RETROFITTING OF RC BEAMS WITH CFRP LAMINATES & SHEETS

A Thesis report submitted
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In
STRUCTURAL ENGINEERING

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CERTIFICATE

This is to certify that the thesis titled, **“RETROFITTING OF RC BEAMS WITH CFRP LAMINATES & SHEETS”**, being submitted by **Mr. PRASHANTSINH BISEN**, 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 thesis has been submitted for the award of any other degree.

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PLACE:

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 & laminate is quite less.

So in the thesis, effect of FRP on RC beams as a retrofitting material is studied. Also comparative effect of laminates with sheets having equivalent area is studied.

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CHAPTER 1

INTRODUCTION

1.1 GENERAL

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. Strengthening 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. 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.

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 fibre and lignin. The cellulose fibre provides strength and the lignin is the "glue" that bonds and stabilizes the fibre. 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 compression strength, whilst 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 & other is fibre based. Fibre based composites are used for civil engineering applications which is composed of fibres 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.

Fibres are added to increase the load-carrying capability of the composite material. The fibres may occupy anywhere from 40 percent to 70 percent (by volume) of the material. These fibres have relatively small diameters. For example, a typical graphite fibre 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 fibre. In the construction industry the most common fibre used is glass fibre (there are 4 types of glass fibres: E-glass, AR-glass, A-glass and high strength glass). Carbon fibre, of which there are 3 types (Type I, II, III) can be used separately or in conjunction with the glass fibre 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 fibre. 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 fibre form, so that, although the same number of random flaws will occur, they will be restricted to a small number of fibres with the remainder exhibiting the material's theoretical strength. Therefore a bundle of fibres will reflect more accurately the optimum performance of the material. However, fibres alone can only exhibit tensile properties along the fibre's length, in the same way as fibres in a rope.

It is when the resin systems are combined with reinforcing fibres 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 fibres and also protects the fibres 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 GFRP, CFRP and 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 ***(Head 1999).***

Table 1.1 Typical mechanical properties of GFRP, CFRP and AFRP composites *(Head 1999)*

Unidirectional advanced composite materials	Fibre content (% BY WT.)	Density (KG/M^3)	Longitudinal tensile modulus	Tensile strength (MPA)
Glass fibre / 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

Regardless of the type of fibres 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. Fig 1.1 shows typical stress strain curves for GFRP, CFRP and Mild steel. 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 existing methods 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

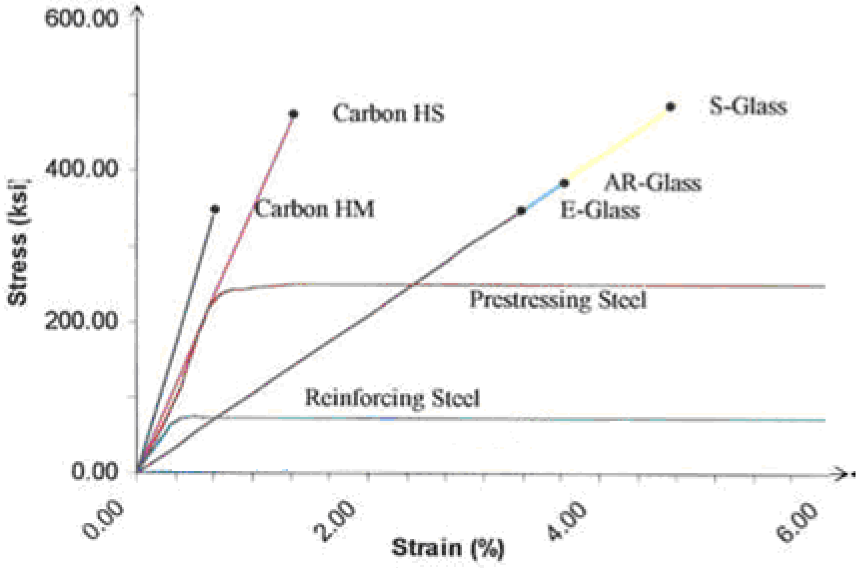


Fig 1.1 Typical FRP and Mild Steel stress strain curves

based on extensive research. In addition, completely new problems arise which also require extensive research.

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 fibre 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.

Fig 1.2 photo showing process of 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 fibres 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 fibre pre-impregnated with resin.

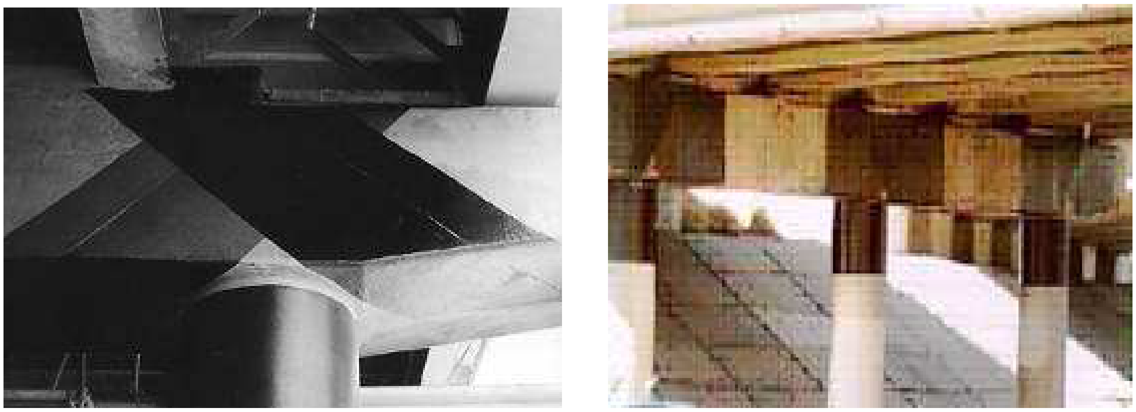


Fig 1.3 Bridge column and bent cap wrapped with FRP

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 fibre pre-preg .Because the carbon fibre 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 1.4)

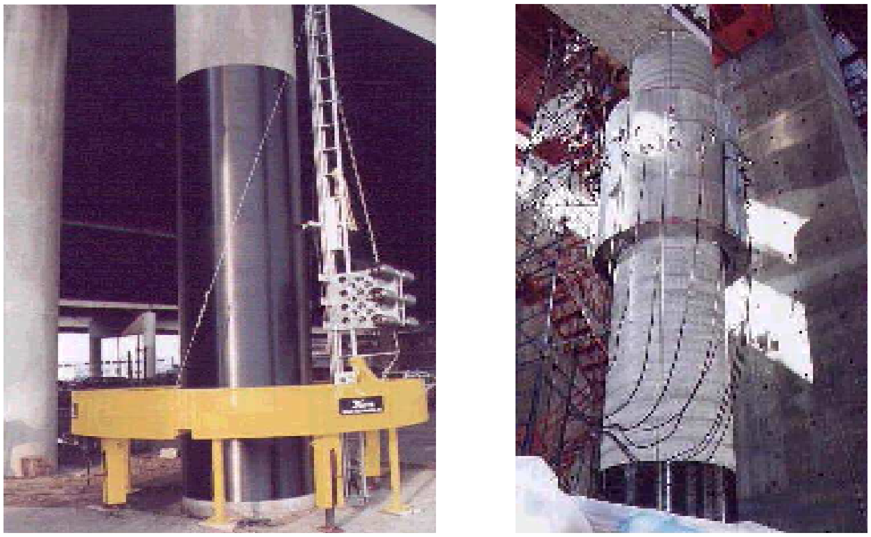


Fig 1.4 Automated column wrapping system and Curing the FRP wrap

- ***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 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. They concluded that CFRP plates for repair of steel bridges

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 FRC as a sheet is quite less and the application of prestressing to the sheet is still lesser. Research work considered in this chapter is divided in three parts, the first section is about the effect of FRP on RC beams followed by the analysis of RC beams wrapped with FRP.

2.2 EFFECT OF FRP ON RC BEAMS

Marco et al. (1997) conducted experiments on strengthened, precracked RC beam specimens. Strengthening was attained with adhesion of CFRP sheets to the concrete surface. Several variables were investigated, including: two CFRP material systems, two concrete surface preparation, two RC cross sections, and number and location of CFRP piles. For two specimens, the presence of applied load (simulating total service load) as well as external prestressing during adhesion of the CFRP reinforcement (reinforcement), were investigated. It is shown that the effect of CFRP strengthening was considerable, but effect of some of the tested variables was modest. Different failure mechanism from ductile to brittle were simulated and verified, adopting mechanical properties of the constituent materials obtained via standard tests or using a simple test for concrete adhesive interface.

Two types of unidirectional CFRP material systems were used with number of piles varying from one to three. The direction of fibres was in most cases arranged in parallel to the axis of beam (longitudinal direction or 0^o) in order to act as flexural reinforcement.

In one case, CFRP sheets were wrapped around three sides of the beam (0^o and 90^o) for shear reinforcement and anchorage of the longitudinal sheets.

The study was divided into two subtopics in order to simulate geometry of shallow & deep beams. To simulate geometry of shallow beams, the height to width ratio (h/b) was kept equal to 0.5; where as deeper section geometry (h/b)= 2 was considered for simulating the behaviour of deep beams (Fig. 2.1). All specimens were tested under four point loading.

