EXPERIMENTAL AND NUMERICAL STUDY ON BEHAVIOR OF EXTERNALLY BONDED RC T-BEAMS USING GFRP COMPOSITES

A THESIS SUBMITTED IN PARTIAL FULFILMENT

OF THE REQUIREMENTS FOR THE DEGREE OF

Master of Technology

In

Structural Engineering

By

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ROURKELA, ORISSA

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CERTIFICATE

This is to certify that the thesis entitled, ***"EXPERIMENTAL AND NUMERICAL STUDY ON BEHAVIOR OF EXTERNALLY BONDED RC T-BEAMS USING GFRP COMPOSITES"*** *submitted by* **Trishanu Shit** *in partial fulfillment of the requirements for the award of Master of Technology Degree in Civil Engineering with specialization in "***Structural Engineering***" at National Institute of Technology, Rourkela is an authentic work carried out by him under my supervision and guidance. To the best of my knowledge, the matter embodied in this Project review report has not been submitted to any other university/ institute for award of any Degree or Diploma.*

Date: *24th MAY 2011*

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ABSTRACT

Fiber-reinforced polymer (FRP) application is a very effective way to repair and strengthen structures that have become structurally weak over their life span. FRP repair systems provide an economically viable alternative to traditional repair systems and materials. In this study experimental investigation on the flexural behavior of RC T-beams strengthened using glass fiber reinforced polymer (GFRP) sheets are carried out.

Reinforced concrete T beams externally bonded with GFRP sheets were tested to failure using a symmetrical two point static loading system. Seven RC T-beams were casted for this experimental test. All of them were weak in flexure and were having same reinforcement detailing. One beam was used as a control beam and six beams were strengthened using different configurations of glass fiber reinforced polymer (GFRP) sheets. Experimental data on load, deflection and failure modes of each of the beams were obtained. The effect of different amount and configuration of GFRP on ultimate load carrying capacity and failure mode of the beams were investigated.

The experimental results show that externally bonded GFRP can increase the flexural capacity of the beam significantly. In addition the results indicated that the most effective configuration was the U-wrap GFRP .A series of comparative studies on deflection between the present experimental data and results from finite element method and IS code method were made. Future area of research are being outlined.

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CHAPTER -1

INTRODUCTION

1.1 GENERAL

Deterioration in concrete structures is a major challenge faced by the infrastructure and bridge industries worldwide. The deterioration is mainly due to environmental effects, which includes corrosion of steel, gradual loss of strength with ageing, repeated high intensity loading, variation in temperature, freeze-thaw cycles, contact with chemicals and saline water and exposure to ultra-violet radiations. This problem, coupled with revisions in structural codes needed to account for the natural phenomena like earthquakes or environmental deteriorating forces, demands development of successful structural retrofit technologies. The structural retrofit problem has two options, repair/retrofit or demolition/reconstruction. Traditionally, the trend within the US construction industries has been towards the latter option. This solution has become increasingly unacceptable due to changing economic and social attitudes concerning existing structures. This fact leads to the necessity for development of appropriate structural retrofit/repair systems.

Traditionally, the retrofitting of reinforced concrete structures, such as columns, beams and other structural elements, involved a time consuming and disruptive process of removing and replacing the low quality or damaged concrete or/and steel reinforcements with new and stronger material. However, with the introduction of new advanced composite materials such as fiber reinforced polymer (FRP) composites, concrete members can now be easily and effectively strengthened using externally bonded FRP composites.

Retrofitting of concrete structures with wrapping FRP sheets provide a more economical and technically superior alternative to the traditional techniques in many situations because it offers high strength, low weight, corrosion resistance, high fatigue resistance, easy and rapid installation and minimal change in structural geometry. In addition, FRP manufacturing offers a unique opportunity for the development of shapes and forms that would be difficult or impossible with the conventional steel materials. Although the fibers and resins used in FRP systems are relatively expensive compared with traditional strengthening materials, labor and equipment costs to install FRP systems are often lower. FRP systems can also be used in areas with limited access where traditional techniques would be impractical.

However, the use of these materials for retrofitting the existing concrete structures cannot reach up to the expectation due to lack of the proper knowledge on structural behavior of concrete structures retrofitted by fiber reinforced polymers (FRP) composites. Successful retrofitting of concrete structures with FRP needs a thorough knowledge on the subject and available user-friendly technologies/ unique guidelines.

Beams are the critical structural members subjected to bending, torsion and shear in all type of structures. Similarly, columns are also used as various important elements subjected to axial load combined with/without bending and are used in all type of structures considering from building to bridge as piers or abutments. Therefore, extensive research works are being carried out throughout world on retrofitting of concrete beams and columns with externally bonded FRP composites. Several investigators took up concrete beams and columns retrofitted with carbon fiber reinforced polymer (CFRP)/ glass fiber reinforced polymer (GFRP) composites in order to study the enhancement of strength and ductility, durability, effect of confinement, preparation of design guidelines and experimental investigations of these members. The results obtained from different investigations regarding enhancement in basic parameters like strength/stiffness, ductility and durability of structural members retrofitted with externally bonded FRP composites, though quite encouraging, still suffers from many limitations. This needs further study in order to arrive at recognizing FRP composites as a potential full proof structural additive.

FRP repair is a simple way to increase both the strength and design life of a structure. Because of its high strength to weight ratio and resistance to corrosion, this repair method is ideal for deteriorated concrete structure due to exposure to de-icing salts and other environmental factors by encasing concrete members .FRP protects from existing salts and other environmental factors .It is noted that in many bridges the majority of corrosive damage occurred on exterior girders. This indicates that deleterious effects may be direct results of surface exposure, to spray of water, de-icing agents and environmental effects. Encasement of these girders not only increases design life, but also protects the members from surface attacks.FRP is a versatile material.

FRP can be applied to wide range of structures. FRP sheet can be cut and easily bonded to any concrete members. It is highly cost effect method of maintaining or upgrading existing structures. Quick application results in lower disruption and shorter contact periods. Reasons for strengthening of structures may include upgrading to accommodate higher loads (such as traffics), loss of pre-stress in existing reinforcement, or degradation of structures (e.g. corrosion of reinforcement).

The technique may allow continued usage of structures or facility during strengthening works. Higher material cost of Carbon /Glass fiber is outweighed by numerous advantages over steel such as low self-weight and less requirement for plate surface preparation. Glass or Aramid fibers offer lower cost alternative, in some instances, to carbon fibers.

FRP plates are an alternative to other forms of strengthening such as use of steel plates, or provision of additional support members. Column wrapping with FRP can be an alternative to jacketing additional reinforced concrete, or complete replacement of structures, with obvious saving in materials energy.

It increases the capacity with minimal addition of dead load to the structure. Materials are easy to transport and handle no lifting gear required. It is easy to use at height. It increases the ability to work in confined areas and in situations with difficult access (e.g. tunnel and basements). This technique is relatively quick with reduced disturbance and installation time.

1.2 ADVANTAGES AND DISADVANTAGES OF FRP

1.2.1 ADVANTAGES

FRP materials have higher ultimate strength and lower density as compared to steel. When these properties are taken together they lead to fiber composites having a strength/weight ratio higher than steel plate in some cases. The lower weight of FRP makes installation and handling significantly easier than steel. These properties are particularly important when installation is done in cramped locations. Other works like works on soffits of bridges and building floor slabs are carried out from man-access platforms rather than from full scaffolding. We all know that steel plate requires heavy lifting gear and are to be held in place while the adhesive gains its strength and bolts are fitted through the steel plate into the parent concrete to support the plate while the adhesive cures.

On the other hand, the application of FRP plate or sheet material is like applying wallpaper; once it has been rolled on carefully to remove entrapped air and excess adhesive it may be left unsupported. Here, no bolts are required; in fact, the use of bolts would seriously weaken the material unless additional cover plates are bonded on. Furthermore, because there is no need to drill into the structure to fix bolts or other mechanical anchors there is no risk of damaging the existing reinforcement. Fiber composite materials are available in very long lengths while steel plate is generally limited to 6 m. The availability of long lengths and the flexibility of the material also simplify installation:

• Laps and joints are not required

• The material can take up irregularities in the shape of the concrete surface

• The material can follow a curved profile; steel plate would have to be pre-bent to the required radius.

• The material can be readily installed behind existing services

• Overlapping, required when strengthening in two directions, is not a problem because the material is thin.

The materials fibers and resins are durable if correctly specified, and require little maintenance. If they are damaged in service, it is relatively simple to repair them, by adding an additional layer. The use of fiber composites does not significantly increase the weight of the structure or the dimensions of the member. The latter may be particularly important for bridges and other structures with limited headroom and for tunnels. In terms of environmental impact and sustainability, studies have shown that the energy required to produce FRP materials is less than that for conventional materials. Because of their light weight, the transport of FRP materials has minimal environmental impact.

These various factors in combination lead to a significantly simpler and quicker strengthening process than when using steel plate. This is particularly important for bridges because of the high costs of lane closures and possession times on major highways and railway lines. It has been estimated that about 90% of the market for plate strengthening in Switzerland has been taken by carbon plate systems as a result of these factors.

1.2.2 DISADVANTAGES

The main disadvantage of externally strengthening structures with fiber composite materials is the risk of fire, vandalism or accidental damage, unless the strengthening is protected. A particular concern for bridges over roads is the risk of soffit reinforcement being hit by over-height vehicles. However, strengthening using plates is generally provided to carry additional live load and the ability of the unstrengthened structure to carry its own self-weight is unimpaired. Damage to the plate strengthening material only reduces the overall factor of safety and is unlikely to lead to collapse. Experience of the long-term durability of fiber composites is not yet available. This may be a disadvantage for structures for which a very long design life is required but can be overcome by appropriate monitoring.

A perceived disadvantage of using FRP for strengthening is the relatively high cost of the materials. However, comparisons should be made on the basis of the complete strengthening exercise; in certain cases the costs can be less than that of steel plate bonding. A disadvantage in the eyes of many clients will be the lack of experience of the techniques and suitably qualified staff to carry out the work. Finally, a significant disadvantage is the lack of accepted design standards.

CHAPTER-2

REVIEW OF LITERATURE

2.1 BRIEF REVIEW

Development of FRP materials in various forms and configurations offers an alternative design approach for construction of new structures and rehabilitation of the existing structures. Research on FRP material for use in concrete structures began in Europe in the mid 1950's [Rubinsky and Rubinsky, 1954; Wines, J. C. et al., 1966]. The pioneering work of bonded FRP system can be credited to Meier [Meier 1987]; this work led to the first on-site repair by bonded FRP in Switzerland [Meier and Kaiser 1991].Japan developed its first FRP applications for repair of concrete chimneys in the early 1980s [ACI 440 1996].By 1997 more than 1500 concrete structures worldwide had been strengthened with externally bonded FRP materials. Thereafter, many FRP materials with different types of fibres have been developed. FRP products can take the form of bars, cables, 2-D and 3-D grids, sheet materials and laminates.

Though the research and application of retrofitting/strengthening of concrete structures using externally bonded FRP composites has started recently, i.e., about few decades ago, it is rapidly gaining the momentum in attracting the research communities and construction industries. The Naval Facilities Engineering Service Center (NFESC) started a project in the early 1990's to explore the use of composite materials for repairing or strengthening waterfront structures under sponsorship by the office of Naval Research (Odello, 2001). During that time the Navy teamed with the Army Corps of Engineers, presently known as the Market Development Alliance (MDA) of the FRP Composites Industry on a project for the Construction of Productive Advancement Research (CPAR) program. The program involved testing of fender piles fabricated with FRP materials. The piles having FRP casing around a concrete core showed very good stiffness and strength properties under bending loads (Hoy et al., 1996; Hoy, 1996). Research investigations that deal with the strengthening of structures using carbon fibre reinforced polymer (CFRP) sheets/strips/plates are conducted by many investigators (Saadtmanesh and Ehsani, 1990 & 1991; Meier et al., 1992; Chajes et al., 1994;

Grace et al., 1999). In these investigations, they are primarily interested to examine the increase in load carrying capacity of the flexural members.

Nanni et al. (1997) made analytical & experimental study of the retrofitting effect of CFRP sheets applied on the tension side & web of the damaged R.C. beams due to loading beyond cracking strength of concrete. Besides, the effect of CFRP sheets on stiffening effect is studied with various orientations of fibres with respect to beam axis. Results showed a substantial enhancement in the strength of the retrofitted damaged beams varying from 20% to 60%. Malek et al. (1998) also carried out an experimental investigation to predict the failure load of RC beams strengthened with FRP plates. Their experimental study showed local failure of the concrete cover along the longitudinal reinforcement in the retrofitted beams due to stress concentration of the plate end. Grace et al. (1999) investigated the behaviour of RC beams strengthened with CFRP and GFRP sheets and laminates. They studied the influence of the number of layers, epoxy types, and strengthening pattern on the response of the beams. It is observed that all beams experienced brittle failure, with appreciable enhancement in strength, thus requiring a higher factor of safety in design.

Experimental investigations, theoretical calculations and numerical simulations showed that strengthening the reinforced concrete beams with externally bonded CFRP sheets in the tension zone considerably increased the strength at bending, reduced deflections as well as cracks width (Ross et al., 1999; Sebastian, 2001; Smith & Teng, 2002; Yang et al., 2003; Aiello & Ombres, 2004). It also changed the behaviour of these beams under load and failure pattern. Most often the strengthened beams failed in a brittle way, mainly due to the loss of connection between the composite material and the concrete. Chajes et al. (1996) conducted an experimental study to understand the nature of the bond between the composite plates and concrete.

The influence of the surface preparation of the concrete, adhesive type, and concrete strength on the overall bond strength is studied as well as characteristics of force transfer from the plate to concrete. They concluded that the surface preparation along with along with soundness of concrete could influence the ultimate bond strength. Thereafter, Study on de-bonding problems in concrete beams externally strengthened with FRP composites are carried out by many researchers.

Many investigators used externally bonded FRP composites to improve the flexural strength of reinforced concrete members. To evaluate the flexural performance of the strengthened members, it is necessary to study flexural stiffness of FRP strengthened members at different stages, such as pre-cracking, post-cracking and post-yielding. However, only few studied are focused on the reinforced concrete members strengthened under pre-loading or pre-cracking (Arduni & Nanni, 1997).

Several investigators (Saadatmanesh et al., 1994; Shahawy, 2000) took up FRP strengthened circular or rectangular columns studying enhancement of strength and ductility, durability, effect of confinement, preparation of design guidelines and experimental investigations of these columns. Saadatmanesh et al. (1994) studied the strength and ductility of concrete columns externally reinforced with fibre composite strap. Chaallal and Shahawy (2000) reported the experimental investigation of fiber reinforced polymer-wrapped reinforced concrete column under combined axial-flexural loading.

Martinez et al (2008) described a procedure, based on a finite element formulation that can be used to perform numerical simulations of RC structures reinforced with FRP. Rajamohan et al (2009) studied the effect of inclined GFRP strips epoxy bonded to the beam web for shear strengthening of reinforced concrete beams. He also studied the effectiveness in terms of width and spacing of inclined GFRP strips, spacing of internal steel stirrups, and longitudinal steel rebar section on shear capacity of the RC beam study to investigate the behaviour of structurally damaged full-scale reinforced concrete beams retrofitted with CFRP laminates in shear or in flexure. Obaidat et al (2010) studied the Retrofitting of reinforced concrete beams using composite laminates and the main variables considered are the internal reinforcement ratio, position of retrofitting and the length of CFRP.

