strengthening than unstressed sheets, because prestressing can prevent premature failures.

Mohammad Reza Aram et al (2008) investigated the effect of Gradually Anchored Prestressed CFRP Strips Bonded on Prestressed Concrete Beams. To prevent premature failure of prestressed CFRP strips due to high shear stresses at the ends, the strips have to be anchored with special devices. ***Reza et*** al in their research induced the prestressing force was induced by using a gradient method. In this method, the prestressing force is reduced gr adually toward both ends of the strip in steps via heating which cures the adhesive. Prestressing system is as shown in **fig 2.12** .

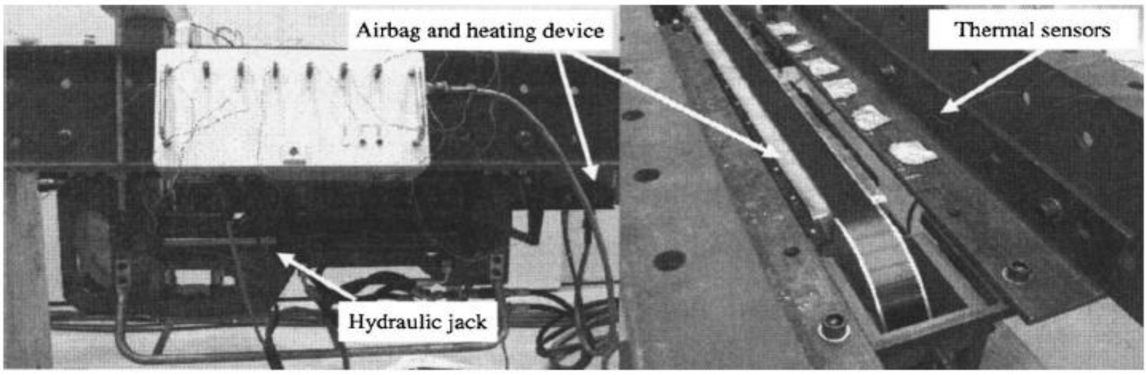


Fig 2.12 Prestressing and heating device at end of CFRP strip.*(Mohammaad et al,2008)*

In comparison to other FRP prestressing techniques, the stressing procedure is the same, with the addition of the heating work step which has the advantage of no corrosion of anchorage devices and a better quality of adhesive. In their study they made four specimens of 2400×150×250 mm. One specimen was taken as control beam ,one was strengthened with CFRP strips and two specimens were prestressed with CFRP strips. Beams were Prestrssed with a force of 60 KN and 30 KN .Beams were loaded with a testing machine with two 150 KN hydraulic actuators. Following results were observed for the four beams :

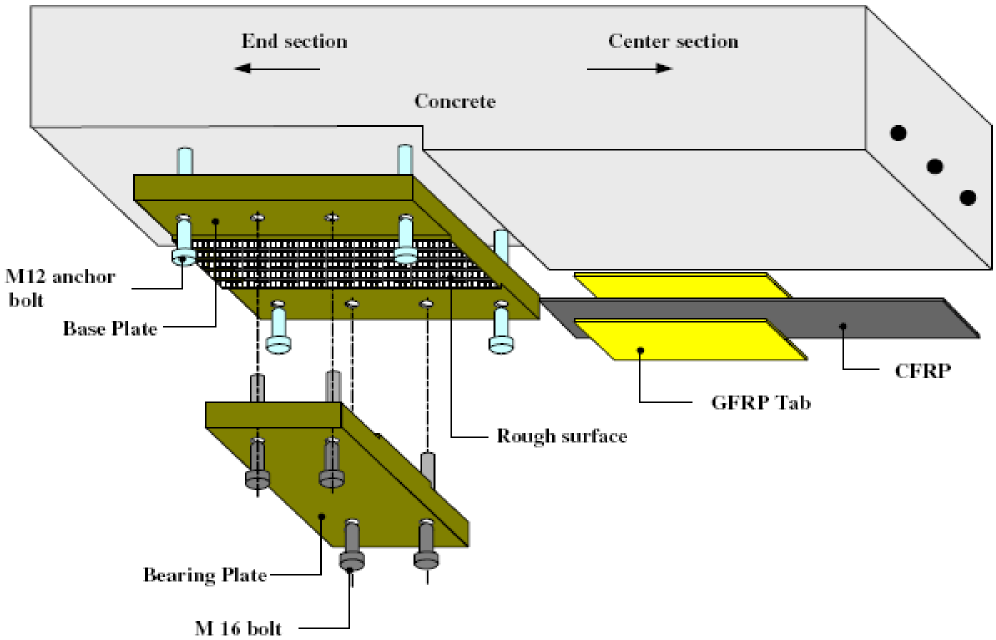
Table 2.2 Comparison between Failure loads of Strengthened and Unstrengthened beam

S.No	Beam	Cracking Load(KN)	Steel yielding (KN)	Failure Load (KN)	Cable Yielding (KN)
1	Beam 1 Control beam	29.6	82.2	94	92.5
2	Beam 2 Strengthened with CFRP strip	30.2	92.5	110.4	108.4

3	Beam 3 Strengthened with Prestressed force of 60 KN	35.3	112.2	100	127.3
4	Beam 4 Strengthened with Prestressed force of 30 KN	32.1	102.2	110.2	117.6

From **Table 2.2** it is observed that all beams except Beam 3 the reinforcement yielded at failure, and only Beam1 and Beam 2 had the cable yield at failure. It was observed that beam 3 and beam 4, in spite of the compression effect of an externally prestressing force on the concrete cross section, the load carrying capacity increased is less than in the unstressed case due to premature debonding of strips. Failure of Beam 3 and 4 was due to sudden failure due to delamination of CFRP strips at strip ends .Crack widths were decreased in beams having prestressed CFRP strips. It was found that critical values for bond shear stress for unstressed CFRP strip was 3.1 MPa . For prestressed strips anchored using the gradient method, the value of bond shear stress depends on anchorage length and prestress level, an additional shear stress in the anchorage zone from loading. The gradient force of a strip in the anchorage zone produces a theoretical initial mean shear stress of 2.4 MPa for beam 3 and 1.2 MPa for Beam 4 between the CFRP and concrete. So **Mohammad Reza et al** concluded that the gradient anchorage method was not effective because the gradient anchorage was in the region of shear stresses from loading. The shear stress from the gradient anchorage accumulated with shear stresses from loading. The short beams resulted in high shear stresses between the CFRP strips and concrete in the shear span. This method would be more effective for large span beams.

Yang et al (2009) conducted the tests to find Flexure behaviour of reinforced concrete beams strengthened with prestressed carbon composites. In his study a total of 13 beams were tested in flexure and flexure performance of RCC members strengthened by CFRP plates, using FRP bonding and prestressing methods were studied. In his experiments, bonding method, the anchorage system, the amount of prestressing, and the span length was taken as experimental variables. For this study 13 beams of three different span lengths 240, 450 ,600 cm × 200 mm (b)×300(h) mm were used , in which one was control beam, two were strengthened with non-prestressed FRP-bonded sheets, four with prestressed FRP-unbonded sheets, four with prestressed FRP-bonded sheets, and two with prestressed FRP-unbonded sheets with different span lengths. All the beams were subjected to three-point and four-point bending tests under deflection control. All the beams were made with Ready Mix concrete of concrete strength 18 MPa. Composite material used in these experiments was consisted of a three-layer component with a bi-directional CFRP sandwiched between two unidirectional CFRP plates. Method of anchoring and prestressing system is as shown in **Fig2.13.** Reinforced concrete beams were provided with prestressed CFRP plates anchored to the tension face in order to obtain the required strengthening capacity. After testing it was observed that the load capacity of the beam strengthened with one laminate (5 cm of width) was 40–60% higher than that of the control beam, and the beam strengthened with two laminates in the width direction (10 cm in width) was 100% or greater than that of the control beam but the ultimate load of the beams strengthened with bonded and non bonded prestressed CFRP sheets remained almost constant. Cracking and yield loads of the bonded prestressed CFRP plated beams were greater than those of the unbonded prestressed CFRP plated-beams, as a result of the increased stiffness due to the composite bonding. As the prestressing force induced to the beams strengthened with unbonded CFRP plates increased, the load capacity of the beams strengthened with unbonded prestressed CFRP plates also increased since the FRP was not bonded, failure was induced by rupture of the



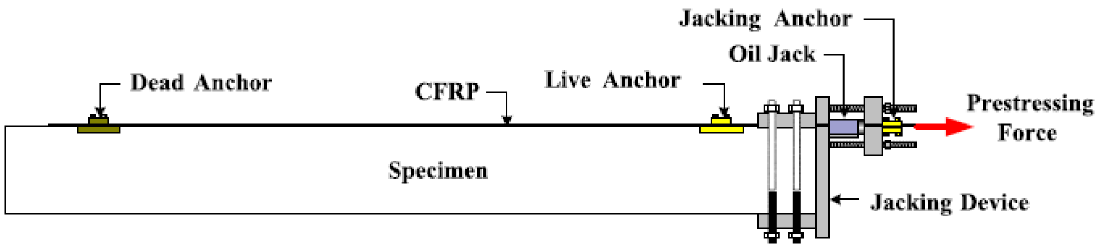


Fig 2.13 Anchorage and Prestressing system.Yang et al 2009

CFRP plates. The ultimate load of the prestressed beams was about 130–150% greater than that of the control beam, but those of the prestressed beams were similar regardless of the amount of prestressing. Beams strengthened with bonded prestressed CFRP plates increased the cracking load by 35% , 30% respectively compared to the control beam and the unbonded non-prestressed beam. Yield loads of beams with the anchorage system and bonded CFRP plates were 35–150% higher than that of the control beam.

It was concluded that Reinforced concrete beams strengthened with prestressed CFRP plates, whether bonded or not, the ultimate load of the beams strengthened with CFRP was reasonably constant. The observed failure mode of the prestressed CFRP plated-beams was not debonding, but it was due to FRP rupture. For Reinforced concrete members strengthened with bonded prestressed CFRP plates, two stages of FRP debonding occurred. After the debonding of the CFRP plates in the bonded system, the behaviour of the bonded CFRP-plated beams changed to that of unbonded CFRP-plated beams due to the effect of the anchorage systemand failure was due to FRP rupture in Reinforced concrete members strengthened with bonded prestressed CFRP plates.