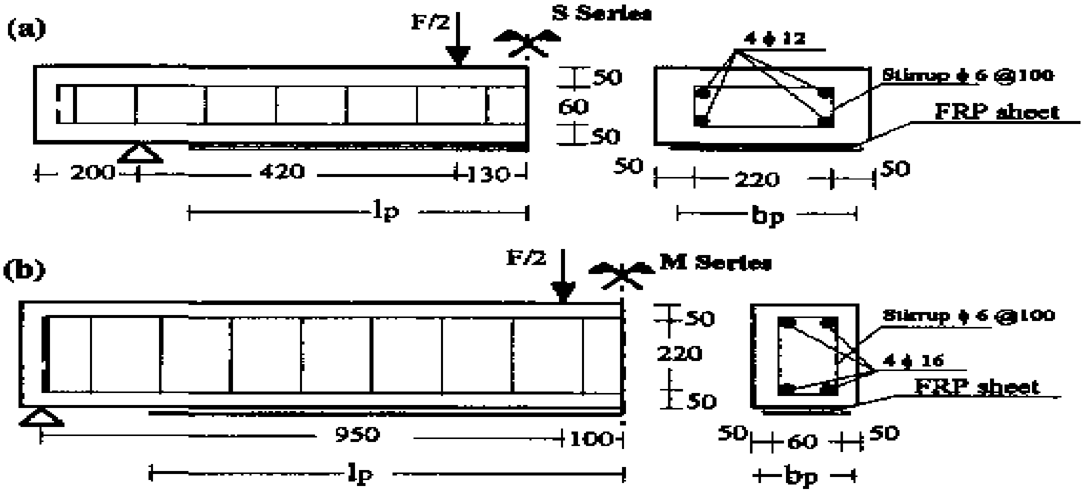


Fig 2.1 Specimen dimensions and test set up in mm

The reinforcement schemes and typical failure mechanisms observed for each beam type are also reported in fig 2.2. The test parameters considered were surface preparation, beam condition and number of CFRP layers. Surface preparation was of two types, sanding & sand blasting respectively. The beam condition was also varied by taking both uncracked (Virgin) and pre-cracked beams.

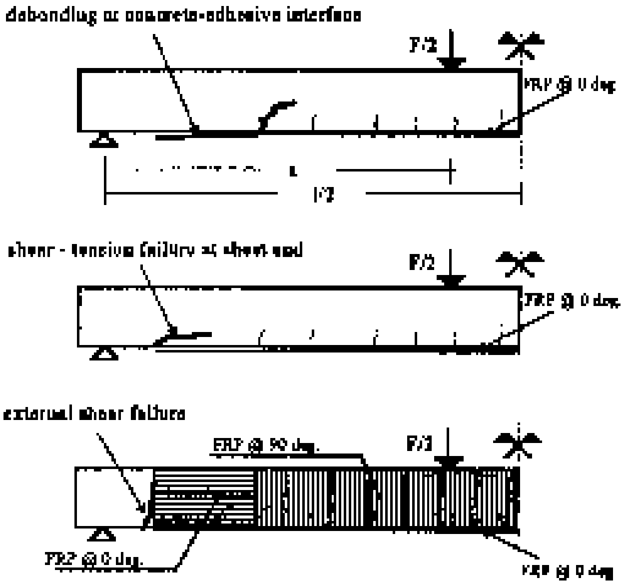


Fig 2.2 strengthening scheme and failure mechanisms

Performance of members strengthened under different loading conditions was considered. Considering specimens, one observes that failure in all cases is controlled by debonding of the FRP; there is not a substantial difference in ultimate capacity of the three specimens.

From the results it was observed that strengthening technology consisting of externally bonded CFRP sheets is easy to perform & ultimate load along with flexural stiffness improves. There is need of attention in FRP concrete adhesion.

Performance of a strengthened precracked specimen (without sealing cracks) is not significantly different from that of strengthened virgin specimen. This is the major conclusion which will be important one for further work. Also FRP strengthening effectiveness depends on cross section shape and amount of steel reinforcement.

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. The test parameters included two levels of steel reinforcement ratio before strengthening and up to four levels.

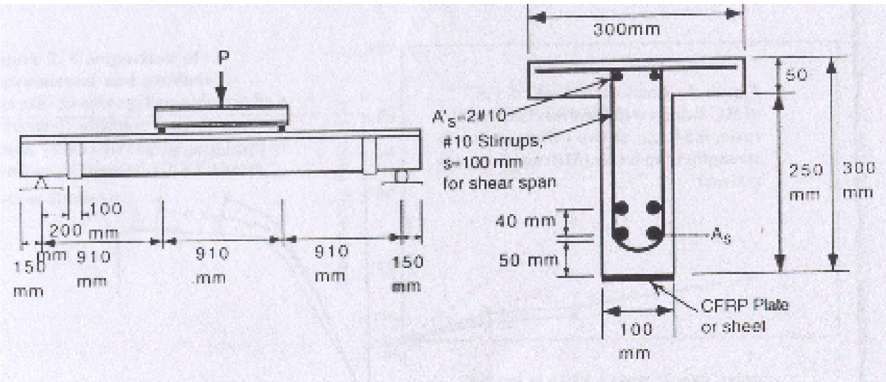


Fig 2.3 Typical cross section of beam and testing set up

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.

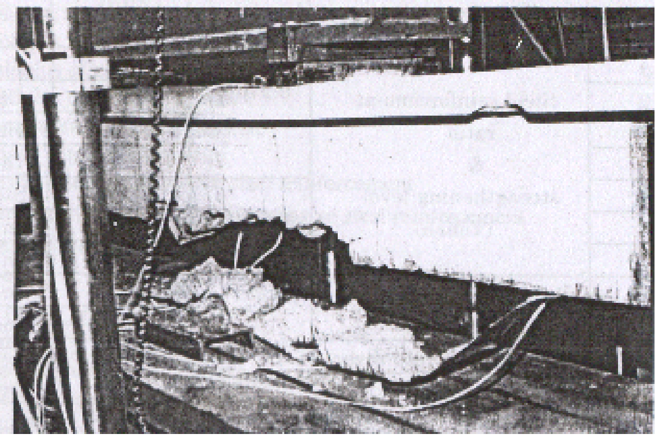


Fig 2.4 Interfacial shear failure of concrete and spalling of concrete cover (Tooth type failure) in beam.

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.

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.

Mukherjee & Rai (In press) 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 prestress. To design 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.

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) did Experimental and numerical analyses are performed to predict the loading carrying capacity of reinforced concrete beams strengthened with carbon fibre 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 X h = 120 X 200, 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 fibre to the support point is the main influence on debonding failure when a single layer fibre is used for strengthening. When the two-layer fibres are used for strengthening, the effect of increase of the length of the second layer of the fibre 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.

Also 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 was carried out. 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 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 fibres.

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.

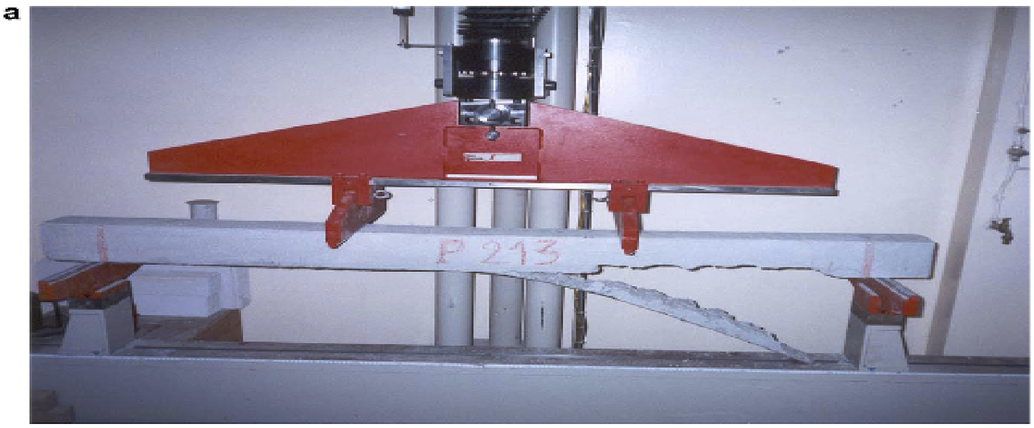


Fig. 2.5 A photo showing a repaired beam failure by peeling off.

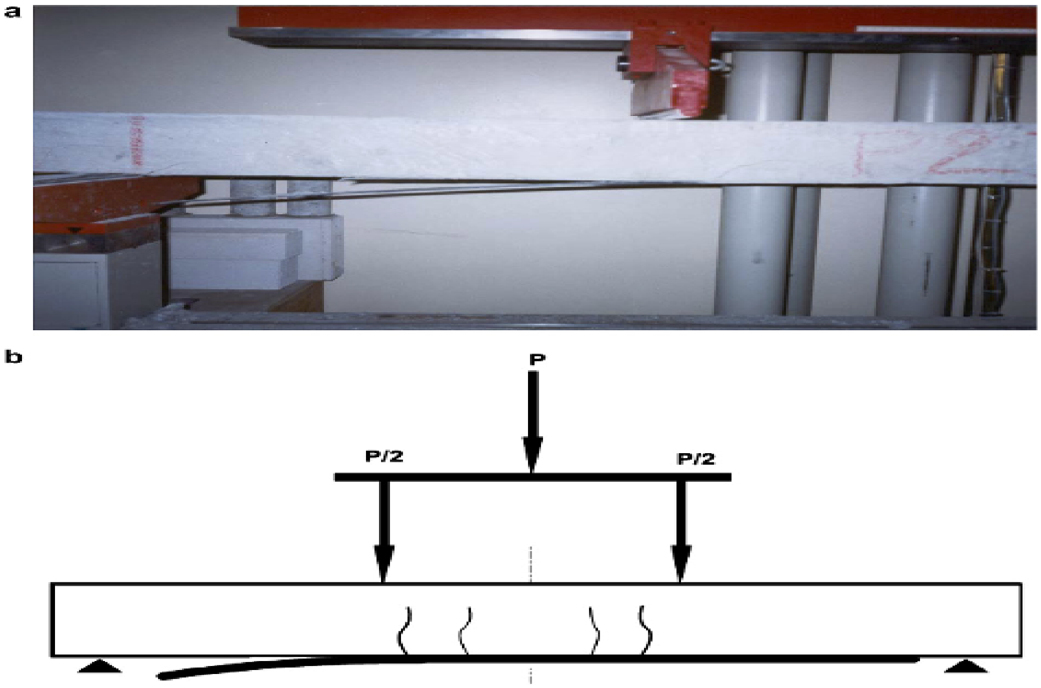


Fig.2.6 A photo showing a repaired beam failure by interfacial debonding and (b) schematic drawing.