The experimental results, indicated that beams retrofitted in shear and flexure by using CFRP laminates are structurally efficient and are restored to stiffness and strength values nearly equal to or greater than those of the control beams Tanarslan et al (2009) analyzed the behavior and failure modes of T-section RC beams strengthened in shear with externally bonded CFRP strips and the test results confirmed that all CFRP arrangements differ from CFRP strip width and arrangement, improved the strength and behavior of the specimens in different level significantly. Nanni et al (2000) studied the shear performance of reinforced concrete (RC) beams with T-section. Different configurations of externally bonded carbon fiber-reinforced polymer (CFRP) sheets are used to strengthen the specimens in shear. the result showed that externally bonded CFRP can increase the shear capacity of the beam significantly. In addition it indicated that the most effective configuration is the U-wrap with end anchorage.

Though several researchers have been engaged in the investigation of the strengthened concrete structures with externally bonded FRP sheets/laminates/fabrics, no country yet has national design code on design guidelines for the concrete structures retrofitted using FRP composites. However, several national guidelines (The Concrete Society, UK:2004; ACI 440:2002; FIB:2001; ISIS Canada:2001; JBDPA:1999) offer the state of the art in selection of FRP systems and design and detailing of structures incorporating FRP reinforcement. On the contrary, there exists a divergence of opinion about certain aspects of the design and detailing guidelines. This is to be expected as the use of the relatively new material develops worldwide. Much research is being carried out at institutions around the world and it is expected that design criteria will continue to be enhanced as the results of this research become know in the coming years.

Structural adhesives are generally accepted to be monomer composites which polymerize to give fairly stiff and strong adhesive uniting relatively rigid adherents to form a load-bearing joint (Shields, 1985). The feasibility of bonding concrete with epoxy resins is first demonstrated in the late 1940s (ACI, 1973), and the early development of structural adhesives is recorded by Fleming and King (1967). Since the early 1950s adhesives have become widely used in civil engineering (Mays, 1985). However, although the building and construction industries represent some of the largest users of adhesive materials, many applications are non-structural in the sense that the bonded assemblies are not used to transmit or sustain significant stresses (e.g. crack injection and sealing, skid-resistant layers, bonding new concrete to old). Truly structural application implies that the adhesive is used to provide a shear connection between similar or dissimilar materials, enabling the components being bonded to act as a composite structural unit.

A comprehensive review of applications involving the use of adhesives in civil engineering is given by Hewlett and Shaw (1977), Tabor (1982) and Mays and Hutchinson (1992). Assessment of an adhesive as a suitable product for structural use must take into account the design spectrum of loads, the strength and stiffness of the material under short term, sustained or cyclic loads and the effect on these properties of temperature, moisture and other environmental conditions during service (Mays, 1993). Concern regarding the durability properties of adhesive joints has meant that resistance to creep, fatigue and fracture are considered of greater importance than particularly high strength (Vardy and Hutchinson, 1986). Temperature is important at all stages in the use and performance of adhesives, affecting viscosity and therefore workability, usable life and contact time, rate of cure, degree of cross-linking and final cured performance (Tu and Kruger, 1996).

Controlled conditions are therefore generally required during bonding. This applies equally during the surface treatment procedures if a durable system is to be achieved. Adhesives, which are workable and cure at ambient temperatures, have been used and are able to tolerate a certain amount of moisture without a marked reduction in performance. These must have adequate usable time under site conditions and a cure rate which does not hinder the construction program. Workmanship under conditions prevalent on site is less conducive to quality control than in other industries, and thus ability to tolerate minor variations in proportioning and mixing, as well as imperfect surface treatment, is important. In addition, the products involved are more toxic, require more careful storage and, bulk for bulk, are considerably more expensive than traditional construction materials. Nondestructive test methods for assessing the integrity of bonded joints are now available for civil engineering applications. Despite some drawbacks, structural adhesives have enormous potential in future construction applications, particularly where the combination of thick bondlines, ambient temperature curing and the need to unite dissimilar materials with a relatively high strength joint are important (Mays and Hutchinson, 1992).

The principal structural adhesives specifically formulated for use in the construction Industry are epoxy and unsaturated polyester resin systems, both thermosetting polymers. The formulation of adhesives is considered in detail by Wake (1982), whilst Tabor (1978) offers guidance on the effective use of epoxy and polyester resins for civil engineering structures. Two-part epoxies, first developed in the 1940s (Lee and Neville, 1967), consist of a resin, a hardener or cross-linking agent which causes polymerization, and various additives such as fillers, tougheners or flexibilisers, all of which contribute to the physical and mechanical properties of the resulting adhesive. Formulations can be varied to allow curing at ambient temperature, the so-called cold cure epoxies, the most common hardeners for which are aliphatic polyamines, whose use results in hardened adhesives which are rigid and provide good resistance to chemicals, solvents and water (Mays and Hutchinson, 1992). Correct proportioning and thorough mixing are imperative when using epoxy resin systems. The rate of curing doubles as the temperature increases by 10°C and halves as the temperature drops by 10 °C and many of the formulations stop curing altogether below a temperature of 5 °C. Fillers, generally inert materials such as sand or silica, may be used to reduce cost, creep and shrinkage, reduce exotherm and the coefficient of thermal expansion, and assist corrosion inhibition and fire retardation. Fillers increase the viscosity of the freshly mixed system but impart thixotropy, which is useful in application to vertical surfaces.

Unmodified epoxy systems tend to be brittle when cleavage or peel forces are imposed. Toughening of the cured adhesive can be achieved by the inclusion of a dispersed rubbery phase which absorbs energy and prevents crack propagation. Epoxies are generally tolerant of many surface and environmental conditions and possess relatively high strength. They are preferred for bonding to concrete since, of all adhesives, they have a particularly high tolerance of the alkalinity of concrete, as well as moisture. By suitable formulation, their ability to wet out the substrate surfaces can even be achieved in the presence of water, the resin being able to disperse the water from the surface being bonded (Tabor, 1978). Unsaturated polyester resins are discovered in the mid-1930s and have adhesive properties obtained by cross-linking using a curing agent. They are chemically much more simple than epoxy resins but have a 10% contraction by volume during curing due to a volume change during the transition from the uncured liquid phase to the hardened resin resulting in further curing shrinkage. As a result of these factors, there are usually strict limits on the volume of material that can be mixed and applied at any one time and as a general rule polyester resins do not form as strong adhesive bonds as do epoxy resins. In storage, the polyester resins are also somewhat less stable and present a greater fire hazard than epoxies. These limitations significantly restrict their applications.

The advantages of epoxy resins over other polymers as adhesive agents for civil Engineering use can be summarized as follows (Mays and Hutchinson, 1992):

- High surface activity and good wetting properties for a variety of substrates.
- May be formulated to have a long open time (the time between mixing and closing of the joint).
- High cured cohesive strength, so the joint failure may be dictated by the adherend strength, particularly with concrete substrates.
- May be toughened by the inclusion of a dispersed rubbery phase.
- Minimal shrinkage on curing, reducing bond line strain and allowing the bonding of large areas with only contact pressure.

- Low creep and superior strength retention under sustained load. Can be thixotropic for application to vertical surfaces.
- Able to accommodate irregular or thick bond lines.
- Formulation can be readily modified by blending with a variety of materials to achieve desirable properties.

These various modifications make epoxy adhesives relatively expensive in comparison to other adhesives. However, the toughness, range of viscosity and curing conditions, good handling characteristics, high adhesive strength, inertness, low shrinkage and resistance to chemicals have meant that epoxy adhesives have found many applications in construction, for example, repair materials, coatings and as structural and non-structural adhesives.

There are many features of an adhesive product, in addition to its purely adhesive properties, which will form the basis for the selection of a particular bonding system. Mays (1985) has considered requirements for adhesives to be used for external plate bonding to bridges under conditions prevalent in the UK. These requirements are extended and refined in a later publication referred to as a proposed Compliance Spectrum (Mays and Hutchinson, 1988), which addresses the general engineering requirements of adhesives, bonding procedures and test methods for structural steel-to-concrete bonding, based on research work at the University of Dundee (Hutchinson, 1986). The requirements proposed for the adhesive itself can be considered to be equally applicable to fiber reinforced polymer (FRP) plate bonding. An epoxy resin and polyamine hardener is recommended. Choice of a suitable adhesive is only one of a number of requirements for a successfully bonded joint. Other factors also affect the joint strength and performance (Maysand Hutchinson, 1988) namely:

- appropriate design of the joint
- adequate preparation of the adherend surfaces
- controlled fabrication of the joint
- protection from unacceptably hostile conditions in service
- Post bonding quality assurance.

Both short term and long term structural performance are likely to be improved by using an appropriately designed joint and suitably preparing the surface of the substrate materials. A review of factors important to the satisfactory design of joints is given by Adams and Wake (1984) and Lees (1985) and will not be considered here. Full account must be taken of the poor resistance of adhesives to peel and cleavage forces; shear strength itself is unlikely to be a limiting factor. With concrete structures, the tensile/shear, or tear-off strength of the concrete should be the critical design factor if a suitable adhesive formulation is selected and appropriate methods of surface preparation implemented. This has been demonstrated through detailed shear testing on site and in the laboratory (Moustafa, 1974; Hugenschmidt, 1975; Schultz, 1976). A number of tests are available for testing adhesive and thin films (Adams and Wake, 1984; Kinloch, 1987). However, appropriate tests for assessing bond strength in construction are complicated by the fact that the loading condition in service is difficult to simulate, and one of the adherends, namely concrete, tends to be weaker in tension and shear than the adhesives which may be used, making discrimination between adhesive systems difficult. As a result, confirmation of the suitability of a proposed adhesive system is generally limited to demonstrating that, when the bond line is stressed in the test configuration chosen, the failure surface occurs within the concrete substrate. Such tests may also be used to exhibit the adequacy of the surface preparation techniques employed, since it is difficult to separate the individual effects on adhesion of the adhesive type and method of surface treatment

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Several possible test methods have evolved to measure the bond strength between adhesive and concrete substrates, mainly for applications in concrete repair (Franke, 1986; Naderi et al., 1986). The Réunion Internationale des Laboratoires d'Essais et de Recherches sur les Matériaux et les Constructions (RILEM) Technical Committee 52-RAC lists some currently used laboratory and field test methods for assessing the bond between resin and concrete (Sasse and Friebrich, 1983). Procedures are mentioned on the strength of adhesion in tension, shear and bending, as well as shrinkage and thermal compatibility in the context of coatings, concrete repair, concrete/ concrete and steel/concrete bonds. Variations of the slant shear test (Kreigh, 1976), in which two portions of a standard cylinder or prism are joined by a diagonal bond line and then tested in compression, have been found to produce discriminating and consistent results (Kreigh, 1976; Naderi, 1985; Wall et al., 1986). Tu and Kruger (1996) used such a configuration to demonstrate that a flexible, tough epoxy provided improved adhesion compared to a more brittle material because it allows redistribution of forces before fracture. However, Tabor (1985) concluded that the slant shear test is of little use in assessing adhesion between resin and concrete because the interfaces are not subjected to tensile forces.

In assessing the shear connection in steel/concrete composite construction, tests at the Wolfson Bridge Research Unit at the University of Dundee employed a kind of doublelap joint configuration as described by Solomon (1976), in which fracture is characterized by shear failure of the concrete adjacent to the interface with the adhesive. The University of

Surrey (Quantrill et al., 1995) have reported a programme of small scale tests to investigate three different adhesives, two of which are two-part cold cure epoxies and the third a two-part acrylic. The tests involved subjecting an adhesive/concrete joint to tensile force and a composite/adhesive/concrete joint to shear, to verify the adequacy of the surface preparation of the concrete and composite bond surfaces. In these tests the Sikadur 31 PBA epoxy adhesive is superior to the two other products and demonstrated strengths in both tension and shear which exceeded those of the concrete. The acrylic adhesive failed within the adhesive under very small ultimate loads. Chajes et al. (1996) used a single-lap specimen, in which a strip of carbon composite is bonded to a concrete prism, to study the bond strength of composite plate materials bonded to concrete. Four different adhesives are used to bond the composite strip; three two-part cold cure structural epoxies and a two-part cold cure urethane.

Three methods of surface preparation are studied, varying in severity from untreated to mechanically abraded to expose the coarse aggregate. It is found that all epoxy-bonded joints failed as a result of the concrete shearing directly beneath the bond surface at similar loads. The final strength is therefore a function of the concrete strength. The surface treatment which involved exposing the coarse aggregate produced the highest average strengths. The urethane adhesive, which is much less stiff and had a much higher ductility to failure in tension than the epoxies, failed within the adhesive at lower ultimate loads. It is of interest to note that a silane surface primer is used on two of Chajes' adhesives (the primer used is Chemglaze 9926) and it improved the bond performance of the joints compared with a jointnot treated thus; when used on concrete the primer tends to improve the bond by strengthening the surface of the concrete and making it water repellent. Karbhari and Engineer (1996) describe the use of a modified peel test for investigation of the bond between composite and concrete, in which a composite strip is pulled away from the concrete at a known angle and at a controlled rate. The test is said to provide a good estimate of interfacial energy and could be used in durability assessment.

2.2 CRITICAL OBSERVATION FROM THE LITERATURE:

From the above information, it is, thus, clear that there lies a vast scope of research in the field of retrofitting of concrete structures especially T-Section Beam using externally bonded FRP composites. In the above section it has been shown how the structural strength and stiffness can be improved by externally bonded material. The worldwide interest in the technique reflects its potential benefits and also the current importance placed on economical rehabilitation and upgrading methods. Although the level of experience in the bonding technique of composite plates is limited, the investigations reported in this chapter have gone some way to illustrate its potential and to establish a basic technical understanding of short term and long term behavior. Despite the growing number of field applications, there is limited number of reports on flexural behavior of strengthened RC T-beams using externally bonded FRP composites. The objective of the present work is to determine the effect of retrofit on the behavior of R.C T-Beam under static loading and also finite element model is developed to analyze the structure and compare the numerical results with experimental values.

2.3 OBJECTIVE AND SCOPE OF THE PRESENT WORK

The objectives of this work is to carry out the experimental investigation of externally bonded R.C. T- Beams using GFRP sheets and study the enhancement of the strength .And compare its results numerically to know the suitability of the FRP composites as retrofit materials for deteriorated R.C.Structures.

In the present work the behavior of T-section reinforced concrete beams, retrofitted with GFRP is observed to know the practical feasibility in the construction industry. Seven number of T-section concrete beams are casted. All these beams except one beam are bonded with GFRP sheets using epoxy in different size and layers. These beams are subjected to flexure by applying two points loading to evaluate the excess of flexural strength due to retrofitting of GFRP. And the results are validated numerically.

CHAPTER-3

EXPERIMENTAL STUDY

3.1 MATERIALS

3.1.1 CONCRETE

It is composed of Portland cement and water combined with sand, gravel, crushed stone, or other inert material such as expanded slag or vermiculite. A strong stone-like mass is formed from a chemical reaction of cement and water. The concrete paste is plastic and can be easily molded into any form or trowelled to produce a smooth surface. Hardening starts immediately after mixing, but precautions are taken, usually by covering, to avoid rapid loss of moisture since the presence of water is necessary to continue the chemical reaction and increase the strength. Excess of water, however, produces a concrete that is more porous and weaker. The quality of the paste formed by the cement and water largely determines the character of the concrete. Proportioning of the ingredients of concrete is referred to as designing the mixture, and for most structural work the concrete is designed to give compressive strengths of 15 to 35 MPa. Concrete may be produced as a dense mass which is practically artificial rock, and chemicals may be added to make it waterproof, or it can be made porous and highly permeable for such use as filter beds. An air-entraining chemical may be added to produce minute bubbles for porosity or light weight. Normally, the full hardening period of concrete is at least 7 days. The gradual increase in strength is due to the hydration of the tricalcium aluminates and silicates. Sand used in concrete is originally specified as roughly angular, but rounded grains are now preferred. The stone is usually sharply broken. Concrete is stronger in compression than in tension, and steel bar, called rebar or mesh is embedded in structural members to increase the tensile and flexural strengths. In addition to the structural uses, concrete is widely used in precast units such as block, tile, sewer, and water pipe, and ornamental products.