ZOU (2003) compared the beams prestressed with CFRP tendons and beams prestressed with steel tendons. In his study he investigated the behaviour of both beams for long term deflection and cracking. Six beams with rectangular cross-sections 300 mm × 150 mm× 6,000 mm were fabricated and the beams contained only two CFRP tendons and no stirrups nor other longitudinal reinforcement. A particular type of CFRP tendon, tensile strength of 2,250 MPa and elastic modulus of 147,000 MPa was used in four of the beams. Seven-wires steel strand tendon, with smooth surface of 8 mm in diameter having tensile strength of 1,860 MPa and elastic modulus of 200,000 MPa was used in the other two beams for comparison purpose. Beams were prestressed with the force of 80KN and 120 KN. Material, prestresssing force, and loading conditions for both the beams were kept same. Ready mixed concrete of 40 and 80 Mpa concrete strength was used for fabricating beams. Load was applied on the all the beams and deflection was checked at 56 days after loading and after 259 days of loading .It was observed that beams casted with CFRP tendons deflected more in initial stage at the beams prestressed with steel tendons. This behaviour indicates that the loss of prestress in Beam prestressed with CFRP tendon was less than that of beam Prestresed with steel tendon. After 56 days when further loads were applied, beam with CFRP tendon showed a lower long-term deflection than that of beam with steel tendon, and the difference in deflection increased with time. It was concluded that, the instantaneous deflection of beam with CFRP tendon after cracking is more than that of beam with steel tendon and this indicates that the sectional stiffness of Beams with CFRP tendon was slightly lower than beam with steel tendon due to lower elastic modulus of CFRP. When beams prestressed with CFRP tendons having similar prestressing force but different concrete strength was compared ,the beam having higher concrete strength tend to have lower deflections for longer time periods. With the increase in concrete strength number of cracks and crack width was also reduced. So it was concluded that that the long-term deflection of beams prestressed by CFRP tendons was comparable to that of identical beams prestressed by steel tendons and beams with the higher concrete strength have lower curvature and lower deflection.

Kim et al (2008) studied the effect of RC beams when prestressed CFRP sheets were applied to the beam .In his study while prestressing CFRP sheets different methods of anchorage were used. Instead of metallic anchors non metallic anchors were used in the study.To conduct the study total of nine beams of size 150×200×1800 mm were fabricated .One beam was made as a control with permanent steel anchors J1. Three beams (J2 J3 J4) were nonanchored and were only bonded with U-wraps, out of these three beams one beam (J4) was having nonprestressed longitudinal CFRP sheets to provide a comparison to the beams with prestressed CFRP sheets. Four beams (J5 J6 J7 J8) were prestressed by using mechanically anchored U wraps. One beam was anchored with CFRP U-wraps (J9). Different kinds of anchorage systems applied on beams are shown in **Fig2.14.** After testing it was found beam strengthened with nonprestressed CFRP sheets J-4 indicated very similar behavior compared to the beams with prestressed CFRP sheets. The ultimate load of J-4 reached 81% of that of the control beam, which was a similar level to that of J-3. No remarkable advantage was noted between the prestressed and nonprestressed CFRP applications.

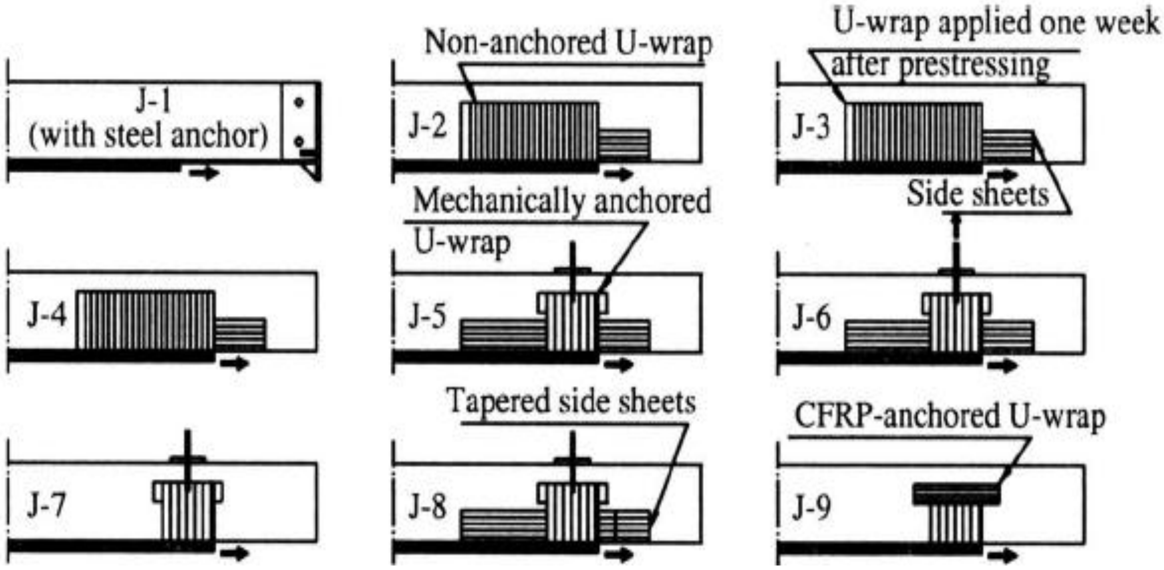


Fig 2.14 Different types of anchorages (Kim et al,2008).

U-wraps were designed and prepared to be mechanically anchored. U wraps were prepared from single layer of CFRP sheet of dimension 750 mm ×150 mm ×0.33 mm thick and this sheet was precut and bonded on the fabricated lateral anchor plate that had a high level of surface roughness to improve the bond. Immediately after the longitudinal prestressing was applied, the fully cured U-wraps were mounted onto the support plates. On two beams U wraps were prestressed .After applying non metallic anchorages prestressing force was applied and mid span deflection for every beam and with different anchorage system was noted and are shown in **Fig 2.15.** Beam strengthened with nonprestressed CFRP sheets indicated very similar behavior compared to the beams with prestressed CFRP sheets. The ultimate load of beam without prestressed fibres reached 81% of that of the control beam, which was a similar level to that of Beam with prestressed sheets.. No remarkable advantage was noted between the prestressed and nonprestressed CFRP. Beam strengthened with nonprestressed CFRP sheets. Additional mechanical anchors for U-wraps dramatically increased the ultimate loads by 25% compared to the strengthened beams with nonanchored U-wraps. Beam was anchored with CFRP U-wraps failed at lowest load as compared with others.

It was observed that the beams with unanchored U-wraps had the average loss of the prestress in the CFRP sheets after transfer was 0.44% at midspan. Losses in the beams with mechanically anchored U-wraps exhibited an average loss of 0.56%. loss in the beam anchored with prestressed U-wraps was significantly lower than other beams, which was most likely due to the active confining effect by the prestressed U-wraps. It was concluded that non metallic anchored has less prestress losses then metallic anchors .

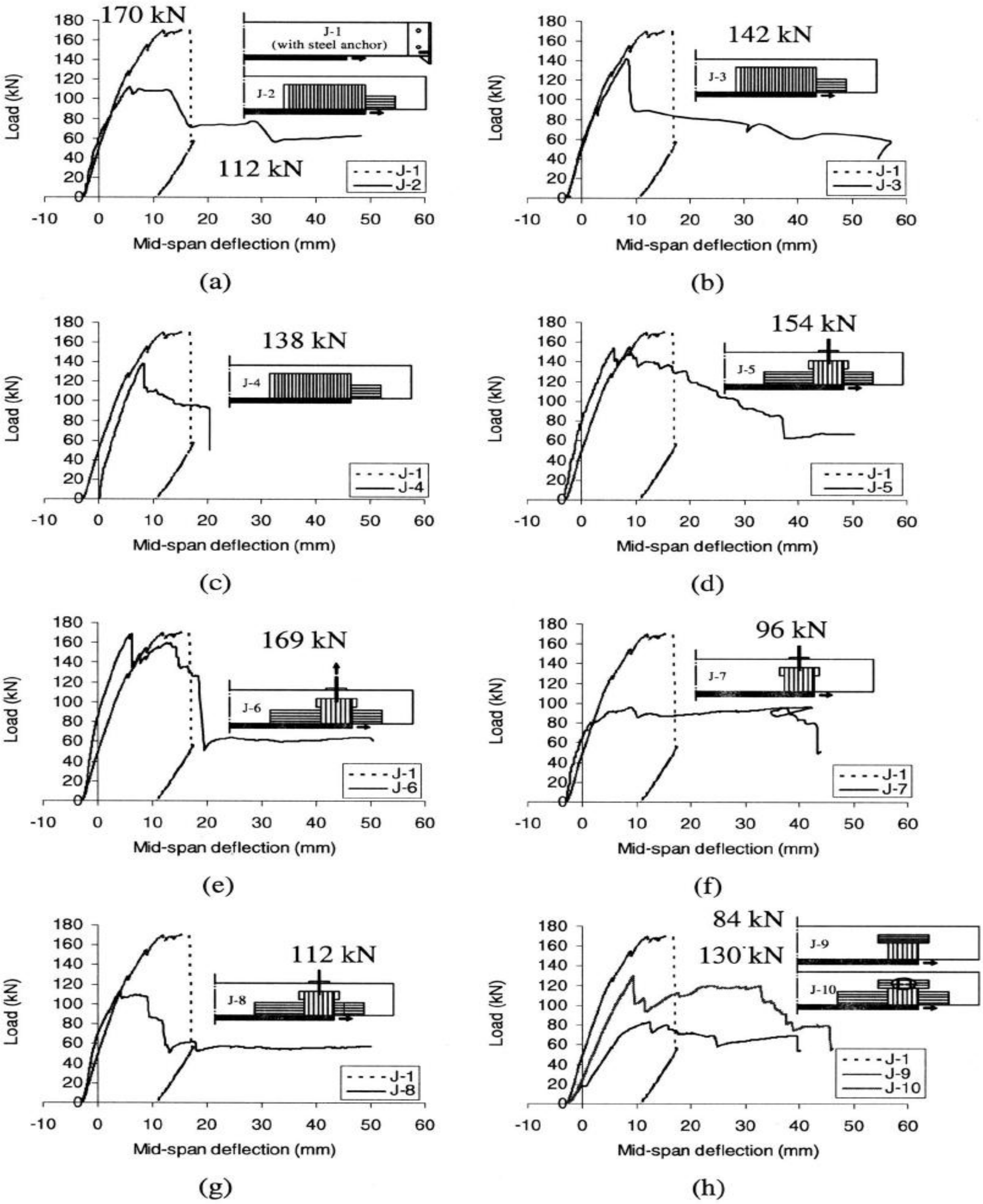


Fig 2.15 Load vs Displacements of beams with different types of anchors (Kim.et al ,2008)

Yail J Kim et al (2008) found the flexure behaviour of slabs with prestressed and non-prestressed CFRP sheets. In this research flexural behaviors of the tested slabs, including the load-deflection response, strain distribution, crack propagation, and crack mouth opening displacement were found for Four large-scale flat plate slabs 3,000 mm×3,000 mm ×90 mm having span length of 2700mm.Slabs were subjected to a monotonic patch load at centre of span with the help of plate.One slab was kept control slab and other three were strengthened with CFRP sheets out of which one was strengthened with non prestressed CFRP sheet and two were strengthened with prestressed CFRP sheets.After applying load an increase in the flexural load-carrying capacities of up to 25 and 72% was achieved for the slab strengthened with nonprestressed and prestressed CFRP sheets, as compared to the unstrengthened control slab. Failure pattern of the control slab was very ductile but for strengthened slabs it was step wise failure. High local strains were observed in the steel and the CFRP sheets under the loading area, accompanying large crack openings. The prestressed CFRP sheets provided a noteable loadsharing mechanism with the steel reinforcement that resulted in higher yield loads with respect to the unstrengthened slab. Crack mouth opening displacements at the location of the CFRP strengthened region were slightly lower than those at the center span.