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 2.5 & Fig 2.6. 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 & 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.

CHAPTER-3

EXPERIMENTAL PROGRAMME

3.1 INTRODUCTION

The main objective of this experimental programme is to study the behavior of under reinforced concrete beams retrofitted with CFRP laminates & to make comparison of performance of laminates & sheets. To carry out the investigation three real size beams were casted.

3.2 TEST PROGRAMME

The objective of test programme was to find out the properties of materials and the behavior of retrofitted beams. The test programme involved

1. Determinations of basic properties of constituent materials namely cement, sand, coarse aggregates and steel bars as per relevant Indian standard specifications.

2. Three real size beams (600 x 300 x 4100mm) were casted using M 20 grade concrete.

3. The beams are stressed up to the level at which they stopped taking further load and then retrofitted with CFRP laminates & sheets.

3.3 MATERIALS

Cement, fine aggregates, coarse aggregates, reinforcing bars are used in casting of beams, cement slurry with bonding agent for grouting is used for retrofitting of these beams. 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.

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.

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.

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 bras are used as shear stirrups.

3.3.6 CFRP material

For retrofitting two types of CFRPs sheets & laminates were used. Laminates have cross section of 50.8 X 1.4 mm & sheets have 300 X 0.1176 mm.

(A) Laminate (B) Sheet

Fig 3.1 CFRP materials used in the experiment

3.3.7 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 3.5.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.

3.4 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. 3.2.

3.5 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 is 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 is poured in the mould and vibrations are given with the help of needle vibrator, so that the mix gets compacted. The vibration is done until the mould is completely filled and there is no gap left. The beams are then removed from the mould after 48 hours. After demoulding the beams are cured for 28 days using jute bags.

3.6 TESTING ARRANGEMENT

All the three beams were tested under simply supported end conditions. Two points loading is adopted for testing. The testing of beams is done with the help of hydraulic operated jack connected to load cell. The load is applied to the beam with the help of load cell and value is obtained from the data acquisition system, which is attached with the load cell. Five dial gauges are placed, one dial gauge is placed at the center, two dial gauges are place at a distance of span/4 from the end and two below point load. The value of deflection is obtained from these dial gauges. Figure 3.2. shows the testing arrangement.

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

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

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

Table 3.4 Properties of Concrete

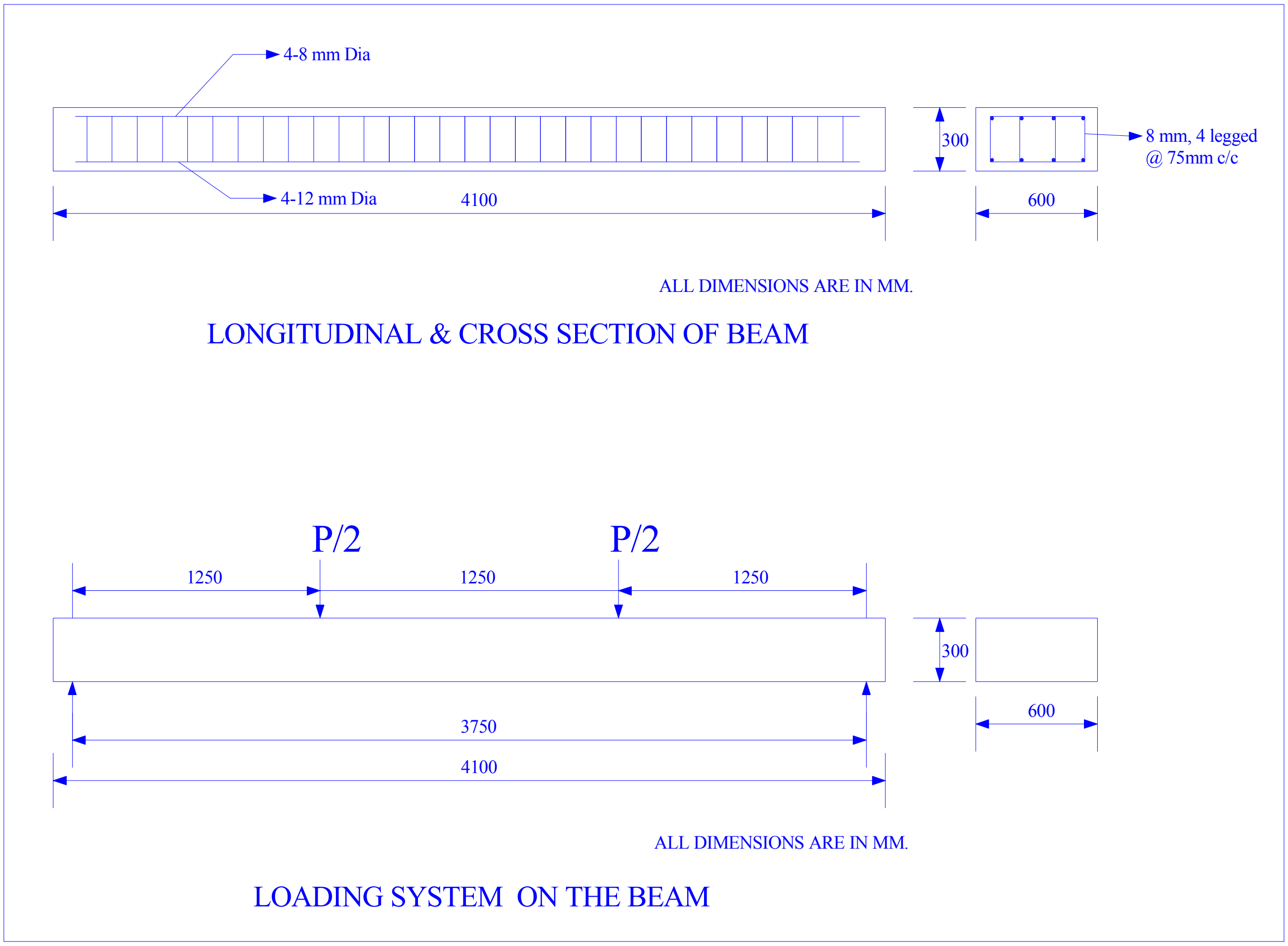
S.NO.	***Physical properties***	***Value***
1.	28 days compressive strength	29.0 N/mm^2

Table 3.5 Properties of CFRP Laminates

S.NO.	***Physical properties***	***Value***
1.	Tensile Strength	2500 N/mm^2
2.	Modulus of elasticity	164000 N/mm^2
3.	Ultimate strain	0.014

Table 3.6 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



LONGITUDINAL & CROSS SECTION OF BEAM

LOADING SYSTEM ON THE BEAM

Fig 3.2 Details of beams & loading system

CHAPTER 4

RESULTS AND DISCUSSION

4.1 INTRODUCTION

In this work effect of number of CFRP laminates, effect of laminates & sheets having equivalent area, on the RCC beam have been studied. A comparative study of flexural strength of control beam and retrofitted beam is presented in the succeeding sections.

4.2 TEST PROCEDURE

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

4.3 BEAM APPLIED WITH TWO LAMINATES

4.3.1 Control beam

First fresh RC beam specimen has been loaded in a four point bend test set up as described in previously. This test has been 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 have been loaded with equal force on the two load points until the beams deformed did not take any 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 as shown in fig 4.1, first crack was observed at 32 KN. As load reached 56 KN more cracks were observed. Almost all cracks were vertical & near top & bottom edge sub cracks were generated connecting to main crack. First crack generated between left point load & beam center. Second crack was just inside the right point load. Third & fourth crack generated 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. At 88 KN spalling started. Major cracking was observed at 90 KN. Beam stopped taking load at 90 KN. Final deflection at the center was 82.65 mm.