3.1.2 CEMENT

Cement is a material, generally in powder form, that can be made into a paste usually by the addition of water and, when molded or poured, will set into a solid mass. Numerous organic compounds used for adhering, or fastening materials, are called cements, but these are classified as adhesives, and the term cement alone means a construction material. The most widely used of the construction cements is Portland cement. It is a bluish-gray powder obtained by finely grinding the clinker made by strongly heating an intimate mixture of calcareous and argillaceous minerals. The chief raw material is a mixture of high-calcium limestone, known as cement rock, and clay or shale. Blast-furnace slag may also be used in some cements and the cement is called Portland slag cement (PSC). The color of the cement is due chiefly to iron oxide. In the absence of impurities, the color would be white, but neither the color nor the specific gravity is a test of quality.

3.1.3 FINE AGGREGATE

Fine aggregate / sand is an accumulation of grains of mineral matter derived from the disintegration of rocks. It is distinguished from gravel only by the size of the grains or particles, but is distinct from clays which contain organic materials. Sands that have been sorted out and separated from the organic material by the action of currents of water or by winds across arid lands are generally quite uniform in size of grains. Usually commercial sand is obtained from river beds or from sand dunes originally formed by the action of winds. Much of the earth's surface is sandy, and these sands are usually quartz and other siliceous materials. The most useful commercially are silica sands, often above 98% pure. Beach sands usually have smooth, spherical to ovaloid particles from the abrasive action of waves and tides and are free of organic matter. The white beach sands are largely silica but may also be of zircon, monazite, garnet, and other minerals, and are used for extracting various elements.

Sand is used for making mortar and concrete and for polishing and sandblasting. Sands containing a little clay are used for making molds in foundries. Clear sands are employed for filtering water. Sand is sold by the cubic yard (0.76 m3) or ton (0.91 metric ton) but is always shipped by weight. The weight varies from 1,538 to 1,842 kg/m3, depending on the composition and size of grain. Construction sand is not shipped great distances, and the quality of sands used for this purpose varies according to local supply. Standard sand is silica sand used in making concrete and cement tests. The fine aggregate is passing through 4.75 mm sieve and had a specific gravity of 2.67. The grading zone of fine aggregate is zone III as per Indian Standard specifications.

3.1.4 COARSE AGGREGATE

Coarse aggregate are the crushed stone is used for making concrete. The commercial stone is quarried, crushed, and graded. Much of the crushed stone used is granite, limestone, and trap rock. The last is a term used to designate basalt, gabbro, diorite, and other dark colored, fine-grained igneous rocks. Graded crushed stone usually consists of only one kind of rock and is broken with sharp edges. The sizes are from 0.25 to 2.5 in (0.64 to 6.35 cm), although lager sizes may be used for massive concrete aggregate. Machine chorused granite broken stone angular in shape is use as coarse aggregate.

3.1.5 MATERIAL TESTING OF CONCRETE

The testing of the ingredient materials of concrete such as cement, fine aggregate and coarse aggregate are carried out and results are presented below.

3.1.5.1 TESTING OF CEMENT

Type: Konark Portland Slag Cement.

(i) Specific gravity	:	2.96
(ii) Normal Consistency	:	32%
(iii) Setting Times	:	Initial : 105 minutes
		Final : 535 minutes.
(iv) Soundness	:	2 mm expansion
(v) Fineness	:	1 gm retained in 90 micron sieve

3.1.5.2 TESTING OF FINE AGGREGATE

(i) Sieve Analysis

The results of sieve analysis for fine aggregate are furnished in table 3.1

Grading Zone = III

(ii) Specific Gravity : 2.67

(iii) Water Absorption : .8%

Table 3.1: Results of Sieve Analysis for Fine Aggregate

Sl. No.	Sieve size (in mm)	Mass retained (in gm)	Mass passing (in gm)	% passing	Remarks
1.	4.75	20	1980	99	90-100
2.	2.36	44	1936	96.8	85-100
3.	1.18	132	1804	90.2	75-100
4.	600µ	346	1458	72.9	60-79
5.	300µ	1216	242	12.1	12-40
6.	150µ	202	40	2	0-10
7.	Pan	40	0	0	-

Given Sand Confirms To Zone - III

3.1.5.3 TESTING OF COARSE AGGREGATE

(i) Sieve Analysis

The results of sieve analysis of coarse aggregate are furnished in table 4.2

(ii) Specific gravity = 2.72

(iii) Absorption value = 0.5%

Table 3.2: Results of Sieve Analysis for Coarse Aggregate (20mm)

Sl.No.	Sieve size (in mm)	Mass retained (in gm)	Mass passing (in gm)	% passing	Remarks
1	80	0	5000	100	-
2	40	0	5000	100	-
3	20	436	4564	91.28	95-100
4	10	3027	1537	30.74	25-55
5	4.75	1478	59	1.18	0-10
6	Pan	59	0	0	0

Table 3.3: Results of Sieve Analysis for Coarse Aggregate (10mm)

Sl.No.	Sieve size (in mm)	Mass retained (in gm)	Mass passing (in gm)	% passing	Remarks
1	80	0	5000	100	-
2	40	0	5000	100	-
3	20	39	4961	99.22	-
4	10	992	3969	79.38	85-100
5	4.75	3828	141	2.82	0-20
6	Pan	141	0	0	-

3.1.5.4 MIX DESIGN OF M20 GRADE CONCRETE

1. **Design Stipulations:-**
   a) Characteristics strength = 20N/mm^2
   b) Degree of quality control = Good
   c) Degree of exposure = Mild
   d) Workability = 62

2. **Materials Supplied:-**
   a) Cement : Konark Portland Slag Cement
   b) Course aggregate : 20mm down
   c) Fine aggregate : Sand conforming to grading zone III

Design Mix Proportions:-

Table 3.4: Results of Sieve Analysis for Coarse Aggregate (10mm)

Description	**Cement**	**Sand (Fine Aggregate)**	**Course Aggregate**	**Water**
Mix proportion (by weight)	1	1.56	3.30	0.5
Quantities of materials (in Kg/m^3)	372	580	1228	186

3.1.6 REINFORCEMENT

High-Yield Strength Deformed bars of 20 mm diameter are used for the longitudinal reinforcement and 8 mm diameter high-yield strength deformed bars are used as stirrups. The yield strength of steel reinforcements used in this experimental program is determined by performing the standard tensile test on the three specimens of each bar. The average proof stress at 0.2 % strain of 20 mm φ bars is 378 N/mm^2.

3.1.6.1 DETAILING OF REINFORCEMENT IN R.C. T- BEAMS

For all the seven reinforced concrete T beams, the same arrangement for flexure and shear reinforcement is made. The tension reinforcement consists of 2 nos of 20 mm diameter HYSD bar. Four bars of 8 mm of HYSD bars are also provided as hang up bars .The shear reinforcement consists of 8 mm diameter 2-legged vertical stirrups of HYSD bars @100mm c/c. The detailing of reinforcement of the beam is shown in Fig 3.1

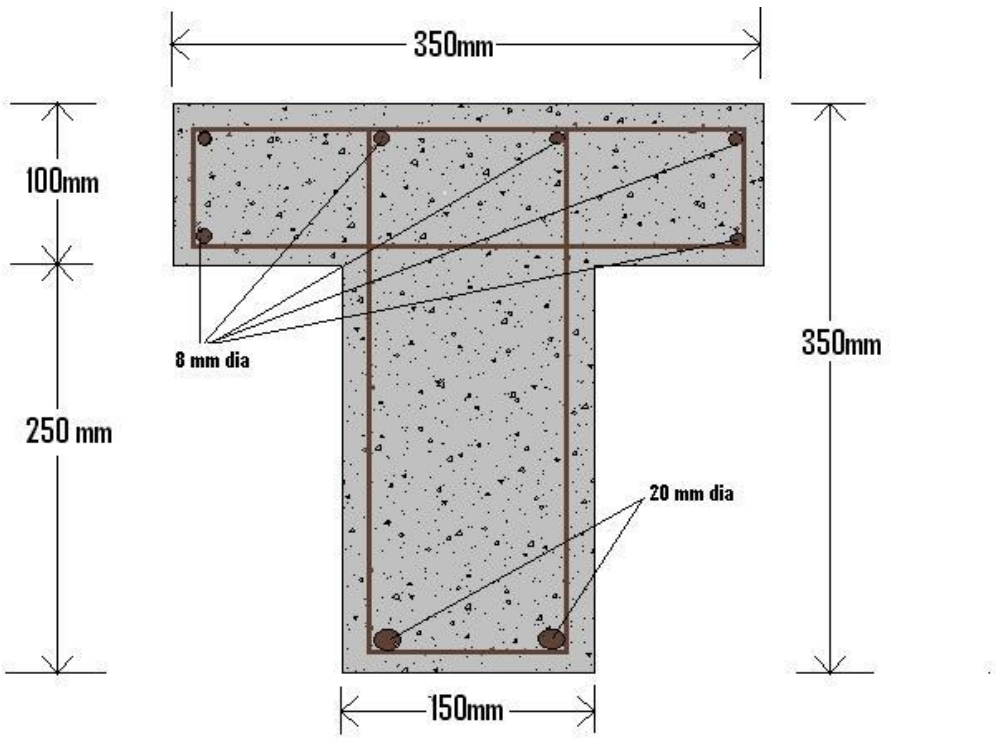


Fig:3.1 Reinforcement Detailing of T- Beam

Fig:3.2 Front View of Reinforcement Detailing of T- Beam

Fig:3.3 3D View of Reinforcement Detailing of T- Beam

3.1.7 FIBER REINFORCED POLYMER (FRP)

Continuous fiber-reinforced materials with polymeric matrix (FRP) can be considered as composite, heterogeneous, and anisotropic materials with a prevalent linear elastic behavior up to failure. They are widely used for strengthening of civil structures. There are many advantages of using FRPs: lightweight, good mechanical properties, corrosion-resistant, etc. Composites for structural strengthening are available in several geometries from laminates used for strengthening of members with regular surface to bidirectional fabrics easily adaptable to the shape of the member to be strengthened. Composites are also suitable for applications where the aesthetic of the original structures needs to be preserved (buildings of historic or artistic interest) or where strengthening with traditional techniques cannot be effectively employed.

Fiber reinforced polymer (FRP) is a composite material made by combining two or more materials to give a new combination of properties. However, FRP is different from other composites in that its constituent materials are different at the molecular level and are mechanically separable. The mechanical and physical properties of FRP are controlled by its constituent properties and by structural configurations at micro level. Therefore, the design and analysis of any FRP structural member requires a good knowledge of the material properties, which are dependent on the manufacturing process and the properties of constituent materials. FRP composite is a two phased material, hence its anisotropic properties. It is composed of fiber and matrix, which are bonded at interface. Each of these different phases has to perform its required function based on mechanical properties, so that the composite system performs satisfactorily as a whole. In this case, the reinforcing fiber provides FRP composite with strength and stiffness, while the matrix gives rigidity and environmental protection.

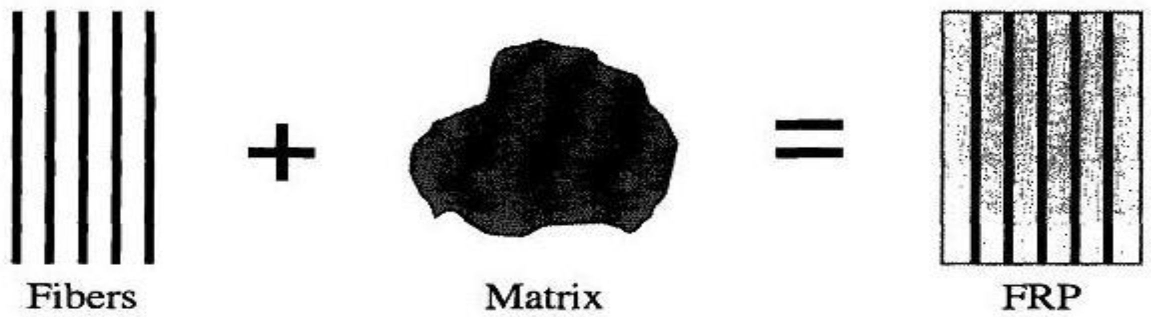


Fig. 3.4 Formation of Fiber Reinforced Polymer Composite

Reinforcement materials

A great majority of materials are stronger ad stiffer in fibrous form than as bulk materials. A high fiber aspect ratio (length: diameter ratio) permits very effective transfer of load via matrix materials to the fibers, thus taking advantage of there excellent properties. Therefore, fibers are very effective and attractive reinforcement materials.

3.1.7.1 FIBER

A fiber is a material made into a long filament with a diameter generally in the order of 10 tm. The aspect ratio of length and diameter can be ranging from thousand to infinity in continuous fibers. The main functions of the fibers are to carry the load and provide stiffness, strength, thermal stability, and other structural properties in the FRP. To perform these desirable functions, the fibers in FRP composite must have:

i) high modulus of elasticity for use as reinforcement;

ii) high ultimate strength;

iii) low variation of strength among fibers;

iv) high stability of their strength during handling; and

v) high uniformity of diameter and surface dimension among fibers.

There are three types of fiber dominating in civil engineering industry-glass, carbon and aramid fibers, each of which has its own advantages and disadvantages.

Material	Density (g/cm^3)	Tensile Modulus (E) (GPa)	Tensile Strength (σ) (GPa)	Specific Modulus (E/σ)	Specific Strength	Relative Cost
E-glass	2.54	70	3.45	27	1.35	Low
S-glass	2.50	86	4.50	34.5	1.8	Moderate
Graphite, high modulus	1.9	400	1.8	200	0.9	High
Graphite, high strength	1.7	240	2.6	140	1.5	High
Boron	2.6	400	3.5	155	1.3	High
Kevlar 29	1.45	80	2.8	55.5	1.9	Moderate
Kevlar 49	1.45	130	2.8	89.5	1.9	Moderate

Table 3.5 Properties of different fibers

Types of fibers used in fiber reinforced polymer composites

- Glass fibers
- Carbon fibers
- Aramid fibers

Glass fibers

These are fibers commonly used in the naval and industrial fields to produce composites of medium-high performance. Their peculiar characteristic is their high strength. Glass is mainly made of silicon (SiO_2) with a tetrahedral structure (SiO_4). Some aluminium oxides and other metallic ions are then added in various proportions to either ease the working operations or modify some properties (e.g., S-glass fibers exhibit a higher tensile strength than E-glass).

	E-glass	S-glass
Silicon oxide	54.3	64.20
Aluminium oxide	15.2	24.80
Iron oxide	-	0.21
Calcium oxide	17.2	0.01
Magnesium oxide	4.7	10.27
Sodium oxide	0.6	0.27
Boron oxide	8.0	0.01
Barium oxide	-	0.20
Various	-	0.03

Table 3.6 Typical composition of fiberglass (% in weight)

The production technology of fiberglass is essentially based on spinning a batch made of sand, alumina, and limestone. The constituents are dry mixed and brought to melting (about 1260 °C) in a tank. The melted glass is carried directly on platinum bushings and, by gravity, passes through ad hoc holes located on the bottom. The filaments are then grouped to form a strand typically made of 204 filaments. The single filament has an average diameter of 10 µm and is typically covered with a sizing. Glass fibers are also available as thin sheets, called mats. A mat may be made of both long continuous and short fibers (e.g., discontinuous fibers with a typical length between 25 and 50 mm), randomly arranged and kept together by a chemical bond. The width of such mats is variable between 5 cm and 2 m, their density being roughly 0.5 kg/m^2. Glass fibers typically have a Young modulus of elasticity (70 GPa for E-glass) lower than carbon or aramid fibers and their abrasion resistance is relatively poor; therefore, caution in their manipulation is required.