Kim et al (2008) strengthened the RC beams by Prestressed CFRP sheets anchored with Non metallic anchor system. Since metallic anchor system causes aesthetic and durability problems when the strengthened structure is in service so to eliminate these problems he used Non metallic anchors like nonanchored U-wraps, mechanically anchored U-wraps, and CFRP sheet-anchored U-wraps. For this experiment ***Kim et al*** fabricated ten reinforced concrete beams of size 150 mm ×200 mm ×1,800 mm long , nine of which were strengthened with prestressed CFRP sheets and one was kept control beam. Three beams were with nonanchored U wraps, three were anchored with mechanically anchored U wraps and remaining were anchored with CFRP anchored U wraps. After applying loads it was observed that the load carrying capacity of beams with nonanchored U wraps was 80% higher then control beam. Beams with non prestressed sheets having non anchored U wraps also had same load carrying capacity. On the addition of mechanical anchors to the beams , ultimate loads dramatically increased. When these U wraps were prestressed then due to presence of prestress in the CFRP U-wraps contributed to the decrease in shear deformation of the beam and slipping of the longitudinal CFRP sheets at the ends of the beam, resulting in less deflection in comparison to the beam with nonprestressed U-wraps at loads above 80 kN. The prestressed U-wraps also contributed to the 10% higher ultimate load. Beams anchored with

CFRP U wraps failed at lower load then beam with mechanically anchored U wraps. Failure of the beam with permanent metallic anchors was attributed to the excessive opening of peeling-off cracks but the failure of the beams strengthened with the replaced CFRP anchors was initiated by the delamination of the side sheets. Beams with mechanically anchored U-Wraps exhibited no delamination of U-wraps due to the mechanical anchors. The confining effect by the mechanically U-wrap anchors kept cracks closed, allowing a significant transfer of force through aggregate interlock across the face of the crack. The confining effect may have also increased the load-carrying capacity of the beam due to which the opening of the cracks was delayed. Failure of the beam with permanent steel anchors was brittle but, the beams with non-metallic anchors exhibited pseudo ductile failure due to the contribution of CFRP anchors. The beams with mechanically anchored U-wraps and side sheets showed almost the same load carrying capacity as the control beam. Beams with non-metallic anchors exhibited better stress redistributions compared to the beam with steel anchors.

2.2.4 Retrofitting by SFRP

Lee et al (2009) found new technique called Sprayed Fibre Technique SFRP to increase the load carrying capacity of beams. In his research he investigated the load capacity, ductility and energy absorption aspects of reinforced concrete (RC) beams retrofitted with sprayed fiber-reinforced polymer composites (SFRP). In his research he performed series of three-point bending tests on both damaged (precracked) and undamaged RC beams to evaluate the performance of deteriorated RC beams after application of SFRP and to examine the influence of SFRP parameters on the performance of RC beams. Size of the beams were 100×100×450mm. The parameters in the experimental program were coating thickness, fiber length, fiber materials and fiber loading. This method of retrofitting by sprayed fiber-reinforced polymer composites (SFRP) consist of randomly oriented chopped fibers of controlled length in a polymer matrix. For the application of the SFRP, a spray gun with a chopper unit and epoxy containers were used.

Fig 2.16 Spray gun

SFRP was applied with the help of spray gun on RC beams .Below is the figure showing how SFRP was applied on RC beams and how beams look after applying SFRP. After applying SFRP on beams load was applied at the midpoint of the beam through the load head and load applied was uniform throughout. Since there were some parameters like coating thickness, fiber type and fiber length certain values were fixed for them and tests were performed and results were taken. The values fixed for the parameters were :

(1) coating thickness: 3.2 and 6.4 mm;

(2) fiber volume fraction: 15% and 30%;

(3) fiber type: Eglass and carbon fibers;

Fig 2.17 Method of applying SFRP

Fig 2.18 Beams after SFRP is applied

(4) fiber length: 13 and 26 mm

(5) precracked and uncracked specimens

Following were the results obtained after testing

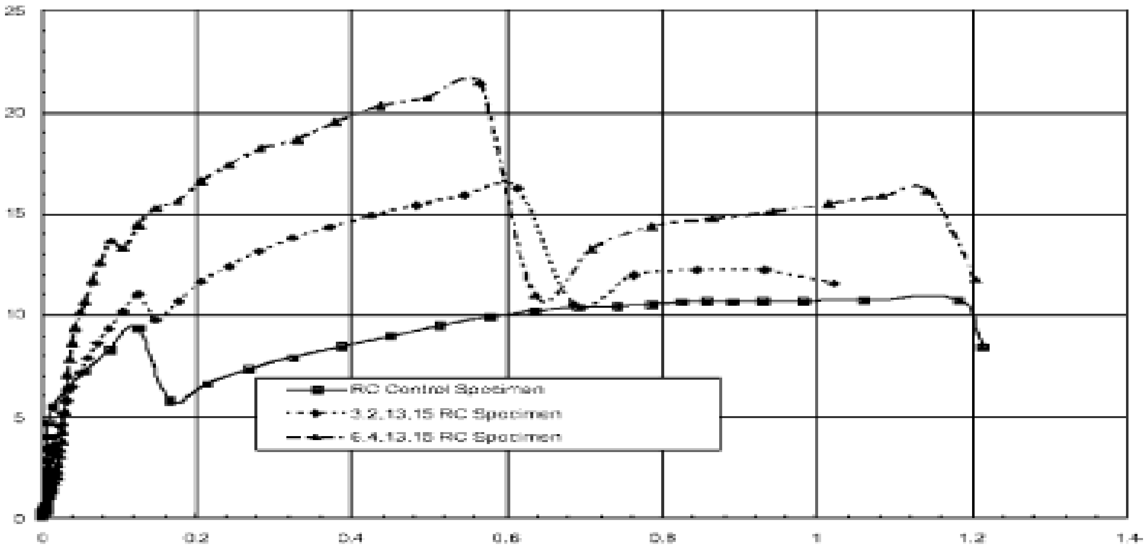


Fig 2.19 Graph showing load –displacement curve for RC specimen with different coating thickness

The typical behaviour of the RC control specimen (no SFRP coating) is also depicted for comparison in the **Fig 2.19**. In **Fig 2.19**, 17% and 51% increases in peak and ultimateload were observed with the thin coating, while the thick coating increased them by 45% and 100%, respectively. The energy absorbing capacity was enhanced by 15% for the thin coating, while a 63% increase in energy absorbing capacity were obtained for the thick coating.

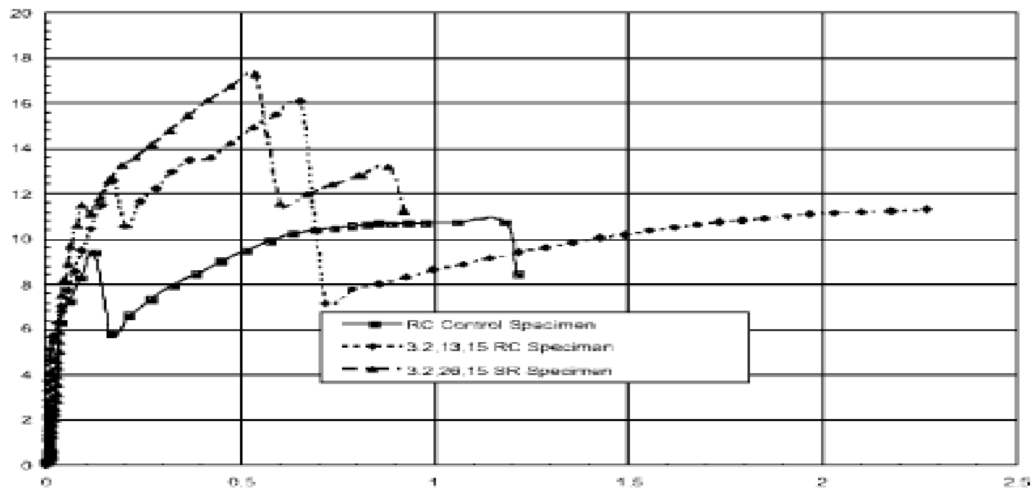


Fig 2.20 Load Displacement curves of RC Specimens with different lengths

Fig 2.20 shows that 34%, 50% and 59% increases in peak, ultimate load and energy absorption, respectively, were observed with the coating having short fibers of length 13 mm, while 22%, 61% and 30% increases in those were recorded by the coating with long fibers of length 26 mm .