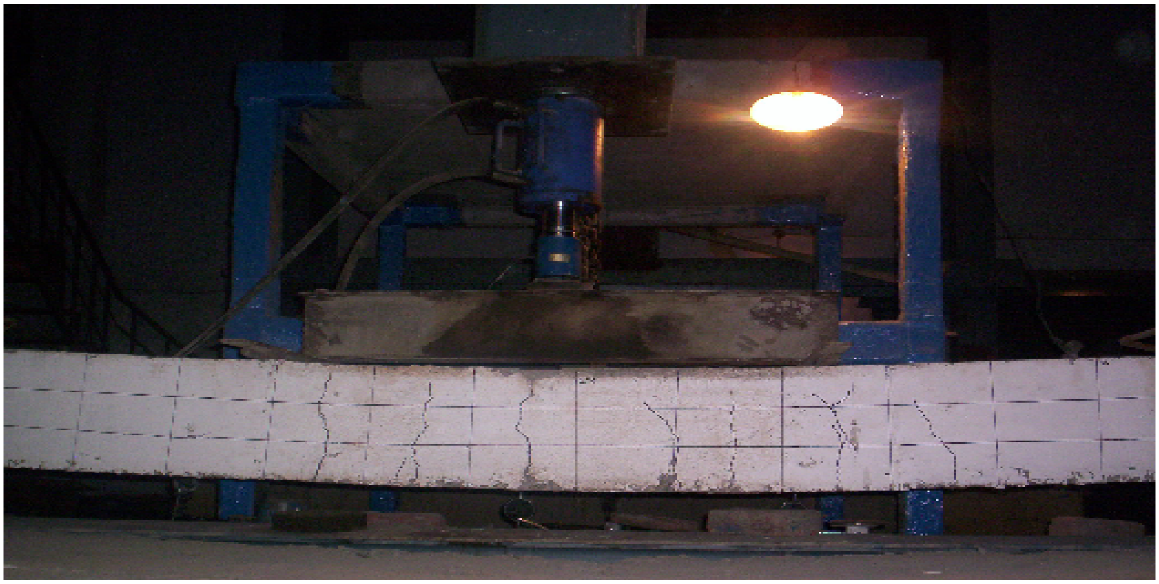


Fig 4.1 Deflected beam & cracks in the central region of the beam

After testing beam was rotated & placed so that its bottom surface will be top surface which can be easily accessed for retrofitting. After rotating Again load was applied on the beam to make it straight.

4.3.2 Method of retrofitting

First surface was grounded by angle grinder having diamond plates. The aggregates should get exposed for better adhesion as shown in fig 4.2. Then for grouting M-seal was applied over the cracks on the three sides of the beam so that slurry will not come out. Cement slurry was made by mixing Cement, Sand & bonding agent of Dr. fixit (Pidilite) company. Slurry was inserted in the cracks with pressure. After grouting, surface was cleaned with acetone & then Primer having product name MBrace primer of BASF company was applied. Primer was prepared by mixing base & hardener & was applied with brush on smooth surface as shown in fig 4.2. Tack free time of primer is six hours.

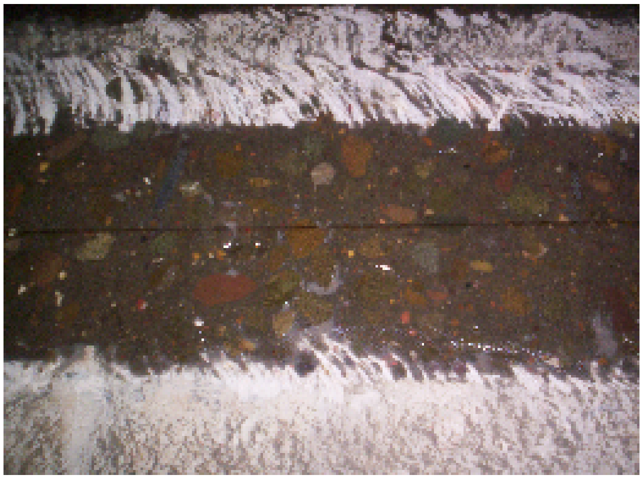


Fig 4.2 Beam surface after grinding & priming

After priming MBrace putty was applied which is 100% solid, non sag paste epoxy resin material. Putty was applied for leveling the uneven surface. Setting time for Putty is 24 hours. After setting of putty, Laminates were applied. Laminates of S & P company having product name CFK laminates were cut having length 50 mm shorter than center to center distance of supports to ensure that no artificial anchorage of the laminate takes place due to support reaction. Adhesive was applied on the surface of laminates & laminates were put on the beam. Then roller was rolled on the laminate to make proper bond of the laminate & beam surface. Beam was left for 4 days of epoxy curing. After curing beam was again rotated to restore its position & now surface with two laminates was at the bottom.

4.3.3 Testing of the beam with two laminates

Load was applied on the beam, spalling started at 52 KN load. Crackling sound of laminates was observed at 66 KN & 96 KN. At 100 KN again spalling was observed as shown in fig 4.3. At 108 KN laminate failed by interfacial debonding as shown in fig 4.4. loading was stopped after laminate failure. After unloading beam immediately recovered by 20 mm. Readings of both test are given in table 4.1. Graphs obtained are as shown in Fig 4.5 & Fig 4.6. It can be observed that, controlled beam failed at 90 KN & the same beam after application of 2 laminates failed at 108 KN. There was a sound like a blast while debonding of laminate.

Fig 4.3 Starting of spalling at the bottom surface

Fig 4.4 Beam with two laminates failed by interfacial debonding.

Table 4.1 Load Vs deflection data for control beam & beam with two laminates

Control beam			Beam with two laminates		
Load (KN)	Deflection at center (mm)	Deflection at L/4(mm)	Load (KN)	Deflection at center(mm)	Deflection at L/4(mm)
0	0	0	0	0	0
1	0.03	0.1	1	0.26	0.24
2	0.06	0.12	2	0.38	0.32
3	0.12	0.21	3	0.6	0.51
4	0.23	0.27	4	0.8	0.65
5	0.35	0.35	5	1.07	0.87
6	0.46	0.45	6	1.43	1.13
7	0.55	0.52	7	1.69	1.35
8	0.64	0.6	8	1.87	1.51
9	0.74	0.69	9	2.15	1.73
10	0.79	0.74	10	2.48	1.96
12	0.91	0.83	12	2.9	2.31
14	1.05	0.95	14	3.5	2.71
16	1.2	1.08	16	4.11	3.12
18	1.33	1.19	18	4.7	3.51
20	1.48	1.49	20	5.4	3.91
22	1.73	1.7	22	5.95	4.41
24	2.05	1.91	24	6.58	4.81
26	2.34	2.27	26	7.35	5.31
28	2.84	2.68	28	8.15	5.79
30	3.46	2.95	30	8.71	6.29
32	3.82	3.4	32	9.27	6.59
34	4.49	3.84	34	9.81	6.95
36	5.15	4.15	36	10.45	7.14

38	5.62	4.56	38	11.1	7.81
40	6.2	5.38	40	12.05	8.41
42	7.38	5.78	42	12.75	8.86
44	7.98	6.25	44	13.4	9.31
46	8.62	6.68	46	14.25	9.86
48	9.25	7.15	48	15.75	10.81
50	9.9	7.6	50	17.03	11.71
52	10.55	7.95	52	18.05	12.41
54	11	8.09	54	19.05	13.09
56	11.8	8.09	56	20.1	13.83
58	11.8	8.5	58	20.95	14.41
60	12.3	8.5	60	21.65	14.86
62	13.06	8.5	62	22.62	15.53
64	13.58	8.75	64	23.45	16.13
66	14.2	8.86	66	24.45	16.81
68	14.78	8.95	68	25.38	17.46
70	15.71	9.25	70	26.42	18.21
72	15.89	9.48	72	27.55	18.95
74	16.09	9.64	74	29	19.96
76	16.51	9.92	76	29.85	20.56
78	17.36	10.32	78	30.2	21.26
80	17.84	10.72	80	30.9	21.76
82	18.26	11.28	82	31.29	22.46
84	19.35	15.28	84	31.65	22.96
86	22	27.23	86	31.91	23.66
88	43.8	35.38	88	32.45	24.24
90	54.24	37.88	90	34.35	24.86
90	58.25	40.01	92	35.25	25.61
90	62.29	41.77	94	36.2	26.46
90	65.19	41.67	96	37.35	27.06

90	69.75	49.95	98	38.55	27.71
90	79.33	51.84	100	39.45	28.41
90	82.65	57.38	102	40.4	29.21
			104	41.4	30.11
			106	42.55	30.91
			108	49.8	38.21