In addition, they are prone to creep and have low fatigue strength. To enhance the bond between fibers and matrix, as well as to protect the fibers itself against alkaline agents and moisture, fibers undergo sizing treatments acting as coupling agents. Such treatments are useful to enhance durability and fatigue performance (static and dynamic) of the composite material. FRP composites based on fiberglass are usually denoted as GFRP.

3.2 EXPERIMENTAL STUDY

The experimental study consists of casting of seven reinforced concrete T beams. All the seven beams weak in flexure are casted, out of which one is taken as controlled beam and other six beams are strengthened using continuous glass fiber reinforced polymer (GFRP) sheets in flexure. The strengthening of the beams is done with varying configuration and layers of GFRP sheets. Experimental data on load, deflection and failure modes of each of the beams are obtained. The change in load carrying capacity and failure mode of the beams are investigated as the amount and configuration of GFRP sheets are altered. The following chapter describes in detail the experimental study.

3.3 CASTING OF SPECIMEN

For conducting experiment, seven reinforced concrete T beam specimen of size as shown in the fig (Length = 2m , flange width = 0.35m , web width = 0.15 m, depth of the flange = .10m,overall depth=.35m) and all having the same reinforcement detailing are casted. The proportion of **0.5: 1: 1.56: 3.30** for water, cement, fine aggregate and course aggregate is taken. The mixing is done by using concrete mixture. The beams is cured for 28 days. For each beam three cubes are casted to determine the compressive strength of concrete for 28 days.

3.3.1 MATERIALS FOR CASTING

3.3.1.1 CEMENT

Portland slag cement (PSC) (Konark Cement) is used for the experiment. It is tested for its physical properties in accordance with Indian Standard specifications. It is having a specific gravity of 2.96.

3.3.1.2 FINE AGGREGATE

The fine aggregate passing through 4.75 mm sieve and having a specific gravity of 2.67 are used. The grading zone of fine aggregate is zone III as per Indian Standard specifications.

3.3.1.3 COARSE AGGREGATE

The coarse aggregates of two grades are used one retained on 10 mm size sieve and another grade contained aggregates retained on 20 mm sieve. It is having a specific gravity of 2.72

3.3.1.4 WATER

Ordinary tap water is used for concrete mixing in all the mix.

3.3.1.5 REINFORCING STEEL

HYSD bars of 20 mm φ are used as main reinforcement. 8 mm φ HYSD steel bars are used for shear reinforcement.

3.3.2 FORM WORK

Fig. 3.5 Steel Frame Used For Casting of T-Beam

3.3.3 MIXING OF CONCRETE

Mixing of concrete is done thoroughly with the help of machine mixer so that a uniform quality of concrete is obtained.

3.3.4 COMPACTION

Compaction is done with the help of needle vibrator in all the specimens .And care is taken to avoid displacement of the reinforcement cage inside the form work. Then the surface of the concrete is leveled and smoothened by metal trowel and wooden float.

3.3.5 CURING OF CONCRETE

Curing is done to prevent the loss of water which is essential for the process of hydration and hence for hardening. It also prevents the exposure of concrete to a hot atmosphere and to drying winds which may lead to quick drying out of moisture in the concrete and there by subject it to contraction stresses at a stage when the concrete would not be strong enough to resists them. Here curing is done by spraying water on the jute bags spread over the surface for a period of 14 days

3.4 STRENGTHENING OF BEAMS

At the time of bonding of fiber, the concrete surface is made rough using a coarse sand paper texture and then cleaned with an air blower to remove all dirt and debris. After that the epoxy resin is mixed in accordance with manufacturer's instructions. The mixing is carried out in a plastic container (100 parts by weight of Araldite LY 556 to 10 parts by weight of Hardener HY 951). After their uniform mixing, the fabrics are cut according to the size then the epoxy resin is applied to the concrete surface. Then the GFRP sheet is placed on top of epoxy resin coating and the resin is squeezed through the roving of the fabric with the roller .Air bubbles entrapped at the epoxy/concrete or epoxy/fabric interface are eliminated.

During hardening of the epoxy, a constant uniform pressure is applied on the composite fabric surface in order to extrude the excess epoxy resin and to ensure good contact between the epoxy, the concrete and the fabric. This operation is carried out at room temperature. Concrete beams strengthened with glass fiber fabric are cured for 24 hours at room temperature before testing.

Fig. 3.6 Application of epoxy and hardener on the beam

Fig. 3.7 Fixing of GFRP sheets on the beam

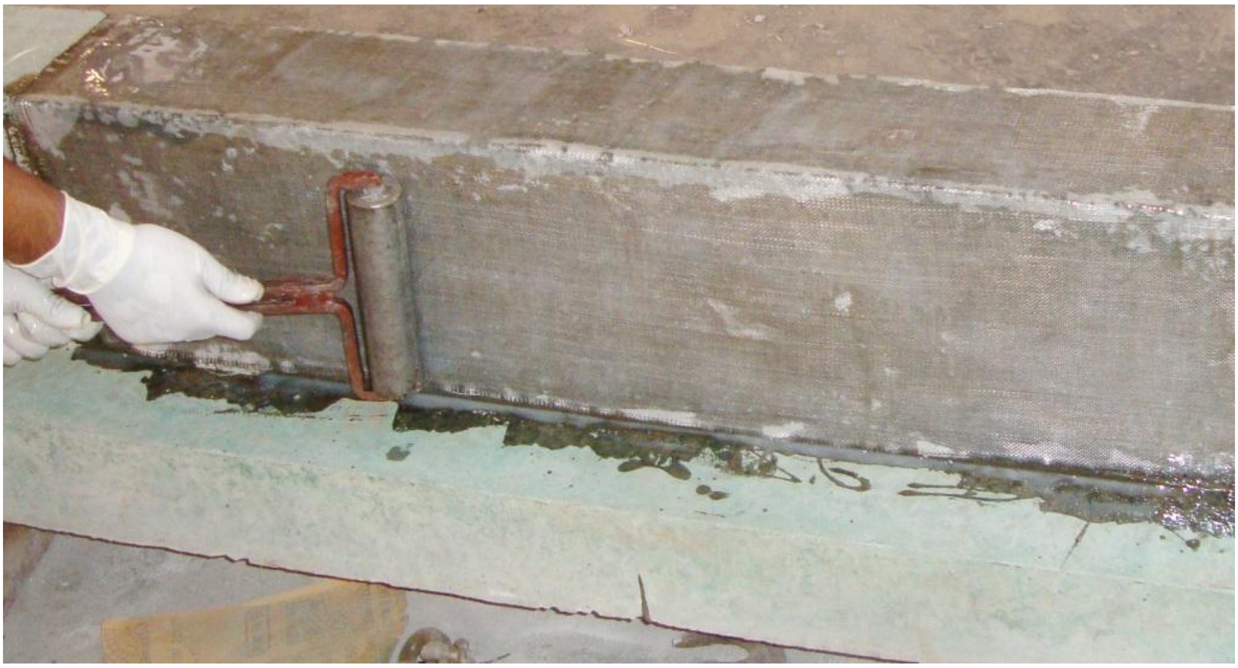


Fig 3.8 Roller used for the removal of air bubble

3.5 EXPERIMENTAL SETUP

The T-beams are tested in the loading frame of the “Structural Engineering” Laboratory of National Institute of Technology, Rourkela. The testing procedure for the all the specimen is same. First the beams are cured for a period of 28 days then its surface is cleaned with the help of sand paper for clear visibility of cracks. The two-point loading arrangement is used for testing of beams. This has the advantage of a substantial region of nearly uniform moment coupled with very small shears, enabling the bending capacity of the central portion to be assessed. Two-point loading is conveniently provided by the arrangement shown in Figure.

The load is transmitted through a load cell and spherical seating on to a spreader beam. The spreader beam is installed on rollers seated on steel plates bedded on the test member with cement in order to provide a smooth leveled surface. The test member is supported on roller bearings acting on similar spreader plates. The specimen is placed over the two steel rollers bearing leaving 150 mm from the ends of the beam. The remaining 1700 mm is divided into three equal parts of 567 mm as shown in the figure. Two point loading arrangement is done as shown in the figure. Loading is done by hydraulic jack .Lines are marked on the beam to be tested at L/3 ,L/2 & 2L/3 locations from the left support(where L=1700mm the center to center distance between the supports) Three dial gauges are used for recording the deflection of the beams. One dial gauge is placed just below the center of the beam at L/2 and the remaining two dial gauges are placed just below the point loads i.e at L/3 and 2L/3 to measure deflections.

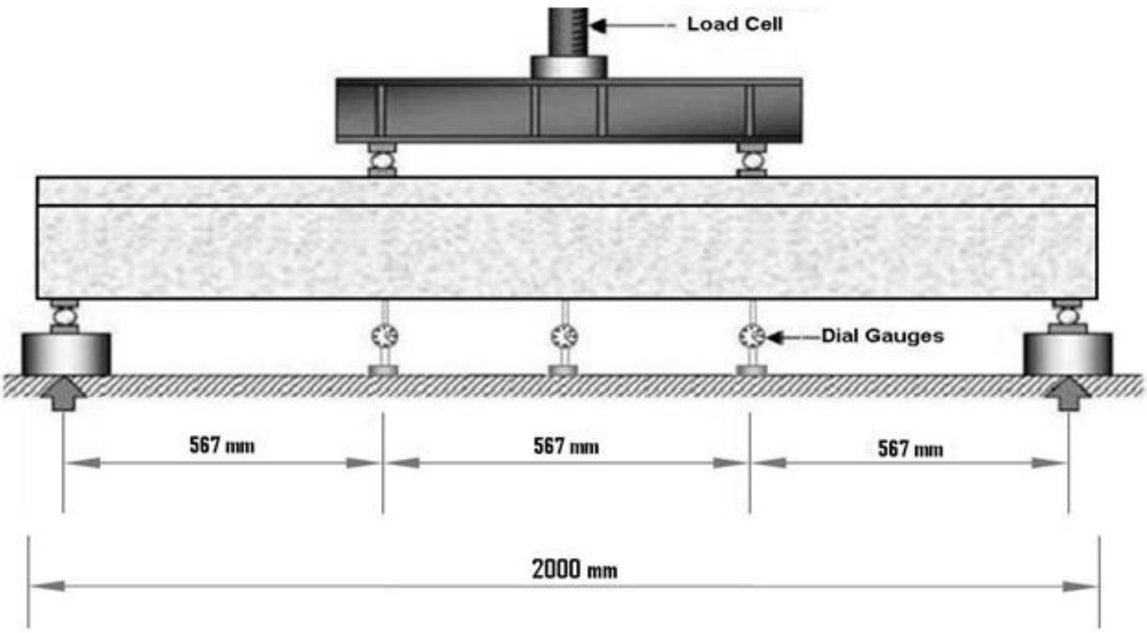


Fig. 3.9 Two point loading experimental setup

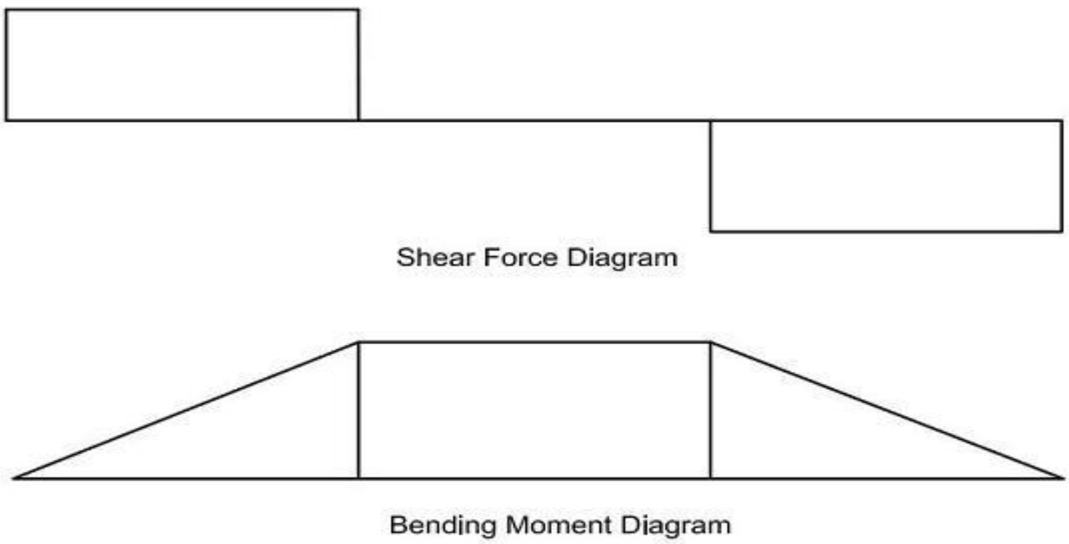


Fig. 3.10 Shear force and bending moment diagram for two point loading

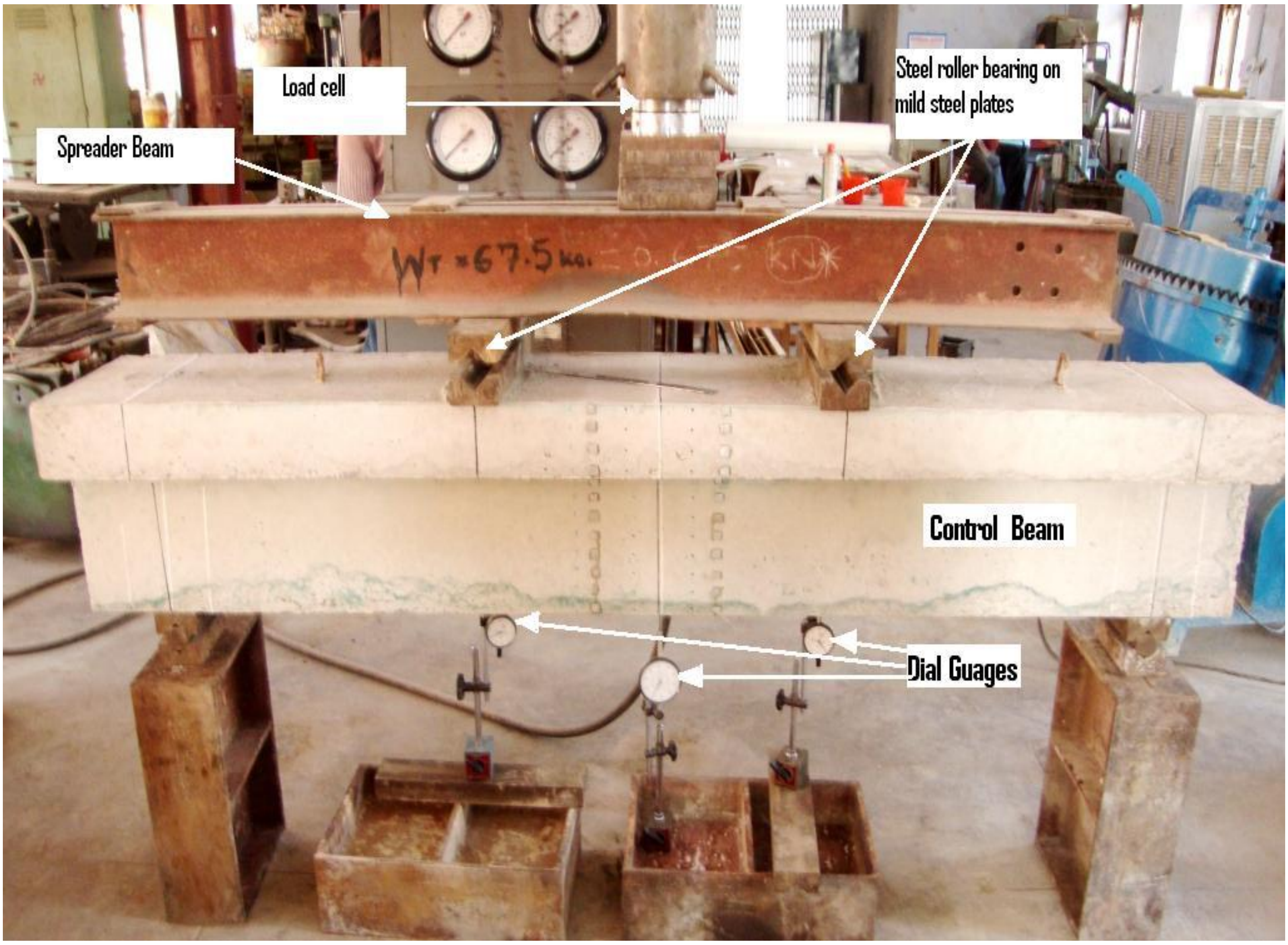


Fig. 3.11 Experimental Setup

3.5.1 TESTING OF BEAMS

All The seven are tested one by one .Six with FRP and one without FRP which is taken as the control Beam .All of them are tested in the above arrangement. The gradual increase in load and the deformation in the strain gauge reading are taken throughout the test. The dial gauge reading shows the deformation. The load at which the first visible crack is developed is recorded as cracking load. Then the load is applied till the ultimate failure of the beam.