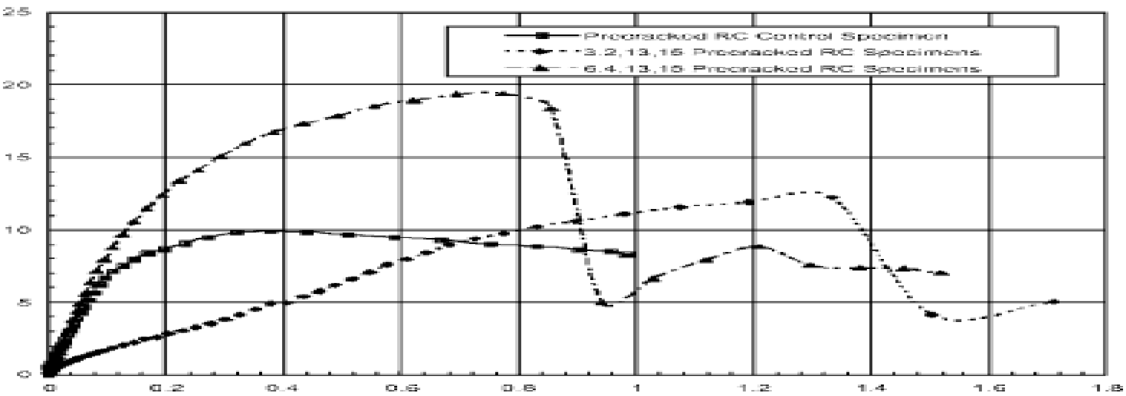


Fig 2.21 Load –Displacement curve of precracked RC specimens with different coating thickness

Fig 2.21 shows that 24% increase in peak and ultimate load was observed for the thin coating, while the thick coating increased it by 96%.

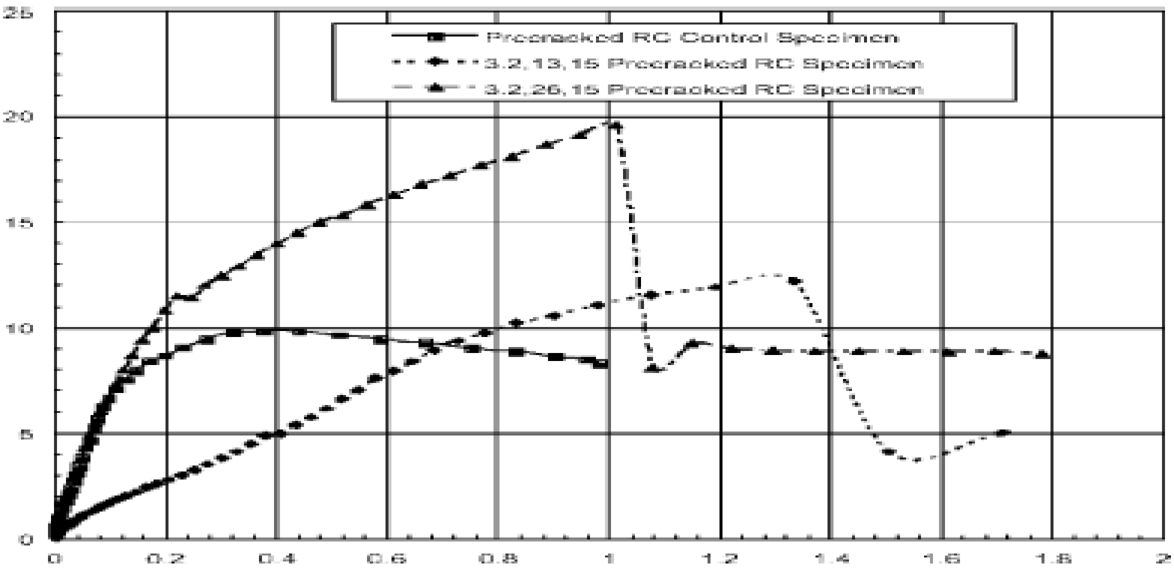


Fig 2.22 Load Displacement curve of precracked beams with different length of fibres.

From **Fig 2.22** it was concluded that 24% increase in peak load , 24%increase in ultimate load and 49% increases in energy absorption was observed with the coating having short fibers of length13 mm, while 98% increase in peak load, 98% increase in ultimate load and 153% increases in energy absorption were recorded by the coating with long fibers of length 26 mm.

So ***Lee et al*** concluded that coating thickness has a significant influence on the peak load, ductility and energy absorption capacity of RC beams. A more ductile failure with a significant increase in energy absorption capacity was observed with a thicker coating and an appropriate fiber length near 23 mm will maximize the increase in load carrying and energy absorption capacities of RC beams. Also he found that Carbon fibers lead to higher increase in load carry ability and lower increase in energy absorption for both damaged and undamaged RC beams due to their brittle characteristics compared.

CHAPTER-3

EXPERIMENTAL PROGRAMME

3.1 Introduction

Study was carried out to find the behaviour of under reinforced concrete beams retrofitted with CFRP sheets and comparison of performance of beams retrofitted by prestressed sheets and non prestressed sheets was found. In this work behavior of RCC beams strengthened with prestressed fiber sheet was studied. First the beam was failed and then it was retrofitted by prestressing the fiber sheet and non prestressed fiber sheet and difference between load carrying capacities of control beam and retrofitted beam was found. Firstly the test beam was failed and then it was retrofitted by prestressed fiber sheets .Its load carrying capacity was then calculated and then the difference between load carrying capacity of control beam and retrofitted beam was calculated.

3.2 Experimental programme

Principal aim was to find the behavior of RCC beams retrofitted with Prestressed fiber sheets and non prestressed fiber sheet. For this study to carry three real size beams of size (600 x 300 x 4100mm) were casted using M20 grade concrete. Two of the beams were retrofitted with Prestressed fiber sheets by using single and double layer and one beam was retrofitted with non prestressed sheet by using single layer.

3.3 Materials

Cement, fine aggregates, coarse aggregates, reinforcing bars are used in casting of beams. Primer is used for preparing base and Saturant is used for fixing fibers with beam, Anchor bolts are also used for fixing prestressing machine. The specifications and properties of these materials are as under:

3.3.1 Cement: Portland pozzolana cement of 53 grade from a single lot was taken 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.

Table 3.1 Properties of cement

Sr.No.	***Characteristics***	***Values obtained***	***Indian Standard(IS: 8112-1989) values***
1.	Initial Setting time	140 min	Not be less than 30 minutes
2.	Final Setting time	330 min	Not be greater then 600 minutes
3.	Fineness	4%	<10%
Compressive strength			
1.	7 days	35 N/mm^2	41N/mm^2
2.	28 days	55 N/mm^2	53 N/mm^2

3.3.2 Fine Aggregates

The sand used for the experimental works was locally procured and conformed to grading zone III. Sieve Analysis of the Fine Aggregate was carried out in the laboratory as per IS 383-1870. The sand was first sieved through 4.75mm sieve to remove any particle greater than 4.75 mm sieve and then was washed to remove the dust. The physical properties results of sand are shown in **Table 3.2.**

Table 3.2 Physical properties of fine aggregates

S.NO.	***Characteristics***	***Value***
1.	Type	Natural Sand
2.	Specific Gravity	2.65
3.	Water absorption	1.02%
4.	Moisture content	0.15%
5.	Fineness Modulus	2.22
6.	Grading Zone	III

3.3.3 Coarse Aggregates

Crushed stone aggregate (locally available) of 20mm are used through out the experimental study. The physical properties of coarse aggregate are given in **Table 3.3.**

Table 3.3 Properties of coarse aggregates

S.NO.	***Characteristics***	***Value***
1.	Type	Crushed
2.	Specific Gravity	2.61
3.	Water absorption	2.37%
4.	Moisture content	0.33%
5.	Maximum Size	20 mm

3.3.4 Water

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

3.3.5 Reinforcing Steel

HYSD steel of grade Fe-415 of 12mm and 8mm diameters were used as longitudinal steel. 12mm dia bars are used as tension reinforcement and 8mm bars are used as compression steel 8mm diameter bars are used as shear stirrups.

3.3.6 CFRP material

For retrofitting CFRP sheets were used having cross section 300 X 0.1176 mm. This sheet consist of two components materials: Matrix material or polymer which is generally low strength and low modulus component and second is the fiber which is relatively high strength and low modulus component. Under stress, fiber utilizes the plastic flow of matrix to transfer the load to the fiber which result in high strength and high modulus composite. Main function of matrix is to combine and to protect the fiber against external environment into which the composite will be placed .

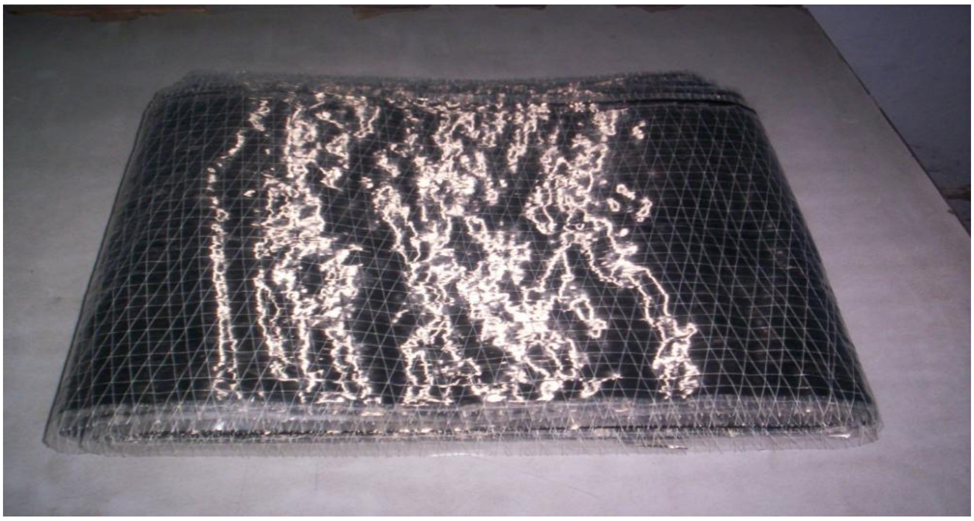


Fig 3.1 CFRP sheet used in the experiment

Table 3.4 Properties of CFRP Sheets

S.NO.	***Physical properties***	***Value***
1.	Tensile Strength	3800 N/mm^2
2.	Modulus of elasticity	240000 N/mm^2
3.	Density	1.7

3.3.7 Bonding materials

Primer –It is a 100% solid epoxy resin based primer for use on porous mineral substrate. Primer being a low viscosity material assist adhesion either by partially penetrating pores of a porous surface or by forming a chemical link between the surface and a relatively high viscosity adhesive .primers also bind and reinforce weak surface layers of certain substrates such as concrete and stone . If primer is not applied properly then it is evident that adhesion failure will occur.Low viscosity of primer enables effective penetration and sealing of porous.. Properties of Primer used are discussed below in **Table 3.5**

Table 3.5 Properties of Primer

S.NO.	***Properties***	***Value***
1.	Mixed Density	1.07 kg/litre
2.	Mixing ratio by weight	100 Base : 50 Hardener
3.	Coverage	4 to 6 m_2/kg

4	Pot Life	40 minutes at 25 degree
5	Aspect	Free flowing liquid

Saturant- It is a 100% solid, blue pigmented, epoxy resin for saturation of MBrace FIBRE SHEET to form in-situ FRP Composite. It was made by mixing Base Saturant and hardner Saturant in ratio 100:40. Epoxy was applied on beam to fix the fiber with beam .Properties of Epoxy are discussed in **Table 3.6**

Table 3.6 Properties of Saturant

S.No	Properties	Values
1	Aspect	Transluscent Blue Liquid
2	Density	1.13 ± 0.03
3	Mixing ratio,by weight (B:H)	100:40
4	Pot life	25 minutes at 25 degree
5	Tensile Strength	> then 17 Mpa
6	Compressive Strength	>40 Mpa after 1 day
7	Flexural strength	>35 Mpa

3.3.8 Concrete Mix

M20 grade concrete is considered as per standard design procedure using the properties of materials as discussed above i.e. Table 3.1 to Table 3.4.The mix proportion of material is 1:1.5:3.0 (cement: sand: aggregate) and compressive strength of concrete after 28 days is 29 N/mm^2.