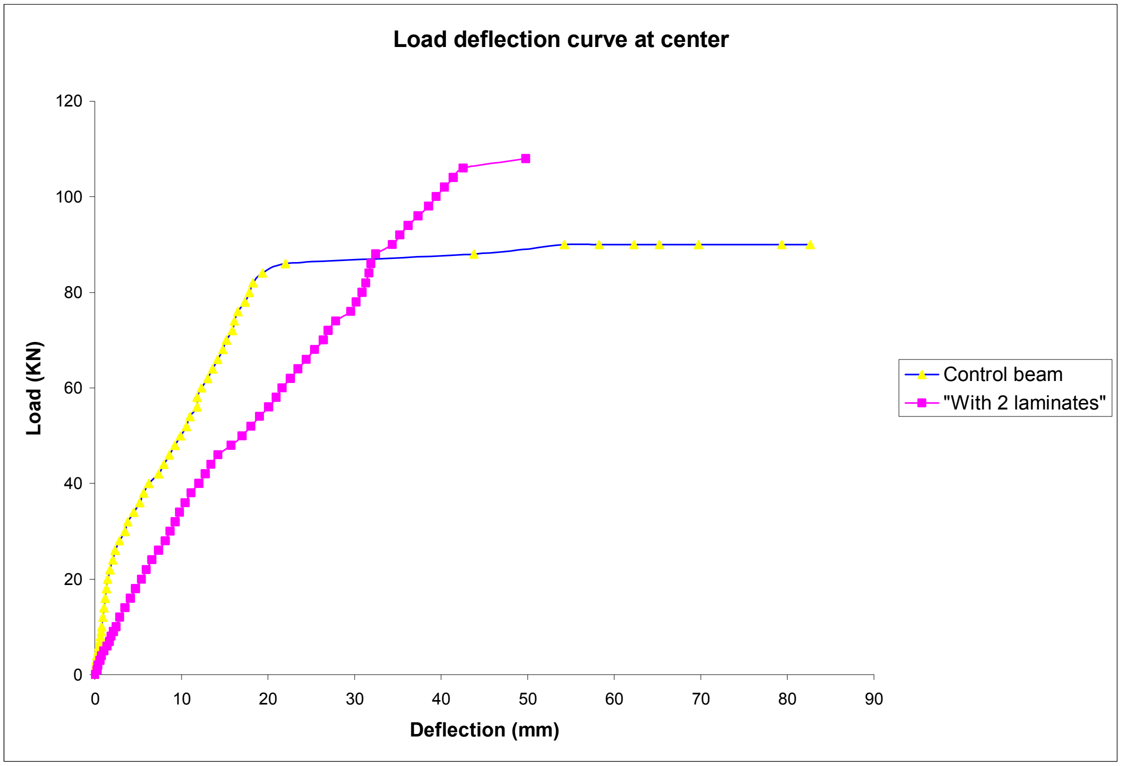


Fig 4.5 Graph of load deflection curve at the center of the beam

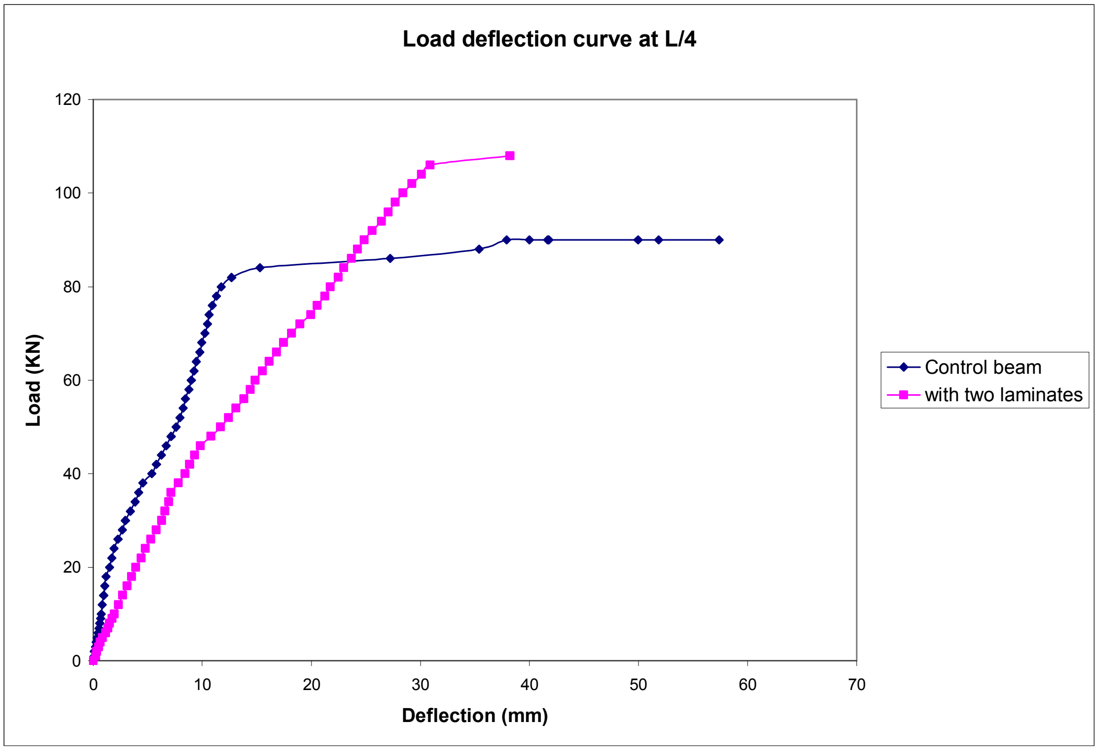


Fig 4.6 Graph of load deflection curve at L/4 from support

4.4 BEAM APPLIED WITH THREE LAMINATES

4.4.1 Control beam

Fresh RC beam was tested as the previously tested control beam. Beam failure was similar to first beam, with major bending cracks in central region. Beam failed at 80 KN. The same process of retrofitting was carried out with the change in number of laminates from two to three. Readings of load & deflection are given in table 4.2. Graphs of load deflection curve are shown in fig 4.10, 4.11 & 4.12.

4.4.2 Beam with three laminates

When loading was started new crack was observed at 35 KN. Crackling sound was there at 60 KN , 65 KN , 75 KN & 80 KN. Spalling started at 105 KN. Also at 105 KN crackling sound was there in fibres. Spalling was there at 148 KN. Spalling was at top & bottom surface edges as shown in Fig 4.7. Also concrete failed in compression in central zone of the top surface around the cracks as shown in Fig 4.8.At 178 KN load two of the three laminates failed, making the sound like a blast. Deflection at the center was 104.68 mm. Partial peeling off & partial debonding failure was observed as shown in fig 4.9. Readings of load & deflection are given in table 4.2. Graphs of load deflection curve are shown in fig 4.10, 4.11 & 4.12.

Fig 4.7 Photo showing starting of spalling at the top surface.

Fig 4.8 Photo showing concrete failure in compression at top surface

Fig 4.9 Photo showing failure of the beam by partial peeling off & debonding of laminates

Table 4.2 Load Vs deflection data for control beam & beam with three laminates

Deflection of Control beam				Deflection of beam having 3 laminates			
Load (KN)	At center (mm)	At point load (mm)	At l/4 (mm)	Load (KN)	At center (mm)	At point load (mm)	At l/4 (mm)
0	0	0	0	0	0	0	0
2	0.55	0.51	0.37	1	1.21	0.64	0.45
4	0.75	0.71	0.52	2	3.16	1.51	1.46
6	1	0.95	0.7	3	3.97	3.25	2.01
8	1.34	1.24	0.92	4	5.56	4.65	2.8
10	1.64	1.52	1.13	6	5.66	5.35	3.83
12	1.88	1.76	1.31	8	7.65	5.76	4.17
14	2.46	2.27	1.69	10	8.43	5.89	4.79
16	2.88	2.67	1.99	12	9.34	6.10	5.29
18	3.19	2.93	2.19	15	10.56	7.02	6.48
20	3.44	3.17	2.37	19	12.39	9.00	7.57
22	3.7	3.39	2.54	22	13.67	10.32	8.51

24	3.9	3.58	2.69	25	14.67	11.51	8.94
26	4.21	3.95	2.87	30	16.18	13.03	9.79
28	4.52	4.1	3.08	35	18.63	14.36	10.66
30	4.8	4.37	3.27	40	20.17	15.45	11.80
32	4.98	4.51	3.39	45	22.75	16.52	13.03
34	5.28	4.78	3.58	50	25.25	18.43	14.98
36	5.48	4.97	3.74	55	28.6	20.80	15.92
38	5.79	5.24	3.96	60	30.8	22.5	17.16
40	6.08	5.49	4.15	65	33.75	24.52	18.78
42	6.32	5.71	4.32	70	37.4	28.02	20.93
44	6.66	6.02	4.52	75	39.8	29.35	22.45
46	6.93	6.28	4.73	80	42.05	30.8	24.56
48	7.36	6.66	5.02	86	44.9	32.62	25.44
50	8.38	6.88	5.18	92	47.05	34.91	26.1
52	8.82	7.1	5.36	98	53.35	36.04	28.27
54	9.24	7.58	5.71	105	57.53	40.33	30.55
56	9.69	7.98	6.02	110	59.04	41.71	31.5
58	10.08	8.37	6.32	114	60.65	43.16	32.55
60	10.68	8.78	6.62	115	62.01	43.67	33.17
62	11.28	9.12	6.89	118	63.35	45.60	34.02
64	12.63	9.67	7.32	120	64.27	46.81	35.03
66	15.13	10.15	8.29	122	65.03	47.39	35.44
68	18.83	11.21	9.74	125	65.7	48.16	35.98
70	21.64	13.13	11.69	127	66.5	48.86	36.43
72	24.03	15.87	13.24	130	67.45	49.74	37.02
74	37.59	17.23	14.5	132	68.35	50.60	37.41
76	64.59	19.43	16.19	134	69.45	51.60	38.48
78	71.52	21.6	18.42	136	70.48	52.78	39.19
80	88.74	21.83	19.58	138	71.53	53.63	39.6
80	95.46	21.88	20.18	140	72.58	54.33	40.34

80	105	21.97	20.90	142	73.67	55.22	40.92
80		22.12	21.79	144	74.55	56.09	41.44
80		34.41	37.02	146	75.35	56.77	41.98
80		41.26	39.84	148	76.14	57.59	42.57
				150	77.35	58.78	43.37
				152	78.3	59.63	44.24
				154	79.03	60.4	44.95
				156	80.47	61.70	45.67
				157	81.33	62.7	47.19
				159	87.13	65.18	48.10
				160	88.25	66.17	49.05
				162	89.18	66.66	49.61
				164	89.88	68.27	50.61
				167	92.42	69.52	51.82
				169	93.98	71.79	54.53
				170	96.27	72.86	55.22
				172	97.38	73.59	55.84
				174	98.43	74.71	56.52
				176	99.88	75.90	57.08
				178	104.68	77.1	57.94