The deflections at three salient points mentioned for the beams with and without GFRP are recorded with respect to increase of load and are furnished in table. The data furnished in this chapter have been interpreted and discussed in the next chapter to obtain a conclusion.

3.5.1.1 BEAM NO.1- CONTRL BEAM

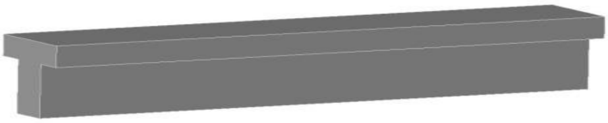


Fig. 3.12 Experimental Setup of the Control Beam 1

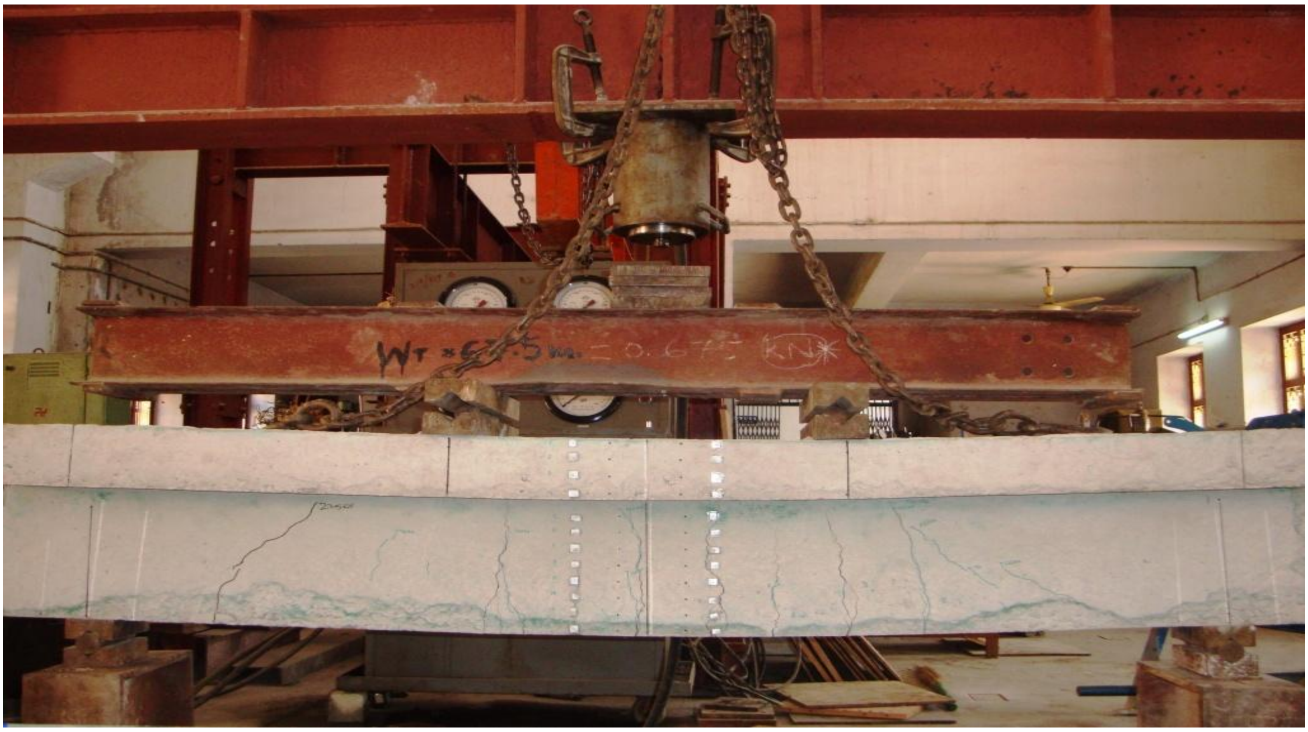


Fig 3.13 Control Beam After Cracking

Fig 3.14 Crack Pattern At 2l/3 (Near Right Support)

Fig 3.15 Crack Pattern At L/2(At Center)

Table 3.7 Deflection Values of Control Beam1

LOAD (in KN)	At Point L/3 (in mm)	At Point L/2 (in mm)	Remarks
0	0	0	
20	0.36	0.44	
30	0.53	0.63	
40	0.64	0.76	
50	0.75	0.89	
60	0.86	1.02	
70	1.00	1.20	
80	1.16	1.33	
90	1.29	1.54	
100	1.46	1.74	
110	1.58	1.88	
120	1.78	2.10	Hairline crack started appearing
130	1.91	2.23	
140	2.06	2.40	
150	2.19	2.52	
160	2.34	2.73	
170	2.54	2.95	
180	2.73	3.16	
190	2.87	3.32	
200	3.01	3.48	
210	3.14	3.63	
225	3.38	3.88	
240	3.63	4.17	Ultimate Load

3.5.1.2 BEAM-2

Single Layered GFRP bonded at Bottom of Web from end to end

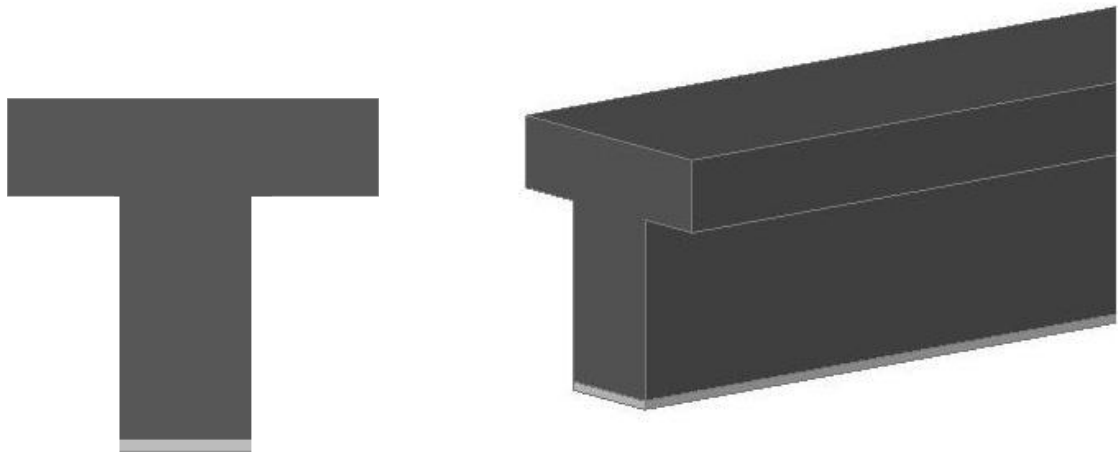


Fig. 3.16 Experimental Setup of the Beam 2

Crack Patterns In Beam 2

Fig 3.17 Debonding of Fiber

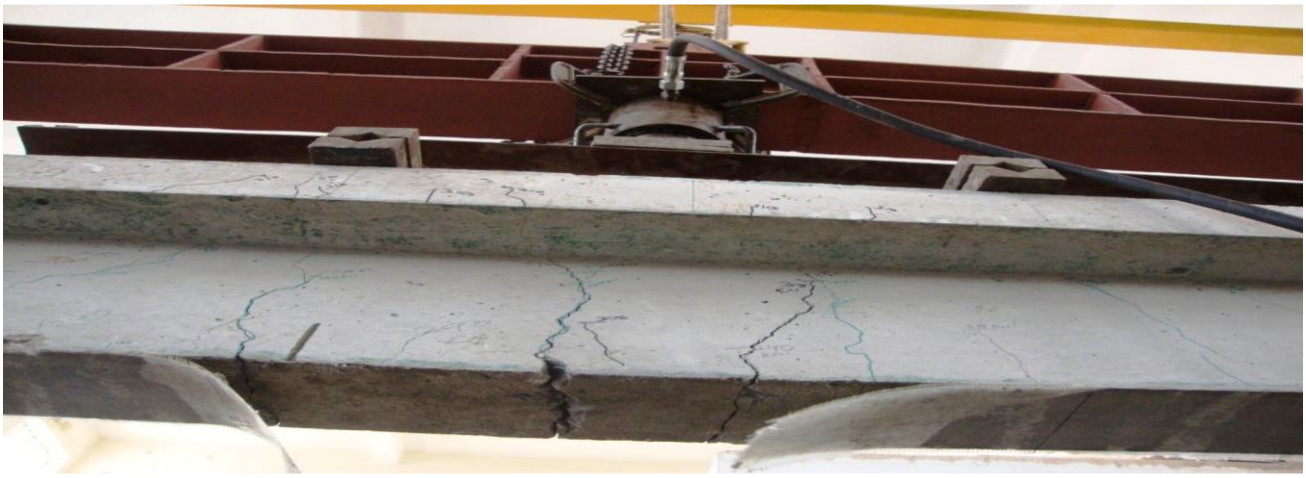


Fig 3.18 Debonding is around 21 cm and cracks occurred near the centre and under the loads

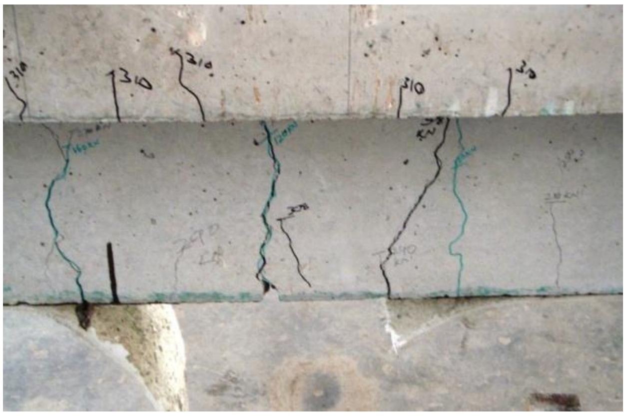


Fig 3.19 Cracks under the load at L/3

Fig 3.20 Cracks under the load at 2L/3

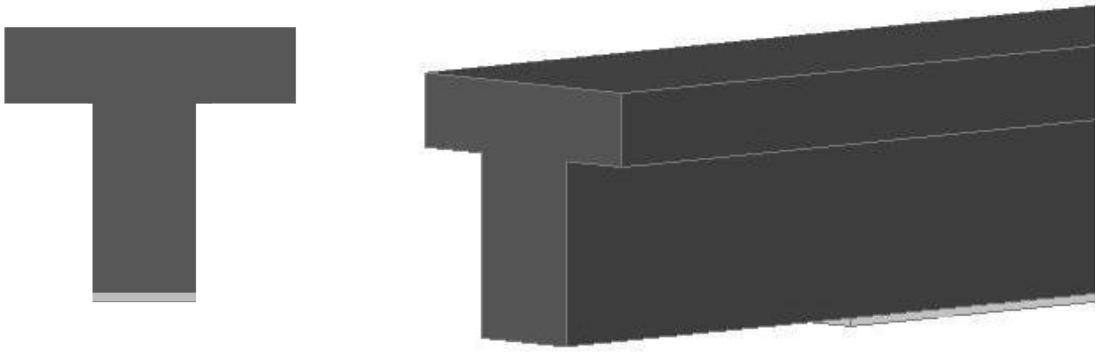
Table 3.8 Deflection Values of Beam 2

LOAD (in KN)	At Point L/3 (in mm)	At Point L/2 (in mm)	Remarks
0	0	0	
20	.26	.31	
30	.36	.44	
40	.45	.55	
50	.56	.67	
60	.72	.87	
70	.82	.99	
80	.97	1.17	
90	1.12	1.33	
100	1.28	1.51	
110	1.40	1.65	
120	1.57	1.85	
130	1.70	1.99	Hairline cracks appeared
140	1.84	2.15	
150	2.00	2.32	
160	2.16	2.49	
170	2.32	2.66	
180	2.45	2.81	Debonding of fiber
190	2.61	2.98	
200	2.74	3.10	
210	2.84	3.23	
220	3.00	3.39	
230	3.10	3.54	
240	3.25	3.68	
250	3.38	3.82	
260	3.50	3.98	
270	3.65	4.14	
276	4.07	4.68	Tearing of fiber
290	4.53	5.40	
308			Tearing and debonding
310			Ultimate Load

3.5.1.3 BEAM-3

Single Layered GFRP bonded at Bottom of Web from L/3 to 2L/3

As the cracks occurred at the centre and under the loads in Beam 2 so the bonding of the GFRP is done only in the mid section i.e. from L/3 to 2L/3 of Beam 3.



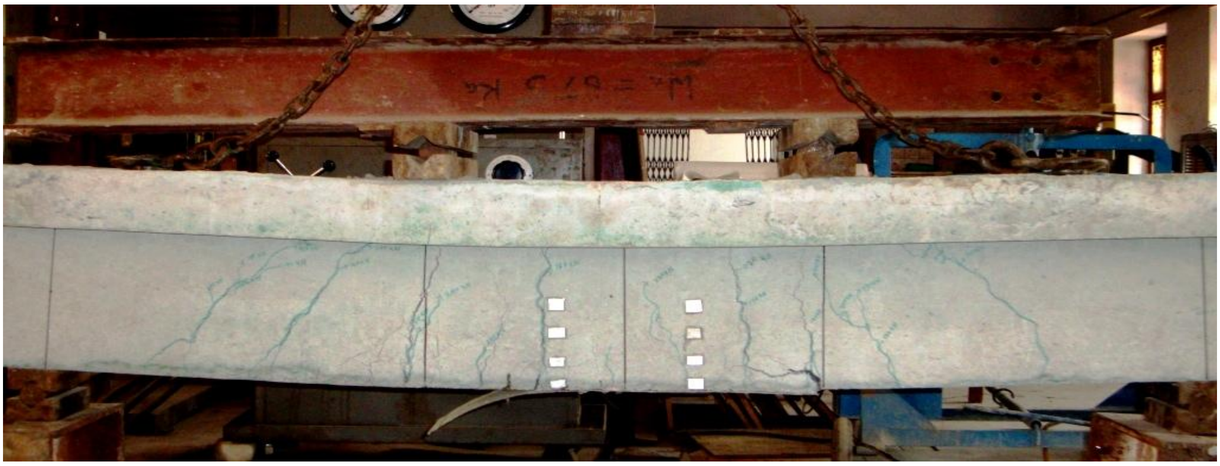


Fig 3.21 Cracks in Beam 3

Fig 3.22 Debonding of fiber at L/3 and 2L/3

Here the debonding of the fiber started before cracks occurred at the midsection so the GFRP strength is not utilised to its full extent .The debonding occurred due to the less anchorage area available between concrete surface and GFRP.As a results no tearing of the GFRP is seen.