Table 3.7 Properties of Concrete

S.NO.	***Physical properties***	***Value***
1.	28 days compressive strength	29.0 N/mm^2

3.4 Equipments

3.4.1 Prestressing machine-It was used to prestress the fiber sheet. Hydraulic jack was fitted in the machine with which force was applied and prestressing of fiber was done. On the sides of this machine dial gauges were fixed to measure the elongation in fiber after applying the force. Front part of the machine was curved and moveable.

Fig 3.2 Prestressing machine

3.4.2 Dial Gauges-Dial gauges were used to measure deflection of the beam after applying load .Least count of dial gauge was .01 mm and range was 2.5 cm and 5.0 cm.

Fig 3.3 Dial gauge

3.4.3 Hydraulic jack-Hydraulic jack was used to apply the load. Range of Hydraulic jack was 500 KN

3.4.4 Roller-Roller was used on fiber sheet to make proper bond between fiber and epoxy. Material of the roller was wood.

3.4.5 Anchor bolts-12 mm dia Anchor bolts were used to fasten the plates of prestressing machines and plate. This bolt consist of two parts, bolt and rod.

3.5 RCC BEAM DESIGN

In the present study the RCC beam is design using M20 grade and Fe415 steel. The RCC beam is design with limit state method considering it to be under-reinforced section. After the design reinforcing steel coming is 4 bars of 8mm at compression face and 4 bars of 12 mm at tension face. The stirrups used were of 8 mm diameter at 75 mm C/C. Cross sectional dimension of the beam are 600 x 300 mm. Longitudinal section and cross- section of beam is shown in **Fig. 3.4**

3.6 CASTING OF COMPOSITE BEAMS

The casting of beams was done in single stage. The beams were cast in mould of size 600 x 300 x 4100 mm. First of the entire beam mould was oiled. So that the beams can be easily removed from the mould after 24 hours. Spacers of size 25mm are used to provide uniform cover to the reinforcement. When the bars have been placed in position as per design concrete mix was poured in the mould and vibrations were given with the help of needle vibrator, so that the mix gets compacted. The vibration was done until the mould was completely filled and there was no gap left. The beams were then removed from the mould after 48 hours. After demoulding the beams were cured for 28 days using jute bags.

3.7 TESTING ARRANGEMENT

All the three beams were tested under simply supported end conditions. Two points loading was adopted for testing. The testing of beam was done with the help of hydraulic operated jack connected to load cell. The load was applied to the beam with the help of load cell and value was obtained from the data acquisition system, which was attached with the load cell. Five dial gauges were placed, one dial gauge was placed at the center, two dial gauges were placed at a distance of span/4 from the end and two below point load. The value of deflection was obtained from these dial gauges. **Figure 3.4**. shows the testing arrangement.

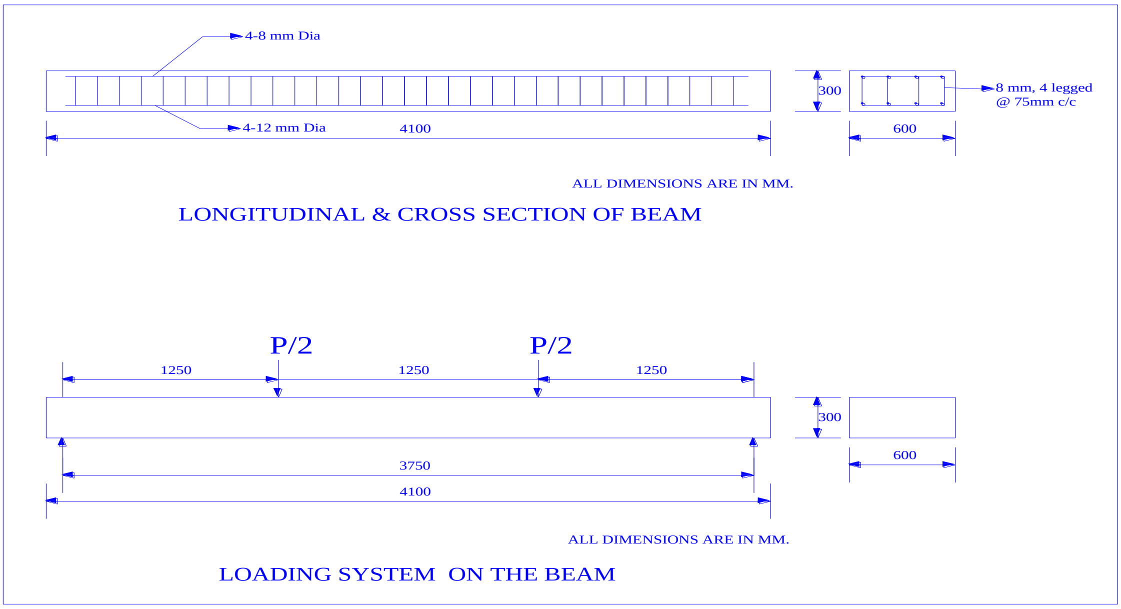


Fig 3.4 Details of beams & loading system

3.8 Test procedure

In the experimental programme the beams were tested using two point loading arrangement. The testing of the beam was done with the help of hydraulically operated jacks connected to the load cell. This load cell was used to apply the load over the surface of the beam and the value of load was read from the data acquisition system connected to the load cell. The value of deflection was measured using dial gauges. Failed beam was then retrofitted with the help of prestressing the fibers.

3.9 Control beam

First fresh RC beam specimen was loaded in a four point bend test set up as described in previously. This test was carried out on the RC beams prior to the application of any FRC. The set up ensures pure bending in the central third portion of the beam. The beams were loaded with equal force on the two load points and loads was applied till the beams deformed and stop taking further load. It may be noted that the beam sections were under-reinforced. Therefore, steel had yielded in all the specimens. The damage in the beams started with bending cracks in the central region of the beam , first crack was observed at 41 KN. As load reached 60 KN more cracks were observed. Almost all cracks were vertical near top and bottom edge, sub cracks were developed connecting to main crack. First crack developed between right point load & beam center. Second crack was just inside the right point load and near to first crack. Third & fourth crack developed was between center point of beam & right point load. Fifth crack was just outside of right point load. Sixth crack was in between third & fourth crack. No shear crack was observed. At 82 KN spalling started. Major cracking was observed at 88 KN. Beam stopped taking load at 92 KN. Final deflection at the center was 86 mm. All the three beams were tested and were make to fail and then were retrofitted with different procedures. After testing all the beams was rotated & placed so that its bottom surface will be top surface which can be easily accessed for retrofitting.

3.10 Steps for preparing surface for Retrofitting: After testing the control beam, beam was rotated so that retrofitting should be done on its bottom surface. Grinding and priming was done on all the beams and same procedure was adopted. Prestressing was done on only two beams and on one beam non prestressed sheet was used for retrofitting. Following steps were followed for retrofitting

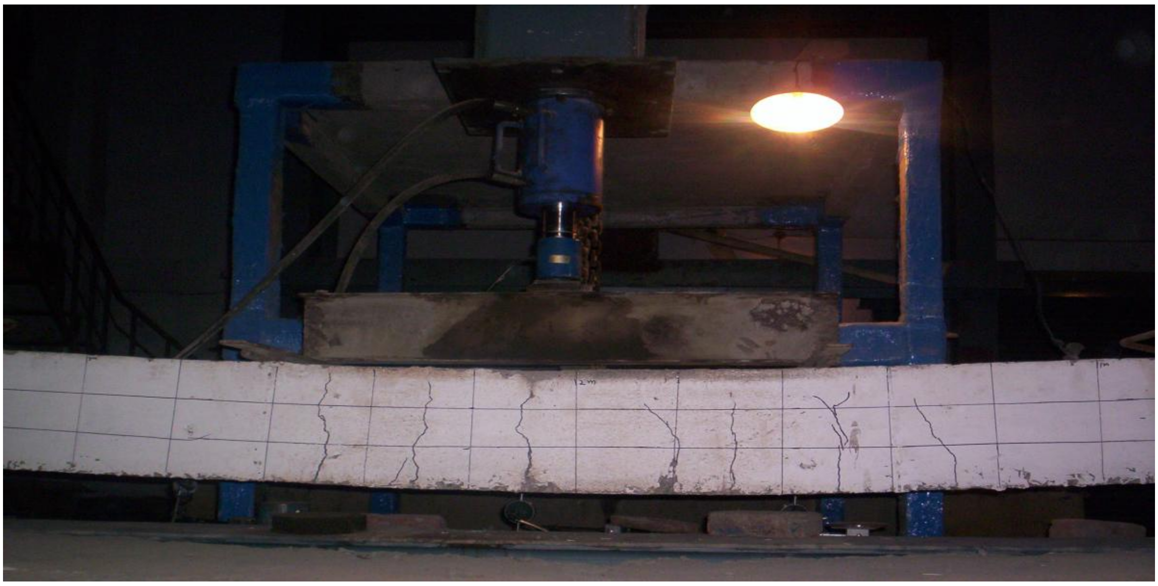


Fig 3.5 Deflected beam & cracks in the central region of the beam

-

- Grinding
- Priming
- Anchorage and Prestressing of fibers
- Testing of retrofitted beam

3.10.1 Grinding

Beam was grinded with the help of grinder having Diamond Plates. Purpose of grinding was to expose the aggregates so as to have better adhesion of the aggregates with the fiber.

3.10.2 Priming Priming was done after grinding process. Priming is done with the help of primer. Primer used was MBrace primer of BASF company. Ratio of base and hardener used in Primer was taken from the data sheets supplied by the company. In this case, it is 100Base:50Hardner. Both base and hardener was properly mixed and were applied on grinded surface with the help of aluminum strip. Before applying primer, grinded surface was cleaned from dust and other particles. After applying the primer it was cured for 24 hours.