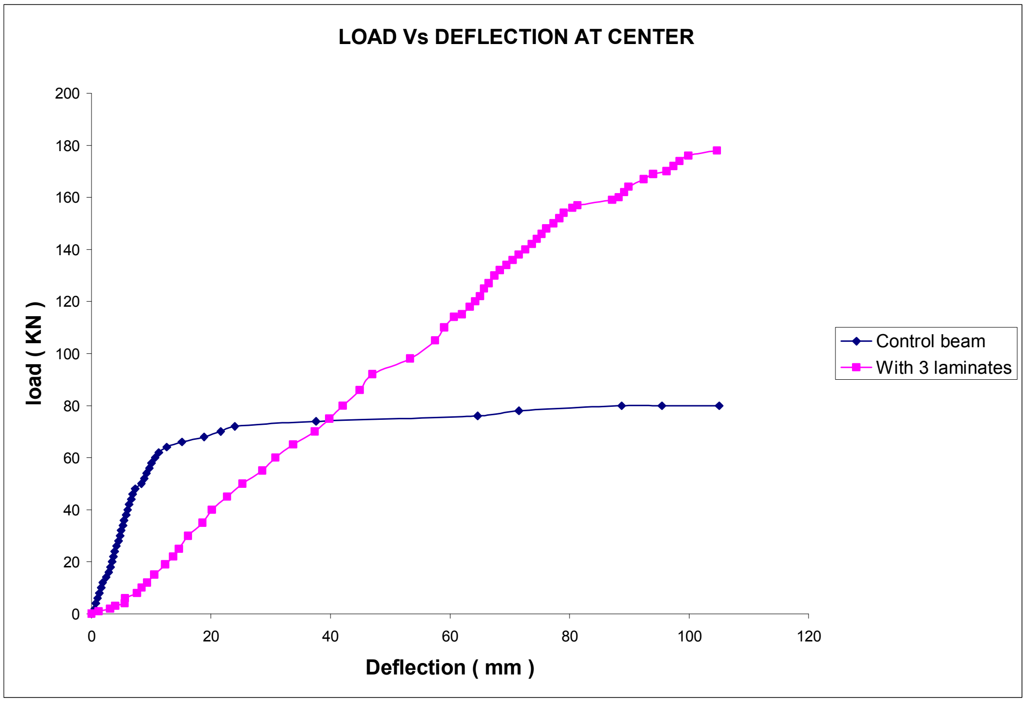


Fig 4.10 Load deflection curve at the center of the beam

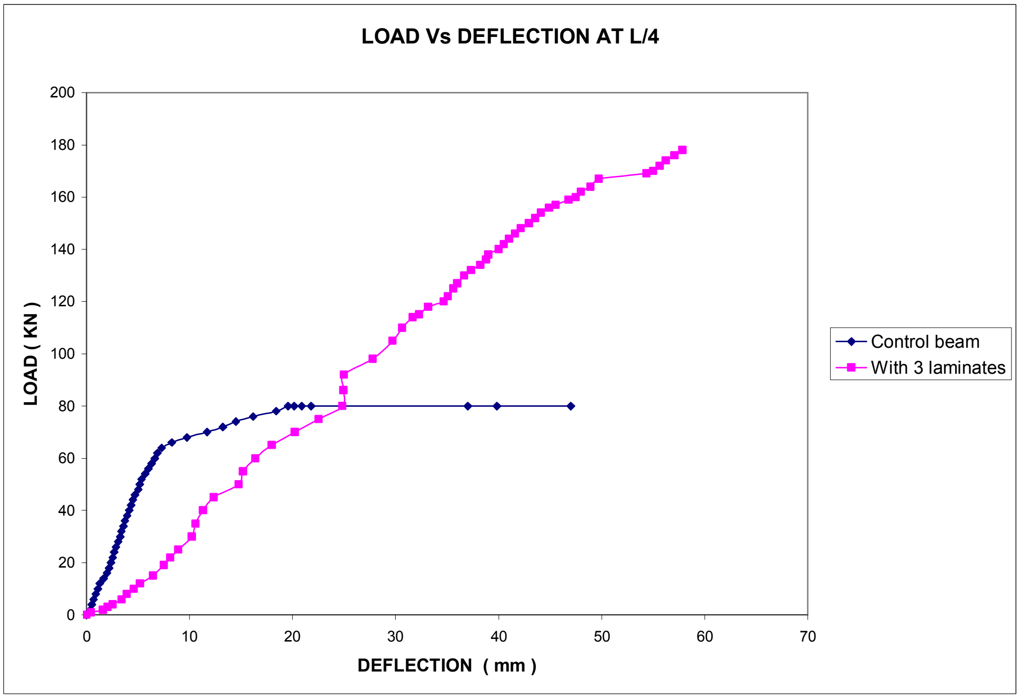


Fig 4.11 Load deflection curve at the L/4 from support of the beam

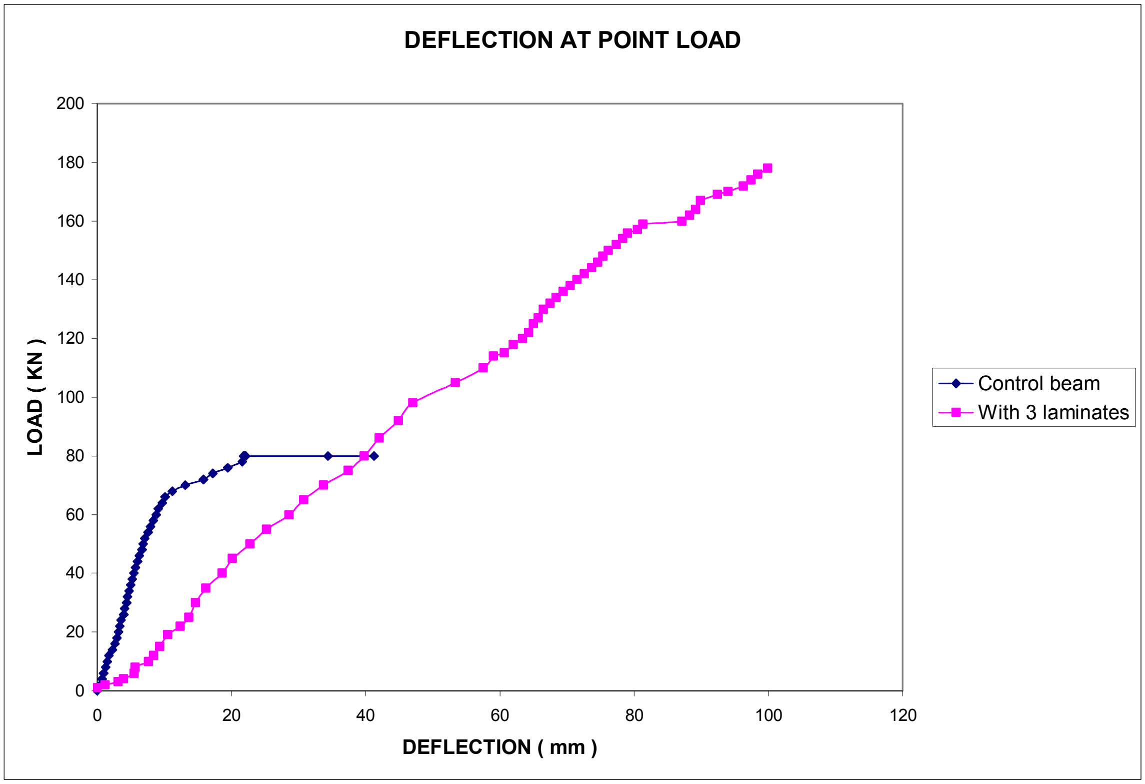


Fig 4.12 Load deflection curve under the point load of the beam

4.5 BEAM APPLIED WITH SHEETS

4.5.1 Control beam

Fresh RC beam was tested as the previously tested control beam. Beam failure was similar to first beam, with major bending cracks in central region. Beam failed at 89 KN with final deflection at the center 53.17 mm. Recovery of the beam after unloading was 15mm. The same process of retrofitting was carried out with the change in FRC material. Load deflection curves obtained from the test are given in Fig 4.17 to 4.19.Refer Table No 4.3 for readings.

4.5.2 Beam with Sheets

To compare performance of laminate & sheet, equivalent area of sheet was calculated considering two strips. So equivalent width was calculated using following formula,

$B_S \times T_S \times E_S = 2 \times (B_L \times T_L \times E_L)$ ________________ (1)

Here in equation (1),

B_S & B_L implies width of sheet & laminate respectively.

T_S & T_L implies thickness of sheet & laminate respectively.

E_S & E_L implies Modulus of elasticity of sheet & laminate respectively.

$B_L = 50.8$ mm

$T_L = 1.4$ mm

$E_L = 164 \times 10^3$ N/mm^2

$T_S = 0.1176$ mm

$E_S = 240 \times 10^3$ N/mm^2

Substituting above values in equation (1)

$B_s \times 0.1176 \times 240 \times 10^3 = 2 \times (50.8 \times 1.4 \times 164 \times 10^3)$

$B_s = 826.5$ mm.