Table 3.9 Deflection Values of Beam 3

LOAD (in KN)	At Point L/3 (in mm)	At Point L/2 (in mm)	Remarks
0	0	0	
20	.27	.29	
30	.35	.38	
40	.45	.50	
50	.56	.61	
60	.68	.74	
70	.78	.85	
80	.92	1.00	
90	1.03	1.14	
100	1.19	1.32	
110	1.28	141	
120	1.42	1.58	
130	1.55	1.73	Hair line cracks appeared at 125
140	1.80	2.00	
150	1.90	2.11	
160	2.13	2.36	
170	2.27	2.50	
180	2.48	2.71	
190	2.60	2.85	
200	2.78	3.04	
210	2.95	3.21	
220	3.10	3.37	
230	3.28	3.58	
240	3.43	3.74	
250	3.61	3.93	
260	3.80	4.11	
270	4.20	4.50	Debonding occured
290		Ultimate load	Debonding but no tearing

3.5.1.4 BEAM-4

U- Jacketed Single Layered GFRP bonded from end to end

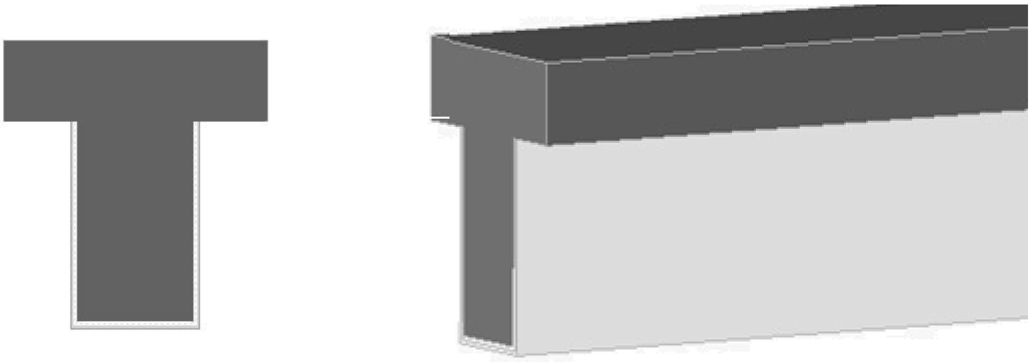


Fig 3.23 U-jacketed GFRP wrapped on the Beam 4

Fig 3.24 Beam 4 after testing

Fig 3.25 Tearing of GFRP in Beam 4 at 300 KN

Fig 3.26 Crack developed in Beam 4 under the GFRP

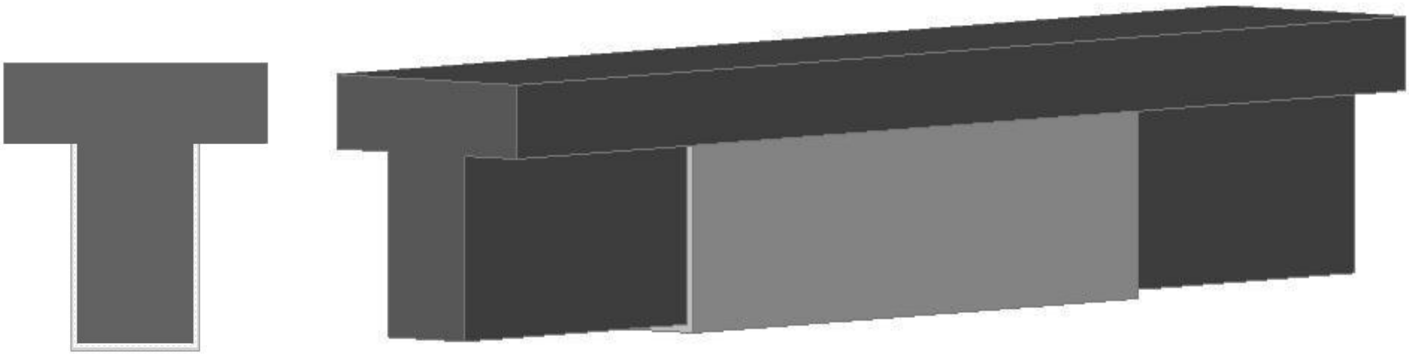
As the cracks developed under the GFRF are within 1000 mm at the mid section therefore in Beam 5 U Jacketed wrapping is done only for that 1000 mm in the mid section.

Table 3.10 deflection values of beam 4

LOAD (inKN)	At Point L/3 (in mm)	At Point L/2 (in mm)	Remarks
0	0	0	
20	.28	.24	
30	.38	.35	
40	.49	.45	
50	.59	.58	
60	.69	.71	
70	.84	.89	
80	.95	1.03	
90	1.10	1.20	
100	1.25	1.37	
110	1.38	1.53	
120	1.56	1.71	
130	1.72	1.88	
140	1.84	2.04	
150	1.98	2.18	
160	2.13	2.35	
170	2.28	2.51	
180	2.48	2.75	
190	2.63	2.92	
200	2.79	3.10	
210	2.95	3.26	
220	3.08	3.43	
230	3.26	3.63	
240	3.41	3.78	
250	3.55	3.96	
260	3.73	4.15	
270	4.65	5.40	
280	6.95	7.60	
290			Tearing started
318	Tearing and	Debonding started	simultaneously
330			Ultimate Load

3.5.1.4 BEAM-5

U- Jacketed Single Layered GFRP bonded only at the mid section



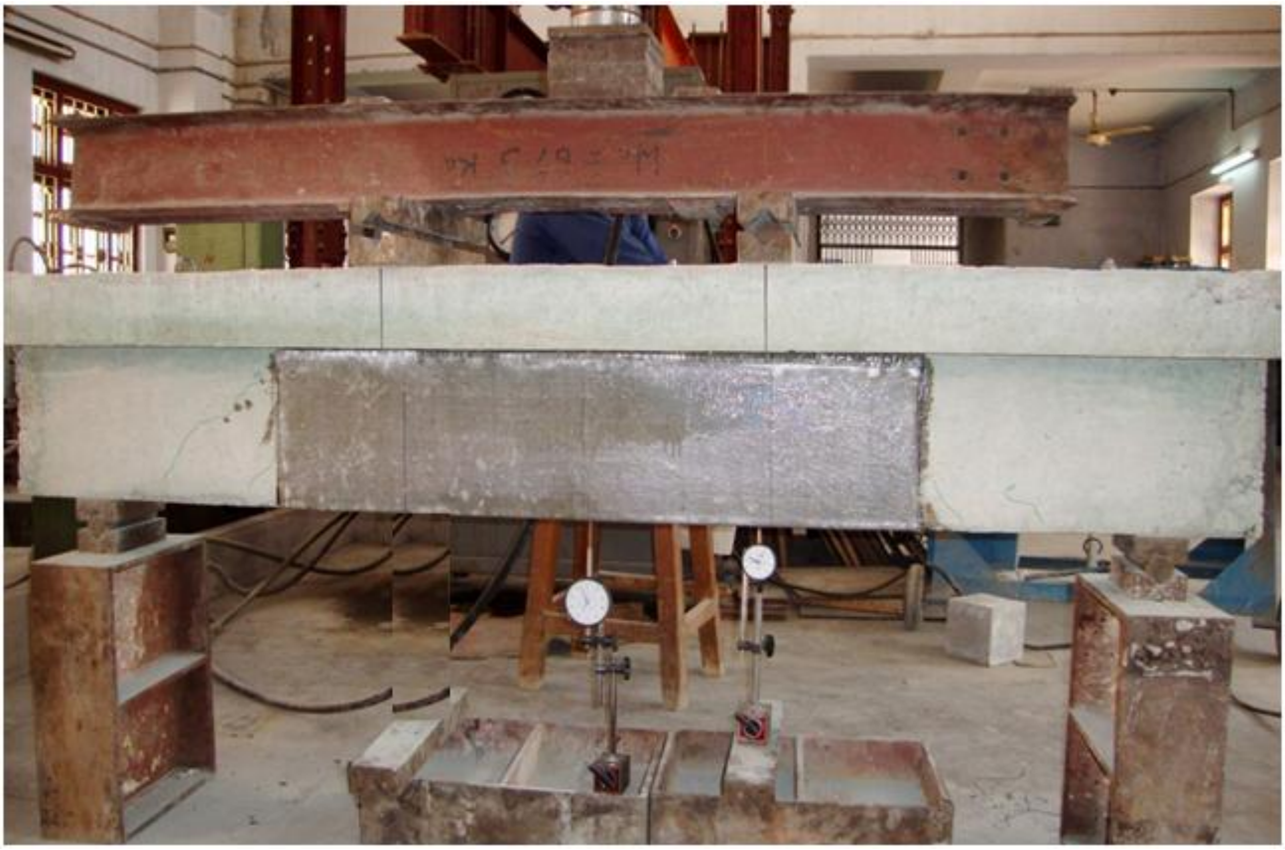


Fig. 3.27 Experimental Setup of the Beam 5

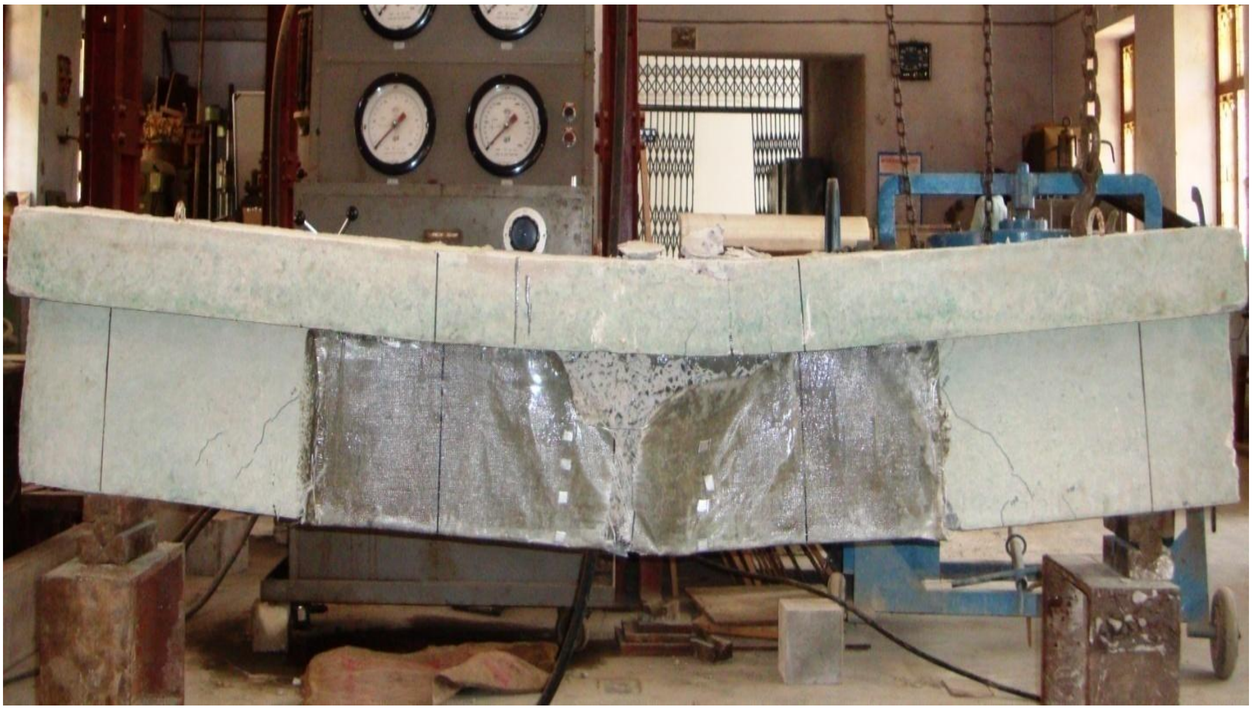


Fig. 3.28 Beam 5 after testing

Fig. 3.29 Tearing started at the mid section at 308 KN

Fig 3.30 After total tearing of the fiber at the midsection debonding started at the end of the fiber. At the top the debonding occurred up to 2L/3.

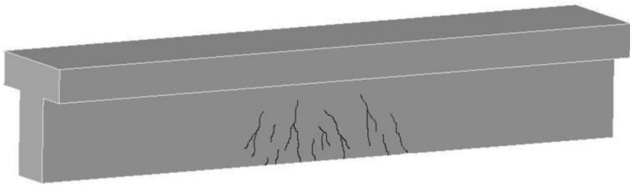
Fig 3.31Crack developed under GFRP in BEAM 5

Table 3.11 Deflection values of Beam 5

LOAD (in KN)	At Point L/3 (in mm)	At Point L/2 (in mm)	Remarks
0	0	0	
20	.27	.27	
30	.32	.32	
40	.40	.41	
50	.48	.51	
60	.60	.64	
70	.70	.77	
80	.82	.92	
90	.92	1.02	
100	1.03	1.15	
110	1.15	1.30	
120	1.30	1.45	
130	1.47	1.68	
140	1.61	1.82	
150	1.75	1.97	
160	1.85	2.14	
170	2.08	2.31	
180	2.23	2.48	
190	2.38	2.66	
200	2.53	2.81	
210	268	2.99	
220	2.84	3.15	
230	2.96	3.30	
240	3.11	3.48	
250	3.27	3.65	
260	3.42	3.81	
270	3.62	4.03	
280	3.78	4.21	
294	4.40	5.00	Tearing started
309			Tearing and debonding started
316			Ultimate Load

3.5.1.6 BEAM-6

U- Jacketed Single Layered GFRP retrofitted on cracked beam at the mid section.