Fig 3.6 Surface after Grinding and Priming

3.10.3 Anchorage and prestressing of fibers

This was the most important and critical part of the experiment. Fiber sheet was anchored at one end and it was pulled from the other end to apply the prestressing force. For anchoring the sheet, plate was used on one side of the beam. First holes were drilled in the beam then fiber was fixed on the beam by using adhesives. Plate was fixed on the fiber sheet and was tightened with anchor bolts as shown in **Fig 3.7.**

Fig 3.7 Fiber fixed with plate at one end

After fixing the fiber it was left for curing for one day. Only fiber of length equal to plate was fixed with adhesive rest of the fiber was not fixed. On the other end of the beam Hydraulic jack was fitted with the help of anchor bolts. On the same system, dial gauges were fitted on both the ends of machine as shown in **Fig 3.9** so as to measure the amount of movement of the jack and hence the elongation of fiber. Along with this, two more dial gauges were fixed, one with the beam to measure relative movement of the fiber and the beam. Other dial gauge was fixed to the permanent base in order to measure relative slippage of the fiber as in **fig 3.8**

Fig 3.8Dial gauges for measuring slippage

Fig 3.9 Dial gauge for measuring elongation

After this surface was cleaned and was made dust free to fix the CFRP sheet to the beam. Adhesive used was MbraceSaturant. Saturant was mixed with the ratio of 100Base:40Hardener. After mixing base and hardener a translucent blue liquid was formed known as epoxy and it was applied to the beam. CFRP sheet was then laid on top of the epoxy. The sheet was then taken on to the curved part of the machine fitted on the other end. and another plate was then fixed below the FRP sheet and they were fastened together with bolts to make proper grip of sheet so that uniform force should be applied. Fiber sheet was fixed on both curved plates to provide proper grip to Fiber sheet so that uniformly force could be applied. Immediately after fixing the sheet prestressing force was applied with jack. On the application of force, the curved part start moving along with the sheet and sheet started elongating. The elongation was measured with the help of dial gauges as mentioned in **Section 3.4.1**.Fibre sheet was elongated upto 50 mm and prestressing force at this elongation was 70 KN. **Fig 3.11** shows the sheet after applying adhesive and after applying prestressing force.

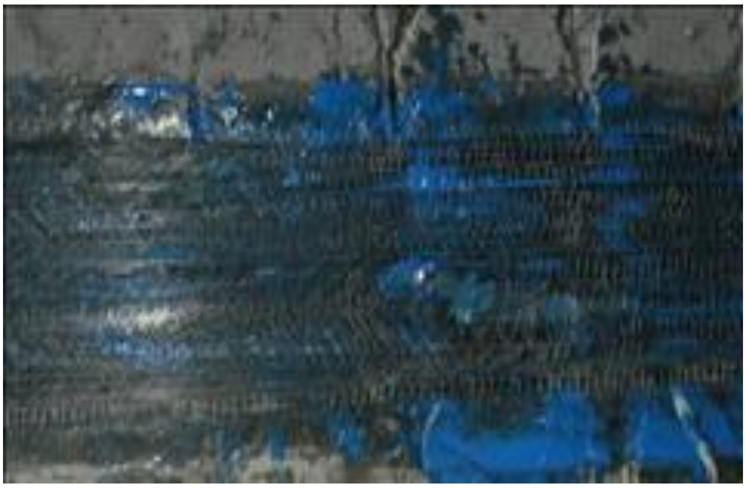


Fig 3.10 Fibre sheet after prestressing

Every fiber in the sheet was under the tension and sheet was elongated. It was prestressed upto 70 KN of force. Dial gauges were also fixed at the centre of beam to measure recovery of deflected beam due to prestressing force applied. After prestressing corners of the beams were wrapped with the fiber so that CFRP sheet should not stuck out. After prestressing the sheet roller was rolled on the Fiber sheet to make the proper bond between beam surface and CFRP sheet. It was left for curing for two days and jack and plate were removed and beam was tested under load for checking the strength.

3.10.4 Beam Retrofitted with two Layers of prestressed Fiber sheet

Procedure adopted for retrofitting the beam was same as done for the beam with single layer. All the operations were performed on this beam also only difference was that two layers of Fiber sheets were prestressed in it. Epoxy was applied on the beam and then first sheet was applied on the beam and then second was applied on the previous sheet and then it was rolled properly till epoxy starts coming out. The sheets were then taken on to the curved part of the machine fitted on the other end and another plate was then fixed below the FRP sheet and they were fastened together with bolts to make proper grip of sheet so that uniform force should be applied. Fiber sheet was fixed on both curved plates to provide proper grip to Fiber sheet so that uniformly force could be applied. Immediately after fixing the sheet prestressing force was applied with jack. On the application of force, the curved part start moving along with the sheets and sheets started elongating. The elongation was measured with the help of dial gauges as mentioned in **Section 3.4.1**. Fiber sheets were elongated upto 50 mm and prestressing force at this elongation was 70 KN. Every fibre in the sheets was under the tension and sheet was elongated. It was prestressed upto 70 KN of force. Dial gauges were also fixed at the centre of beam to measure recovery of deflected beam due to prestressing force applied. After prestressing corners of the beams were wrapped with the fiber so that CFRP sheet should not stuck out. After prestressing the sheet, roller was rolled on the fiber sheet to make the proper bond between beam surface and CFRP sheet. It was left for curing for two days, jack and plate were removed and beam was tested under load for checking the strength.

3.10.5 Beams with Non prestressed sheet

Control beam was tested and then it was rotated and then retrofitted with a fiber sheet without prestressing it. Grinding and Priming was done on the beam as explained in section **3.10.1 & 3.10.2** and epoxy was applied on the beam .After applying epoxy sheet was fixed on beam and was properly rolled until the epoxy starts coming out of the fiber. Then it was left for curing for 72 hours and then was tested.

3.11 Testing of beams retrofitted with prestressed sheets

After curing the applied epoxy for 72 hours machine was taken off and beam was rotated keeping retrofitted side below and was subjected to testing under hydraulic jack. Dial gauges were fitted below the beam to find the deflections due to applied loads. Dial gauge was fitted at centre, at L/4 and at point load. After taking readings of deflection these were compared with the readings of control beam. Beam retrofitted with non prestressed sheet was directly subjected to testing after curing of epoxy for 72 hours.

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Beam Retrofitted with one FRP Sheet

Grinding and Priming was done on the beam after testing it as control beam and epoxy was applied on the beam. After applying epoxy, sheet was fixed on beam and was properly rolled until the epoxy starts coming out of the fiber and it was left for curing for 72 hours and then beam was rotated and was subjected to two point load testing. When load was applied to the beam it was observed that existing cracks start widening up at load of 30 KN although and new cracks start developing. Cracks start developing at 35 KN load. Spalling started at load of 75KN. Cracking sound was heard at 80 KN and at 95 KN loud cracking sound was heard resulting in the failure of beam. Fracture of fiber occurred at 102 KN and it was observed that retrofitted beam take 10KN more load then control beam. The ultimate load carrying capacity of beam was increased by 10.8% as compared to control beam. Failure in the beam occurred due to fracture in fiber as shown in **Fig 4.1**. Fiber cracked in zone of pure bending and finer cracked completely in transverse direction as shown in **Fig 4.1**.

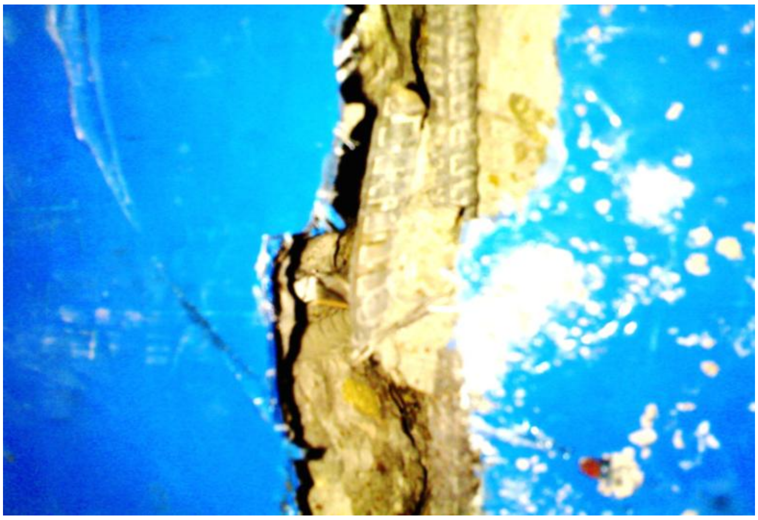


Fig 4.1 Fiber fracture in beam Retrofitted with Non Prestressed fiber sheet

Bending cracks were developed during the failure of beam, no shear cracks were developed in the beam. It indicates that the failure is due to pure bending .Load vs deflection curve at center and at L/4 are shown in **Fig 4.2 and Fig 4.3**. It is found from the **Fig 4.2 and Fig 4.3** that there is slightly decrease in deflection in beam retrofitted with one sheet as compared with control beam indicating increase in stiffness of cracked beam by applying FRP sheet.