Width of the beam is 600 mm & FRC sheet available in 300 mm width. So to use width of 826.5 mm three layers of FRC sheet should be applied. Two layers of 300 mm directly on beam surface & one layer on the sheets in the center of the surface will be applied. After grouting, grinding , priming & application of MBrace putty we can apply sheet. First epoxy was applied on the sheet & then sheet was placed on the surface as shown in fig 4.13. Then roller was rolled from the center of the beam towards ends on the sheets. Pressure was applied on the sheet with the help of roller so that we can see epoxy coming out on the top surface of the sheet from the gap of the fibres. Beam was left for epoxy curing for 4 days.

When load was applied on the beam, first crackling sound was heard at 20 KN, and after that again at 51 KN, 65 KN & at 111 KN sound of FRC sheet was heard. Spalling was observed at 101 KN & at 115 KN. Concrete failed in compression at top as shown in Fig 4.14.At bottom new cracks were observed & width of previous cracks was widen. Also concrete was crushed as shown in fig 4.15. Beam stopped taking further load at 127 KN. As compared to laminates, sheets did not fail over large area, only slight debonding was there in the center as shown in the Fig 4.16. Debonding was not throughout the width of the beam. It was near edges only as at the edges only single layer of the sheet was there. Also increase in load carrying capacity was observed as beam with two laminates failed at 108 KN & beam with sheets failed at 127 KN. So there was increase of 19 KN in load carrying capacity. But it was less than beam applied with three laminates by 51 KN. Load deflection curves obtained from the test are given in Fig 4.17 to 4.19.Refer table no 4.3 for readings.

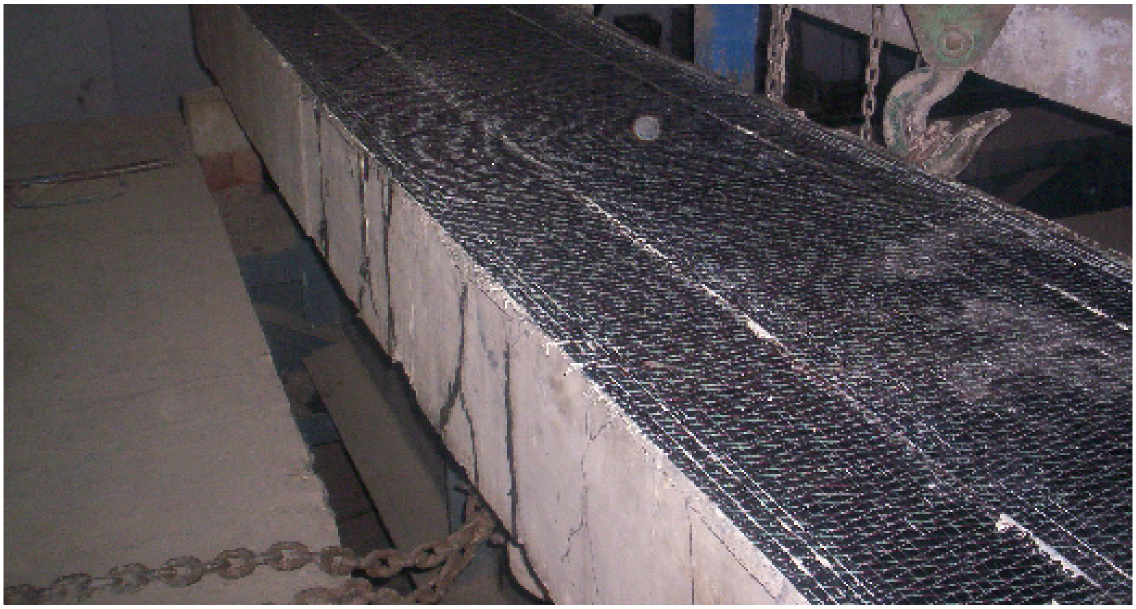


Fig 4.13 Photo showing applied sheets on the surface.

Fig 4.14 Photo showing failed concrete in compression at the top surface.

Fig 4.15 Photo showing crushed concrete just above the laminate; widen cracks & new cracks at bottom.

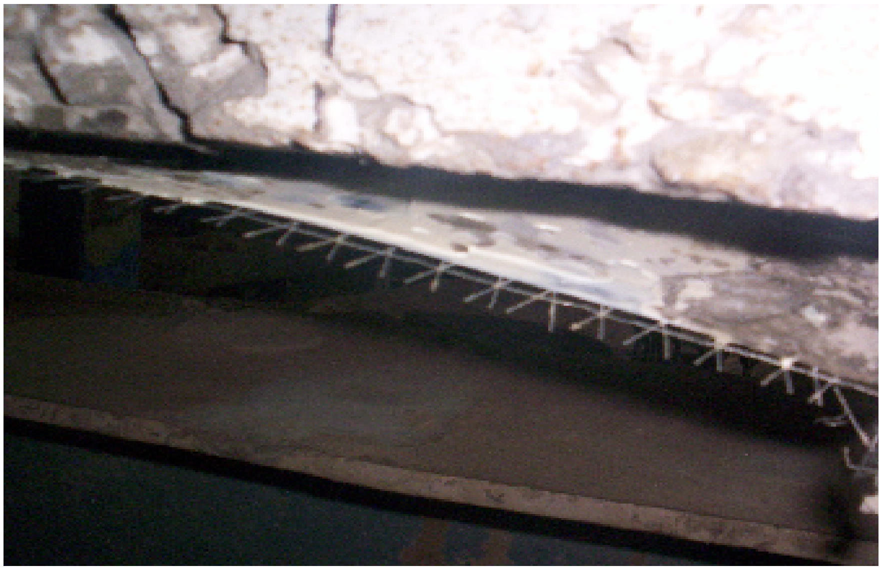


Fig 4.16 Photo showing debonding of sheet

Table 4.3 Load Vs deflection data for control beam & beam with Sheets

Deflection of Control beam				Deflection of beam with sheet			
Load (KN)	At center (mm)	At Point load (mm)	At L/4 (mm)	Load (KN)	At center (mm)	At Point load (mm)	At L/4 (mm)
0	0	0	0	0	0	0	0
1	0.06	0.03	0.02	1	0.17	0.12	0.11
2	0.19	0.1	0.08	2	0.44	0.3	0.16
3	0.27	0.21	0.14	3	0.83	0.56	0.36
4	0.53	0.44	0.23	4	1.22	0.85	0.58
5	0.89	0.66	0.42	5	1.61	1.16	0.82
6	1.29	1.11	0.67	6	2.04	1.51	1.07
7	1.83	1.53	1	7	2.63	1.95	1.39
8	1.99	1.82	1.22	8	3.18	2.37	1.74
9	2.23	1.99	1.46	9	3.5	2.57	1.9
10	2.66	2.25	1.77	10	4.14	3.15	2.265
12	2.8	2.62	2.21	12	5.27	4.02	2.94

14	3.3	3.17	2.51	14	6.2	4.77	3.405
16	3.97	3.65	2.86	16	7.31	5.66	3.985
18	4.15	3.97	3.15	18	8.11	6.33	4.44
20	4.65	4.3	3.44	20	10.07	7.9	4.855
22	4.81	4.60	3.69	28	13.59	12.77	8.04
24	5.2	4.84	3.87	35	16.54	16.54	11.09
26	5.56	5.19	4.06	45	23.12	20.1	12.66
28	5.74	5.48	4.22	47	24.62	23.7	13.15
30	5.93	5.76	4.42	49	25.72	24.58	13.82
32	6.2	6.07	4.78	51	27.67	25.33	14.38
34	6.78	6.59	5.13	53	27.82	26.05	14.96
36	7.27	6.95	5.38	55	28.45	26.77	15.48
38	7.59	7.22	5.63	57	29.32	27.52	16.1
40	8	7.56	5.86	59	30.3	28.06	16.51
42	8.29	7.84	6.08	61	31.22	28.77	17.20
44	8.62	8.02	6.27	63	32.34	29.62	18.04
46	8.85	8.25	6.46	65	33.4	30.67	18.81
48	9.12	8.48	6.65	67	34.69	31.6	19.4
50	9.4	8.73	6.83	69	35.82	32.56	20.34
52	9.64	8.95	7	71	36.92	33.5	20.65
54	9.92	9.17	7.19	73	38.02	34.35	21.22
56	9.92	9.43	7.41	75	38.87	35.2	21.73
58	10.5	9.7	7.62	77	39.72	35.87	22.32
60	10.78	9.95	7.83	79	40.57	36.64	22.79
62	11.13	10.25	8.04	81	41.52	37.35	23.48
64	11.4	10.49	8.28	83	42.32	38.12	23.72
66	11.78	10.89	8.53	85	43.33	38.82	24.47
68	12.08	11.1	8.76	87	44.19	39.65	25.19
70	12.42	11.38	9.02	89	45.47	40.17	26.04
72	12.84	11.8	9.30	91	48.84	41.08	26.81

74	13.24	12.12	9.56	93	49.83	43.63	27.78
76	13.6	12.48	9.86	95	50.69	44.27	28.65
78	14.12	12.94	10.43	97	51.29	44.67	29.34
80	15.45	13.98	12.17	99	51.99	45.23	29.98
81	19.18	16.9	14.28	101	53.99	45.82	30.58
82	20.75	18.1	15.18	103	55.09	46.77	31.21
83	22.15	19.24	16.75	105	56.29	47.77	32.42
84	25	21.44	18.75	107	57.71	48.72	33.26
85	29.3	24.56	20.86	109	58.77	49.82	33.99
86	32.18	26.4	22.97	111	59.98	50.76	35.17
87	36.15	29.1	25.78	113	61.19	51.80	35.53
88	41.12	32.9	27.49	115	61.6	54.02	36.63
89	47	36.4	30.24	117	63.24	54.94	37.49
89	51.95	40.13	34.11	119	65.34	56.66	39.48
89	53.17	41.64	37.27	121	68.64	58.47	41.61
				123	78.06	60.82	43.50
				125	80.69	62.98	45.15
				127	93.04	70.05	51.84
				127	96.74	71.12	53.34
				127	100.74	73.89	
				127	104.24		
				127	111.24		