Cracked Beam. Retrofitted beam

Fig 3.32 Crack formed after initial loading in Beam 6

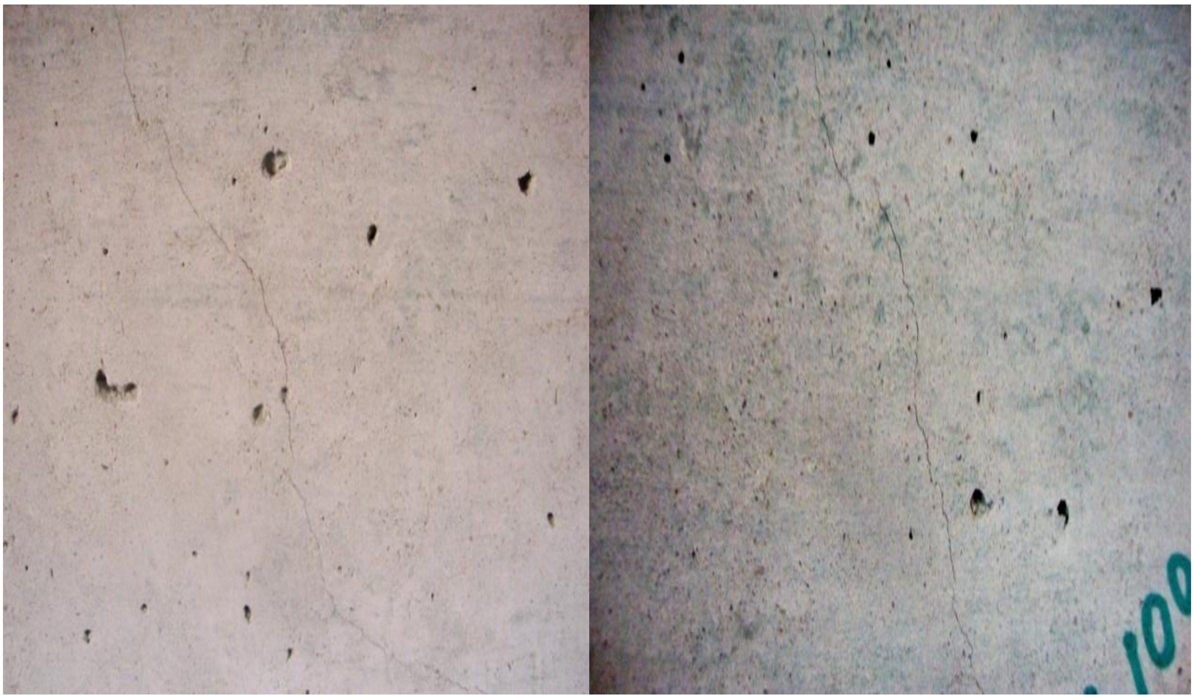


Fig 3.33 Magnified picture of the hair line cracks formed during the initial loading in Beam 6

Fig 3.34 Retrofitted Beam 6

Fig 3.35 Debonding of the Retrofitted Beam 6

Fig 3.36 Crack pattern under the GFRP in Beam 6

Table 3.12 Deflection of BEAM 6 (before retrofitting)

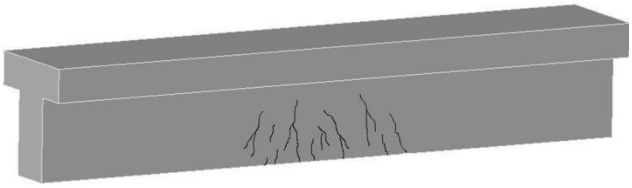
LOAD (in KN)	At Point L/3 (in mm)	At Point L/2 (in mm)	Remarks
0	0	0	
20	.30	.36	
30	.36	.44	
40	.46	.56	
50	.58	.68	
60	.71	.82	
70	.83	.94	
80	.97	1.07	
90	1.11	1.23	
100	1.25	1.37	
110	1.4	1.54	
120	1.54	1.69	
130	1.70	1.87	
140	1.85	2.02	
150	2.00	2.20	

Table 3.13 Deflection of Beam 6 (after retrofitting)

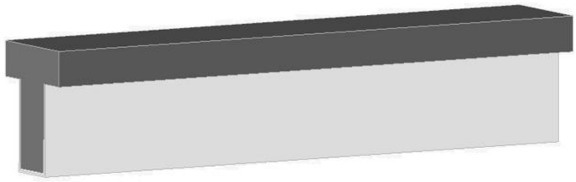
LOAD (in KN)	At Point L/3 (in mm)	At Point L/2 (in mm)	Remarks
0	0	0	
30	.37	.42	
40	.48	.54	
50	.58	.64	
60	.67	.74	
70	.76	.85	
80	.87	.98	
90	.97	1.08	
100	1.05	1.17	
110	1.15	1.28	
120	1.25	1.39	
130	1.35	1.51	
140	1.45	1.65	
150	1.57	1.74	
160	1.68	1.87	
170	1.79	1.99	
180	1.97	2.16	
190	2.10	2.30	
200	2.25	2.48	
210	2.50	2.70	
220	2.68	2.91	
230	2.85	3.08	
240	3.00	3.24	
250	3.15	3.41	
260	3.38	3.70	
270	4.90	6.10	

3.5.1.7 BEAM-7

U- Jacketed Single Layered GFRP retrofitted on cracked beam from end to end.



Cracked Beam.



Retrofitted beam

Fig 3.37 Crack patterns after initial loading in Beam 7

Fig 3.38 Magnified picture of the crack patterns in Beam 7

Fig 3.39 Retrofitted Beam 7

Fig 3.40 Tearing of GFRP in Beam 7

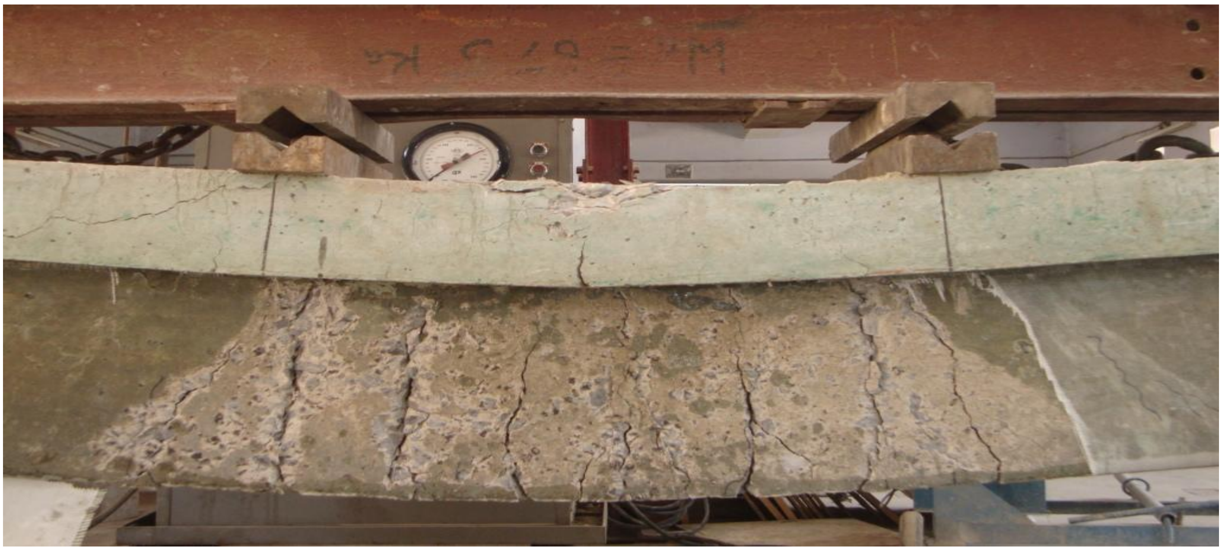


Fig 3.41 Cracks under GFRP in Beam 7

Table 3.14 Deflection of Beam 7(before retrofitting)

LOAD (in KN)	At Point L/3 (in mm)	At Point L/2 (in mm)	Remarks
0	0	0	
	.45	.52	
40	.54	.63	
50	.64	.75	
60	.75	.86	
70	.85	.98	
80	.96	1.12	
90	1.08	1.28	
100	1.21	1.41	
110	1.35	1.58	
120	1.49	1.73	
130	1.63	1.88	
140	1.79	2.05	
150	1.96	2.25	

Table 3.15 Deflection of Beam-7(after retrofitting)

LOAD (in KN)	At Point L/3 (in mm)	At Point L/2 (in mm)	Remarks
0	0	0	
20	.44	.45	
30	.53	.55	
40	.65	.68	
50	.79	.83	
60	.90	.96	
70	1.01	1.09	
80	1.13	1.22	
90	1.24	1.36	
100	1.35	1.48	
110	1.46	1.60	
120	1.58	1.72	
130	1.69	1.84	
140	1.81	1.96	
150	1.93	2.10	
160	2.05	2.22	
170	2.19	2.38	
180	2.31	2.55	
190	2.52	2.72	
200	2.68	2.89	
210	2.82	3.06	
220	2.98	3.23	
230	3.14	3.41	
240	3.32	3.62	
250	3.48	3.78	
260	3.68	4.01	
270			Debonding
306			Tearing
326			Ultimate

CHAPTER -4

RESULTS AND DISCUSSIONS

4.1 INTRODUCTION

In this chapter the experimental results of all the beams with different types of layering of GFRP are interpreted. Their behavior throughout the test is described using recorded data on deflection behavior and the ultimate load carrying capacity. The crack patterns and the mode of failure of each beam are also described in this chapter. All the beams are tested for their ultimate strengths.Beams-1 is taken as the control beam. It is observed that the control beam had less load carrying capacity and high deflection values compared to that of the externally strengthened beams using GFRP sheets.

All the beams except the control beam are strengthened with GFRP sheets in different patterns.Beam-2 is strengthened only at the soffit from end to end. Beam-3 is also strengthened only at the soffit but for the length L/3 to 2L/3. In Beam-4 the web part is strengthened throughout its length with U-Jacketed single layered GFRP. Beam-5 is also strengthened in the web but for 1 m length in the middle portion where most of the cracks are occurring. Beam-6 is first loaded till hairline cracks are formed then it is retrofitted in similar pattern as is done for Beam-5. Beam-7 is also loaded till hairline cracks are formed then it is retrofitted in similar pattern as is done for Beam-4.Deflection behavior and the ultimate load carrying capacity of the beams are noted. The ultimate load carrying capacity of all the beams along with the nature of failure is given in Table 4.1.

4.2 FAILURE MODES

The following failure modes are be investigated for a GFRP strengthened section:

• Yielding of the steel in tension followed by rupture of the GFRP laminate;

• Yielding of the steel in tension followed by concrete crushing;

• Debonding of the FRP from the concrete substrate (FRP debonding).

A number of failure modes have been observed in the experiments of RC T- beams strengthened in flexure and shear by GFRPs. These include flexure failure, flexural failure due to GFRP rupture and crushing of concrete at the top. Rupture of the FRP laminate is assumed to occur if the strain in the FRP reaches its design rupture strain before the concrete reaches its maximum usable strain.GFRP debonding can occur if the force in the FRP cannot be sustained by the substrate. In order to prevent debonding of the GFRP laminate, a limitation should be placed on the strain level developed in the laminate.

The GFRP strengthened beam and the control beams are tested to find out their ultimate load carrying capacity. It is found that all the failed in flexure. Beam-2 failed due to fracture of GFRP sheet and then flexural failure of the beam took place. Beam-3 failed due to debonding of the GFRP sheet and then flexural failure of the beam. Beam-4 failed due to fracture of GFRP at the center followed by debonding and finally flexural failure of the beam.Beam-5 failed in the manner as in Beam-4. Beam-6 failed due to debonding of GFRP and then flexural failure of beam. In Beam-7 first fracture of GFRP took place at the center followed by debonding of the top portion in the center.

4.3 LOAD DEFLECTION ANALYSIS

Here the deflection of each beam at different positions is analyzed. Mid-span deflections of each beam are compared with the control beam. Also the load deflection behavior is compared between different wrapping schemes having the same reinforcement. It is noted that the behavior of the flexure deficient beams when bonded with GFRP sheets are better than the control beams. The mid-span deflections are lower when bonded externally with GFRP sheets. The use of GFRP sheet had effect in delaying the growth of crack formation.

When all the wrapping schemes are considered it is found that the Beam-6 with GFRP sheet in wrapped in the web for a length of 1m in the middle part had a better load deflection behavior compared to the others strengthened beams with GFRP.

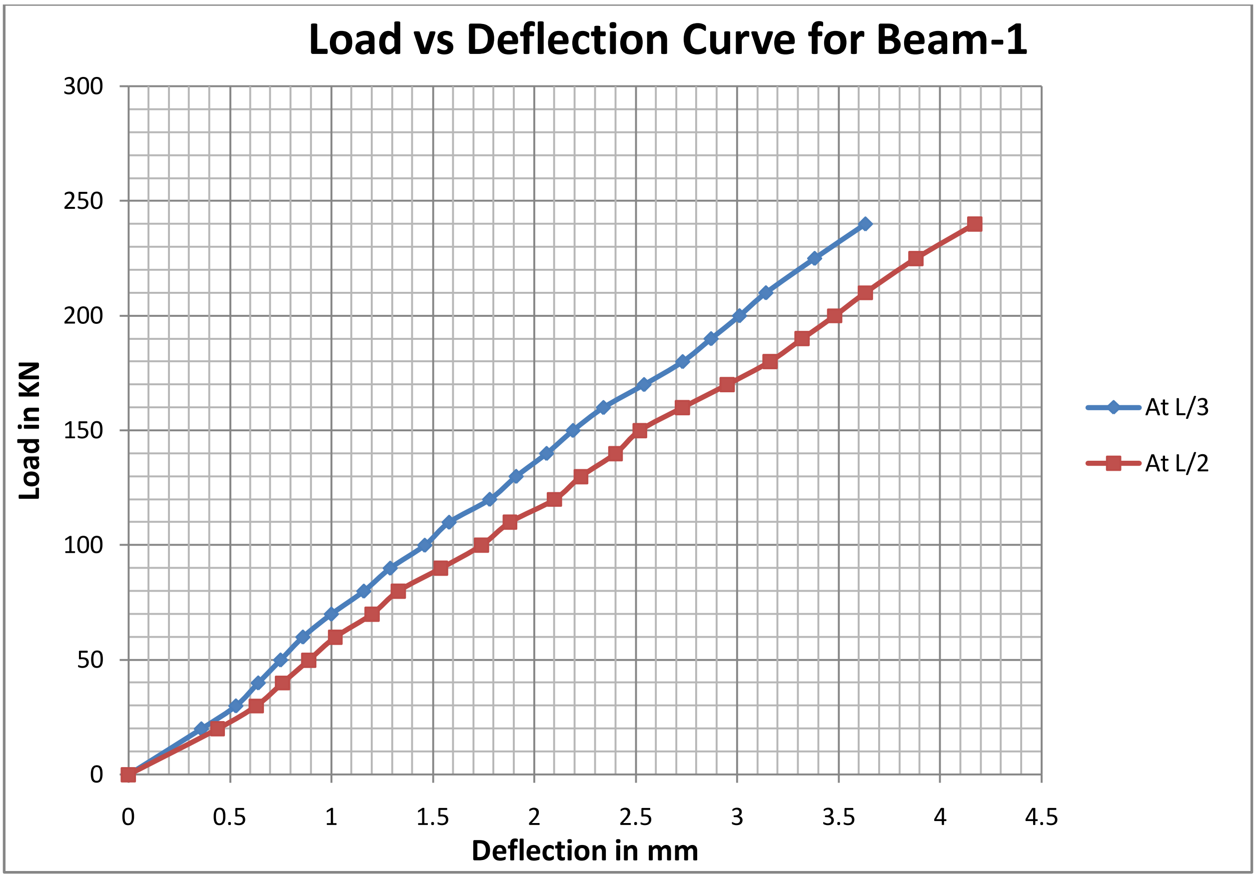


Fig. 4.1 Load vs. Deflection Curve for Control Beam 1

Beam 1 is taken as the control beam which is weak in flexure. In Beam1 no strengthening is done. Two point static loading is applied on the beam and at the each increment of the load, deflection at L/3, L/2 and 2L/3 are taken with the help of dial gauges. Using this load and deflection data, load vs. deflection curve is plotted. At the load of 120 KN initial hairline cracks appeared. Later with the increase in loading values the crack propagated further. The Beam1 failed completely in flexure