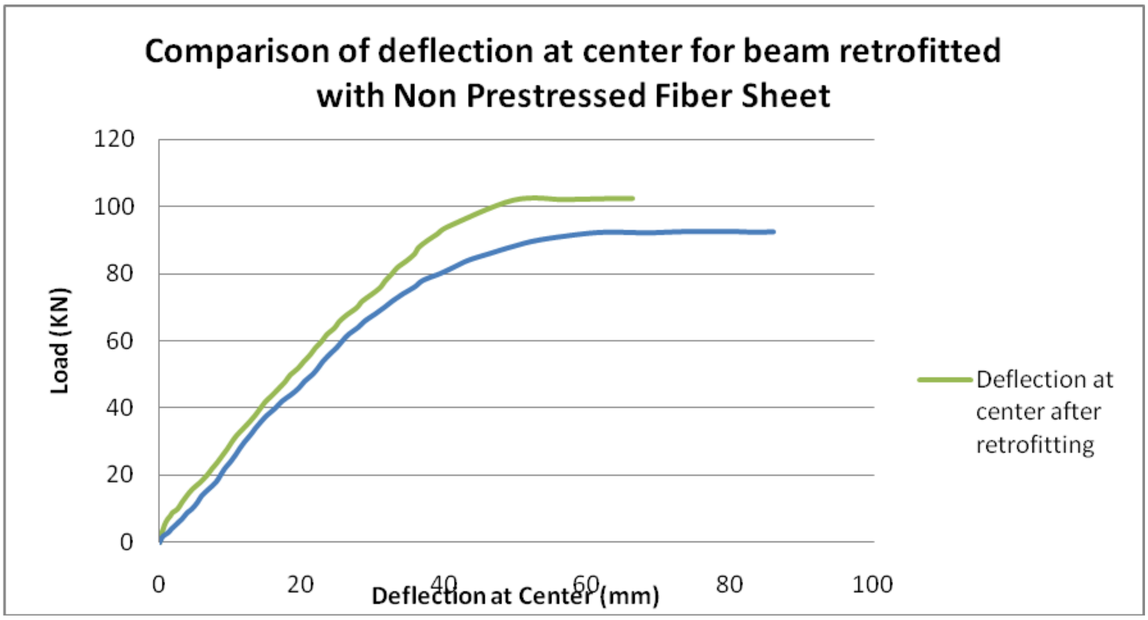


Fig 4.2 Graph showing comparison of deflection at center for beam retrofitted with non prestressed sheet

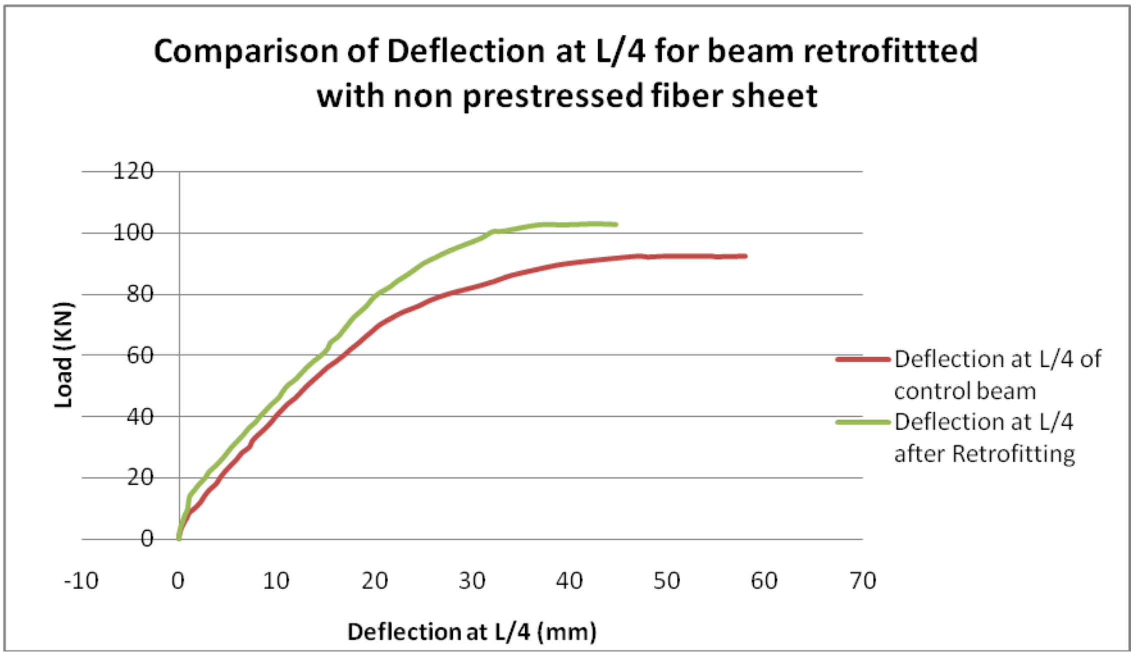


Fig 4.3 Comparison of deflection at L/4 for beam retrofitted with non prestressed sheet

4.2 Beam retrofitted with Single layer of prestressed sheet

Grinding and Priming was done on the beam and sheet was anchored from the one end and epoxy was applied on the beam. After applying epoxy sheet was fixed on beam and was properly rolled until the epoxy starts coming out of the fiber and was prestressed. After prestressing the fiber it was allowed to cured for 72 hours and then prestressing machine and anchorage plate was put off and beam was rotated and subjected to testing. When load was applied it was observed that existing cracks starts widening up at 50 KN and few new cracks developed .Width of new cracks was small as compared to existing cracks. New cracks formed were of small width and were lesser in number as compared to beam retrofitted with single FRP sheet. Cracking sound was observed at 90 KN and debonding of sheet starting at this load. Sheet was debonded at load of 103 KN and beam failed and sudden deflection was observed. Load carrying capacity was increased by 12%. Deflection of this beam was less as compared with beam retrofitted with without prestressing fiber. Load vs deflection curve at center and at L/4 is shown in **Fig 4.5 & 4.6**. Graph shows increase in load carrying capacity and decrease in deflection. Failure occurred due to debonding of sheet as shown in **Fig 4.4.**

Fig 4.4 Debonding of sheet from the beam

Loud sound of crack was heard and sudden jerk in the beam was observed when debonding occurred. Sheet was debonded from two places and at this point beam stopped taking load with sudden increase in deflection. Debonding occurred below the two point loads. At failure bending cracks were observed and spalling of concrete take place as shown in **Fig 4.5**. Stiffness of the beam retrofitted with single layer prestressed sheet was more then control beam and beam retrofitted with single FRP sheet.

Fig 4.5 Bending cracks and spalling of concrete

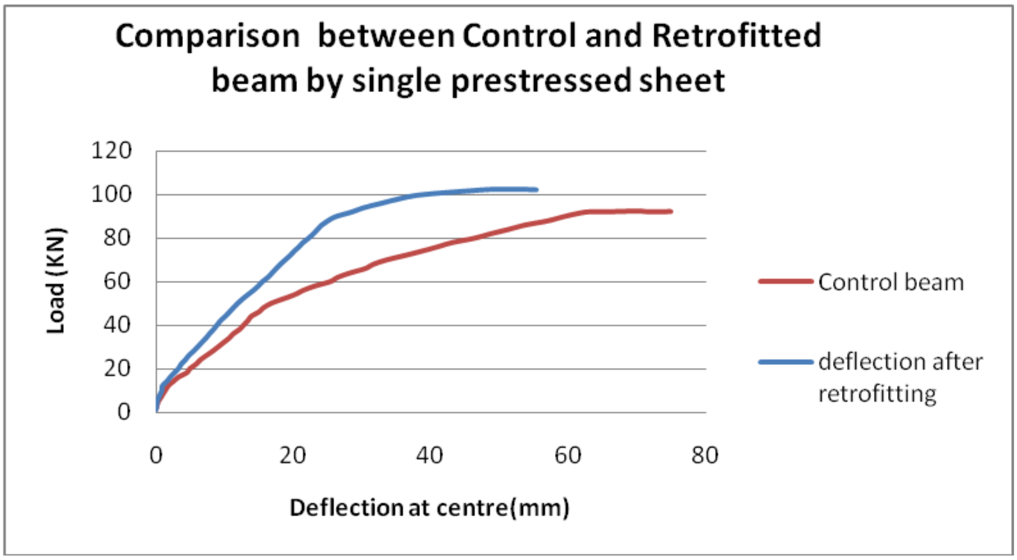


Fig 4.6 Comparison between load and deflection for retrofitted beam and Control beam at center.

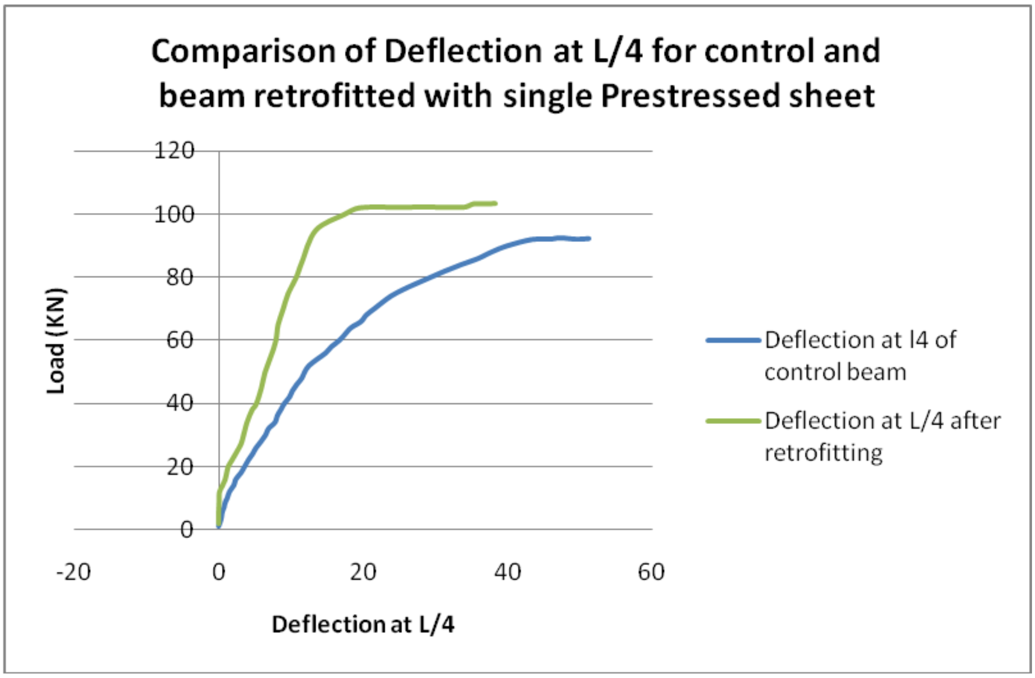


Fig 4.7 Comparison at L/4

4.3 Beam Retrofitted with Two layers of prestressed sheet

After performing Grinding and Priming operations on failed beam double layer of sheet, sheet was anchored from the one end and epoxy was applied on the beam. After applying epoxy, sheet was fixed on beam and was properly rolled until the epoxy starts coming out of the fiber and was prestressed. After prestressing the fiber it was allowed to cured for 72 hours and then prestressing machine and anchorage plate was put off and beam was rotated and subjected to testing. During testing it was observed during initial stages not much of deflection was observed resulting in straight curve in Load vs Deflection curve as shown in **Fig4.10.** Cracks start widening up at 70 KN of load and no new cracks developed in the beam**.** Fiber wrapped around the beam was also fractured. Cracking sound was heard at 100 KN and at this point concrete start spalling out. Sheet start debonding at 120 KN .Loud cracking sound was heard at 130 KN and sheet debonded at this load and beam was failed**.** Beam retrofitted with double layer of prestressing fiber showed significant increase in load carrying capacity. Load carrying capacity was increased by 37.5% and deflection in this beam was very much less as compared to control beam and other beams which were retrofitted with single layer prestressed sheet and with single FRP sheet. This beam failed due to debonding of sheet from the beam as shown in **Fig 4.8**.