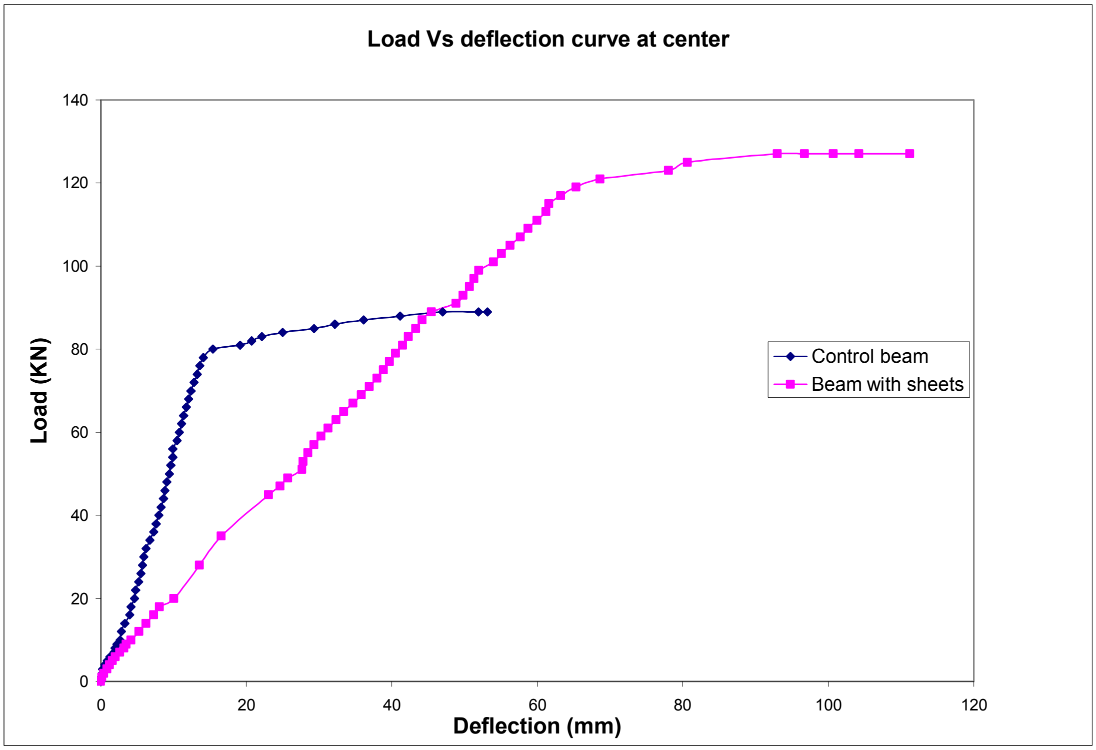


Fig 4.17 Load deflection curve at the center of the beam

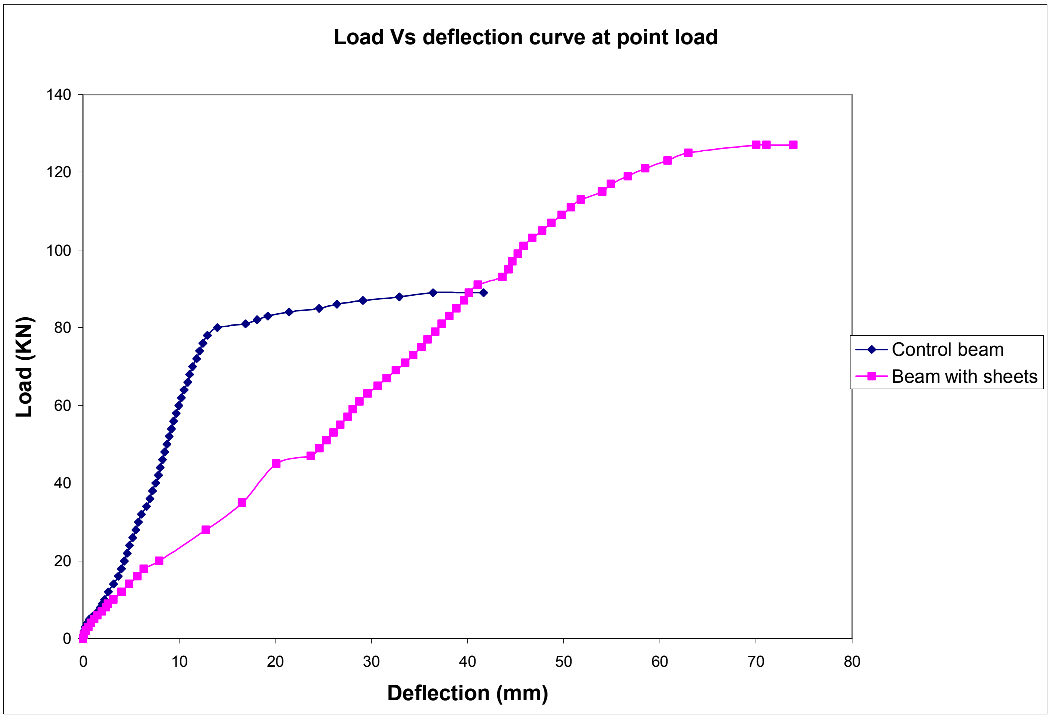


Fig 4.18 Load deflection curve under the point load of the beam

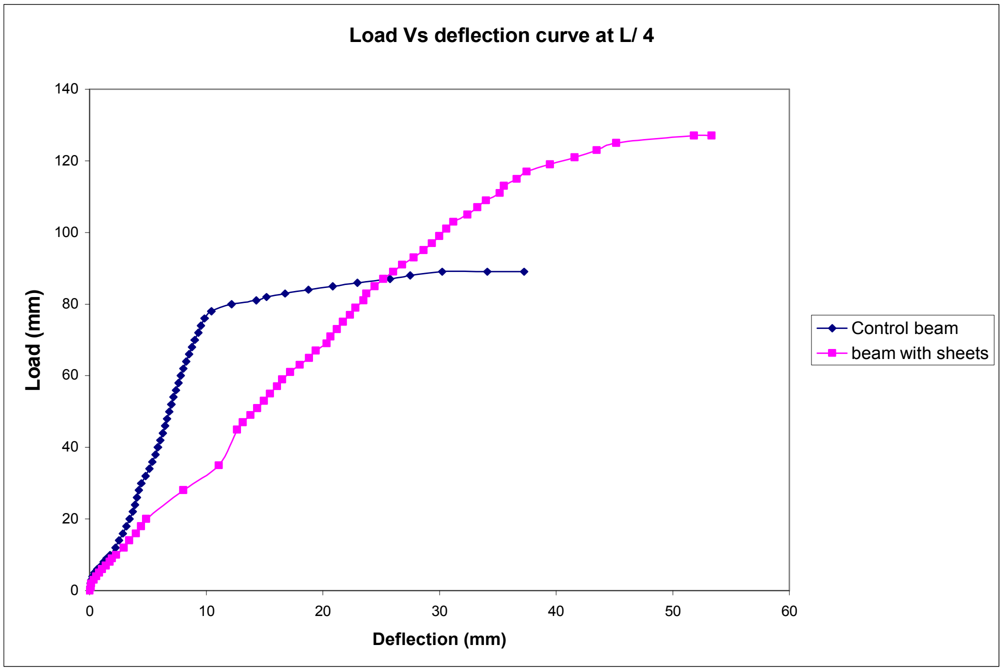


Fig 4.19 Load deflection curve at L/4 from support of the beam

Table No 4.4 Values of work done & initial stiffness.

Sr No	Beam type	Max load (P in KN)	Max. Deflection(mm)	Work done* (KN-mm)	Initial stiffness from graph (KN/mm)
1	First fresh beam	90	82.65	1880.9	5.05
2	Beam with two laminates	108	49.8	3267.51	3.46
3	Second fresh beam	80	105	2232	5.75
4	Beam with three laminates	178	104.68	10143.11	1.42
5	Third fresh beam	89	53.17	2362	5.64
6	Beam with sheets	127	111.24	7994.68	2

(*Work done is Area under the load deflection curve of graph considering tangential stiffness as a parameter for comparison)

CONCLUSION

CFRP for retrofitting 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. Beam with two laminates at bottom have taken 20% more load than fresh beam.
3. Beam with three laminates at bottom have performed well as it has taken 122.5% more load than fresh beam.
4. Beam with sheets applied at bottom have shown 42.7% increase in load carrying capacity as compared to fresh beams.
5. Considering tangential stiffness as a limiting parameter, work done by retrofitted beams is more than control beams. With respect to control beams work done is increased by 73.72% for beam with two laminates, 354.44% for beam with three laminates & 238.47% for beam with sheets.
6. Comparing beam applied with two laminates & beam applied with sheets having equivalent area, Sheets have taken 17.59% more load than laminates.
7. As compared to two laminates, failure of three laminates was quite satisfactory as there was partial peeling off.
8. Due to debonding total utilization of strength of laminates was not achieved.

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