One main point in the crack pattern of this beam is that the tensile failure of concrete took place on compression side of beam. This indicates concrete reaches its ultimate capacity therefore retrofitting with more than two sheets might not increase the load carrying capacity of beam since concrete will fail at that level. Cracks developed in this were not pure bending cracks as shown in **Fig 4.9**. It is shown in **Fig 4.9** that beam is uplifted from one side which means one part of beam slided up.

Fig 4.8 Debonding of Fiber sheets from the beam

Failure of concrete is due to tension not due to compression which indicates that concrete has reached the stage where it start taking tensile force and failed.

Fig 4.9 Failure pattern of beam retrofitted with double layer prestressed fiber sheet

Load vs deflection curves at center and at L/4 are shown in **Fig 4.9 & 4.10**. Graph shows that load carrying capacity of beam has increased significantly and deflections are reduced as compared to other beams. Load vs deflection curve of this beam is different from other beam. Initially curve is going straight for retrofitted beam as compare to other retrofitted beam. It load is taken by concrete upto 20 KN and then gradually it is transferring to tensile reinforcement resulting in tensile failure of concrete as mentioned above.

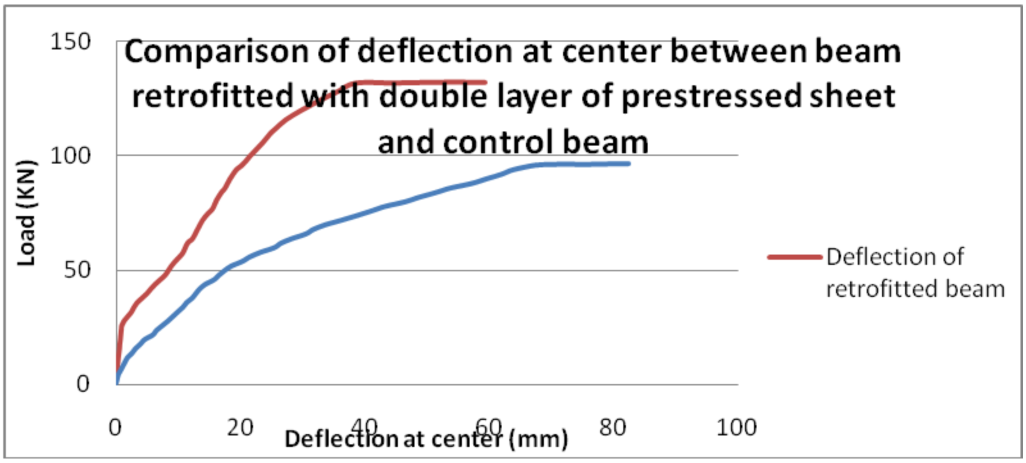


Fig 4.10 Deflection at Center for beam retrofitted with double layer prestressed fiber sheet

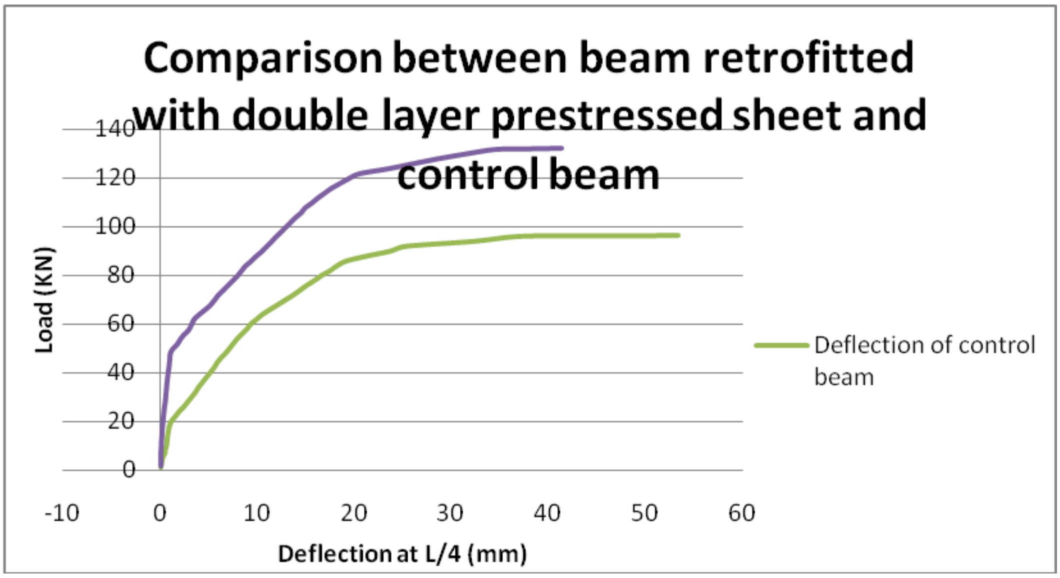


Fig 4.11 Deflection at L/4 for beam retrofitted with double layer prestressed fiber sheet.

4.4 Comparison of beams retrofitted with prestressed and non prestressed fiber sheet

After study it was concluded that load carrying capacity of beam retrofitted with double layer of prestressed sheet is more then other beams. **Fig 4.12** and **Table 4.1** shows the comparison of all the three beams at center. As can be seen in **Fig 4.12** that beam retrofitted with double layer prestressed fiber sheet has more load carrying capacity then other beams and less deflections. Beam retrofitted with double layer prestressed sheet was found to be more stiff at higher loads. Failure mode of all the three beams was different. In prestressed beams, failure occurred due to debonding of sheet and in non prestressed sheet failure was due to fiber fracture. In beam with double layer prestressed sheet, concrete failed in tension. Stiffness of the beam retrofitted with double layer prestressed sheet as found to be more then other retrofitted and control beams. As in **Fig 4.12** beam retrofitted with prestressing fiber initial part of curve is straight indicating that concrete is taking load initally and gradually load is transferring to tensile reinforcement.

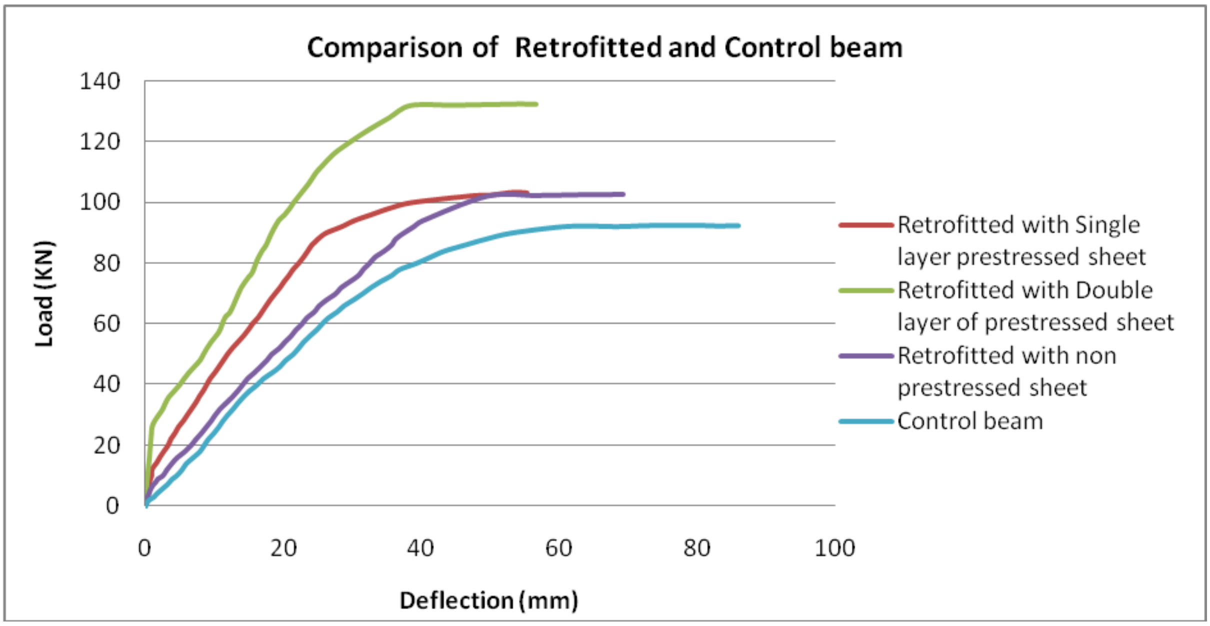


Fig 4.12 Comparison of retrofitted beams

Table 4.1 Comparison of parameters of retrofitted and control beams

Sr No	Beam type	Max load (P in KN)	Max. Deflection(mm)	Stiffness from graph (KN/mm)
1	First fresh beam	92.3	75	5.096
2	Beam Retrofitted with single prestressed sheet	103.3	55.3	6.76
3	Second fresh beam	96.6	82	5.096
4	Beam retrofitted with two prestreesed sheets	132.3	59	9.7
5	Third fresh beam	92.3	78	5.98
6	Beam retrofitted with Non prestressed sheet	102.3	69.3	6.3

CONCLUSIONS

Retrofitting by prestressing Fibre has proven itself to be a better feasible option than other methods. So the future prospects for the utilisation of CFRP in Civil engineering infrastructure are good. Researchers around the world are now looking at the new and innovative ways of utilisation of the same. Based upon the test results of the experimental study undertaken, the following conclusions may be drawn:

1. Load carrying capacity of retrofitted beam was significantly improved as compared to fresh beams.

2. Load carrying capacity of beam retrofitted by double layer prestressed sheet increased by 37% as compared to control beam, beam retrofitted with single layer of prestressed sheet carried 11.4% more load then control beam, beam retrofitted with non prestressed sheet carried 10.8% more load then control beam.

3. Beam retrofitted with double layer prestressed sheet showed significant increase in load carrying capacity and decrease in deflection as compared to beam retrofitted with single layer prestressed sheet

4. Deflection in the beams retrofitted with prestressed beam was less then beam retrofitted with non prestressed sheet and control beam.

5. Beam retrofitted with prestressed fibre was found to be more stiff then control beam and beam retrofitted with non prestressed sheet.

6. Failure in the beam retrofitted with prestressed sheet was due to debonding of sheet from beam but in beam with non prestressed sheet failure was due to fiber fracture.

7. Concrete failed in tension in beam retrofitted with double layer prestressed sheet.

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