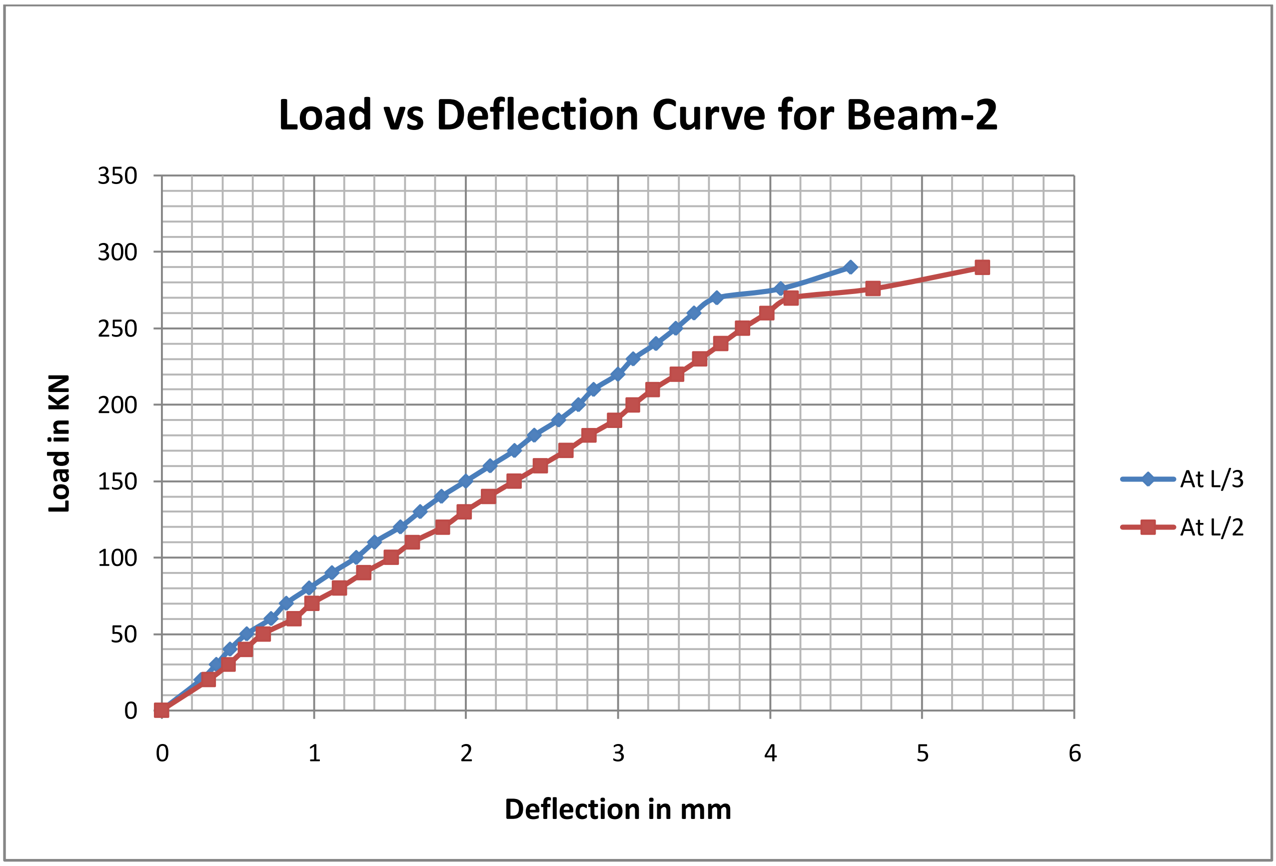


Fig. 4.2 Load vs. Deflection Curve for Beam 2

Beam-2 is strengthened only at the soffit from end to end .Two point static loading is applied on the beam and at the each increment of the load, deflection at L/3, L/2 and 2L/3 are taken with the help of dial gauges. Using this load and deflection data, load vs. deflection curve is plotted. At the load of 130 KN initial hairline cracks appeared. Later with the increase in loading values the crack propagated further. Beam-2 failed due to fracture of GFRP sheet and then flexural failure of the beam took place.

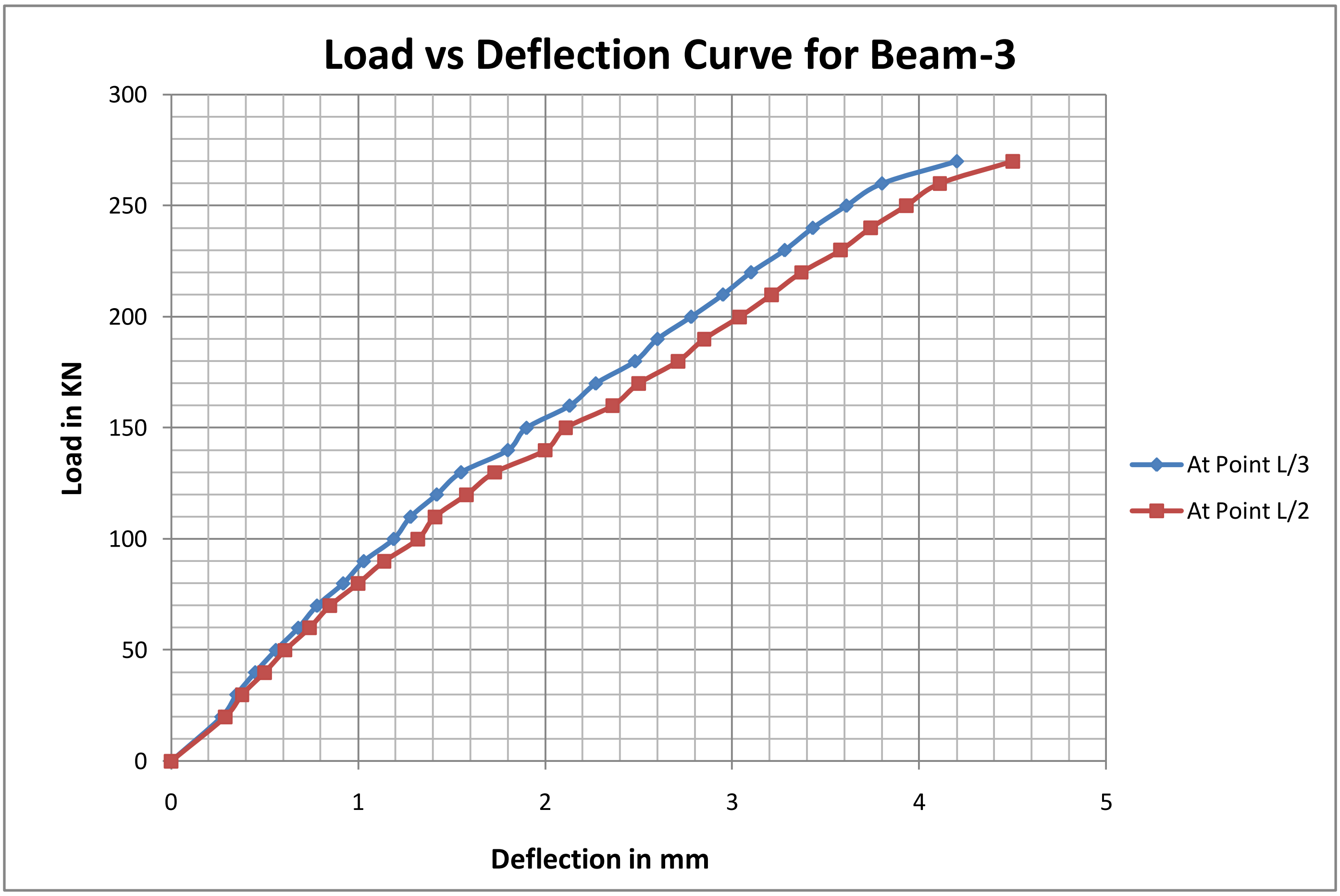


Fig. 4.3 Load vs. Deflection Curve for Beam 3

Beam-3 is also strengthened only at the soffit but for the length L/3 to 2L/3. Two point static loading is applied on the beam and at the each increment of the load, deflection at L/3, L/2 and 2L/3 are taken with the help of dial gauges. Using this load and deflection data, load vs. deflection curve is plotted. At the load of 125 KN initial hairline cracks appeared. Later with the increase in loading values the crack propagated further. Beam-3 failed due to debonding of the GFRP sheet and then flexural failure of the beam.

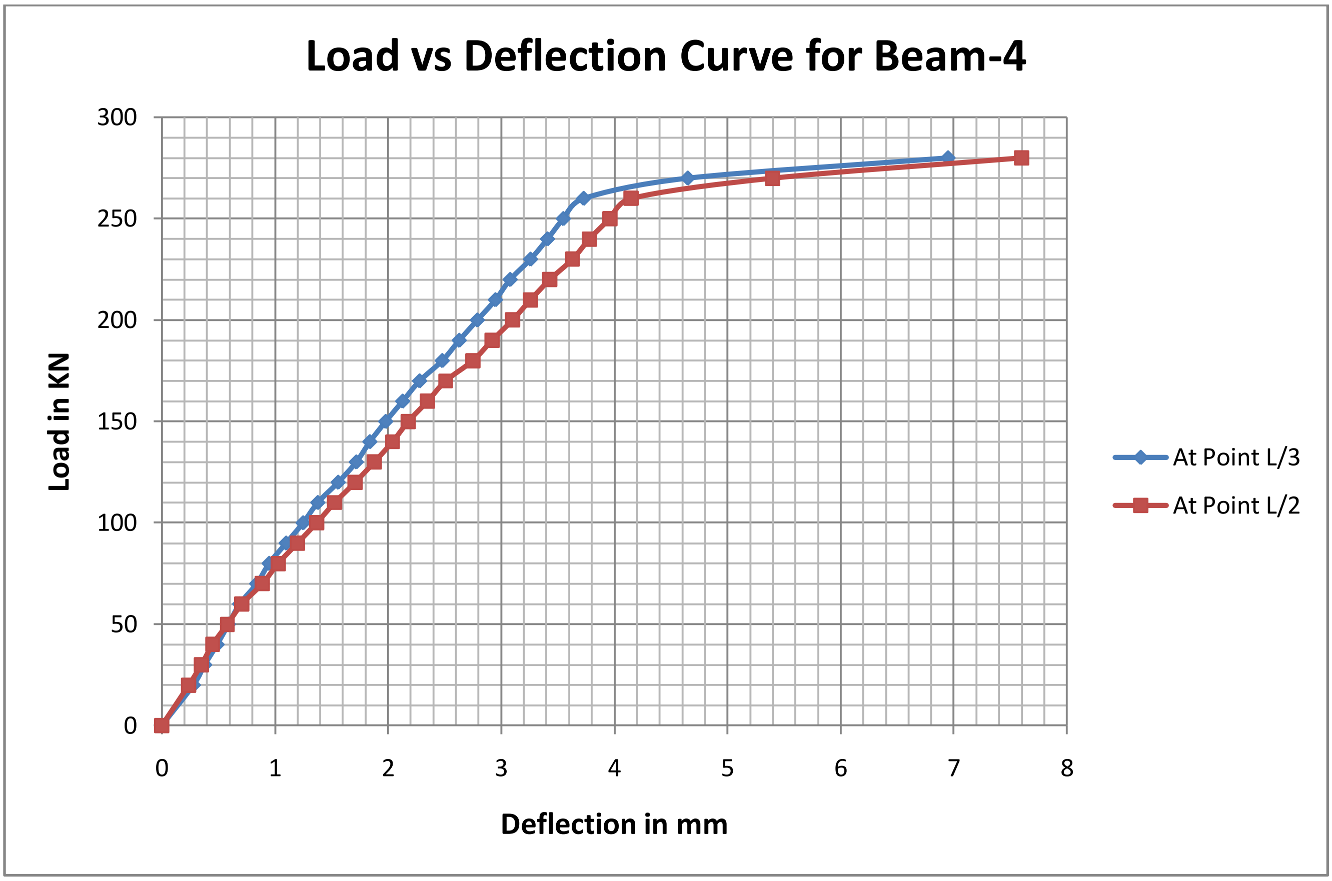


Fig. 4.4 Load vs. Deflection Curve for Beam 4

In Beam-4 the web part is strengthened throughout its length with U-Jacketed single layered GFRP. Two point static loading is applied on the beam and at the each increment of the load, deflection at L/3, L/2 and 2L/3 are taken with the help of dial gauges. Using this load and deflection data, load vs. deflection curve is plotted. No initial hairline cracks are visible due to the covering of GFRP.Later with the increase in loading values the crack propagated further under the GFRP. Beam-4 failed due to fracture of GFRP at the center followed by debonding and finally flexural failure of the beam.

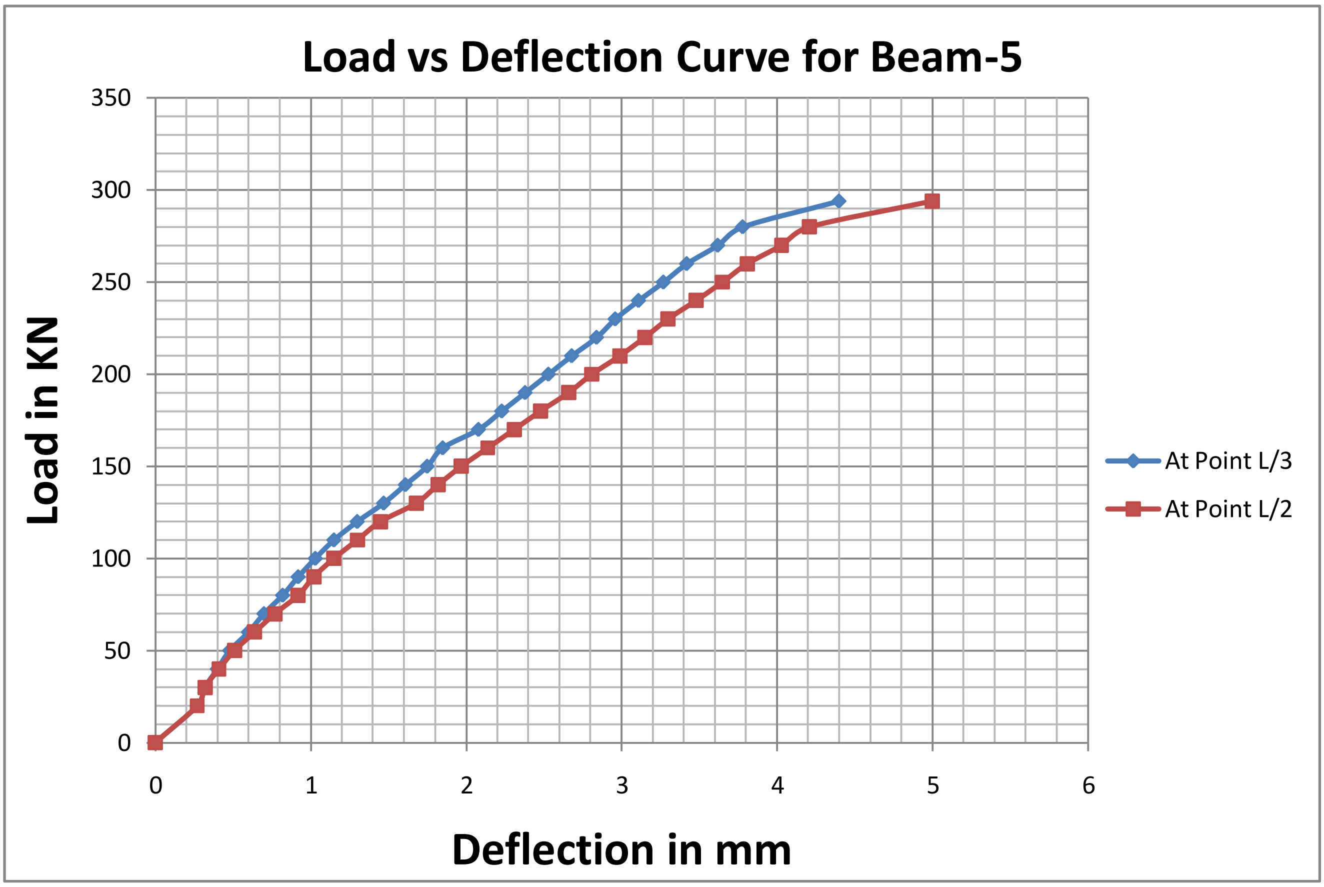


Fig 4.5 Load vs. Deflection Curve for Beam 5

Beam-5 is also strengthened in the web but for 1 m length in the middle portion where most of the cracks are occurring. Two point static loading is applied on the beam and at the each increment of the load, deflection at L/3, L/2 and 2L/3 are taken with the help of dial gauges. Using this load and deflection data, load vs. deflection curve is plotted. No initial hairline cracks are visible due to the covering of GFRP.

Later with the increase in loading values the crack propagated further under the GFRP. Beam-5 failed in the manner as in Beam-4

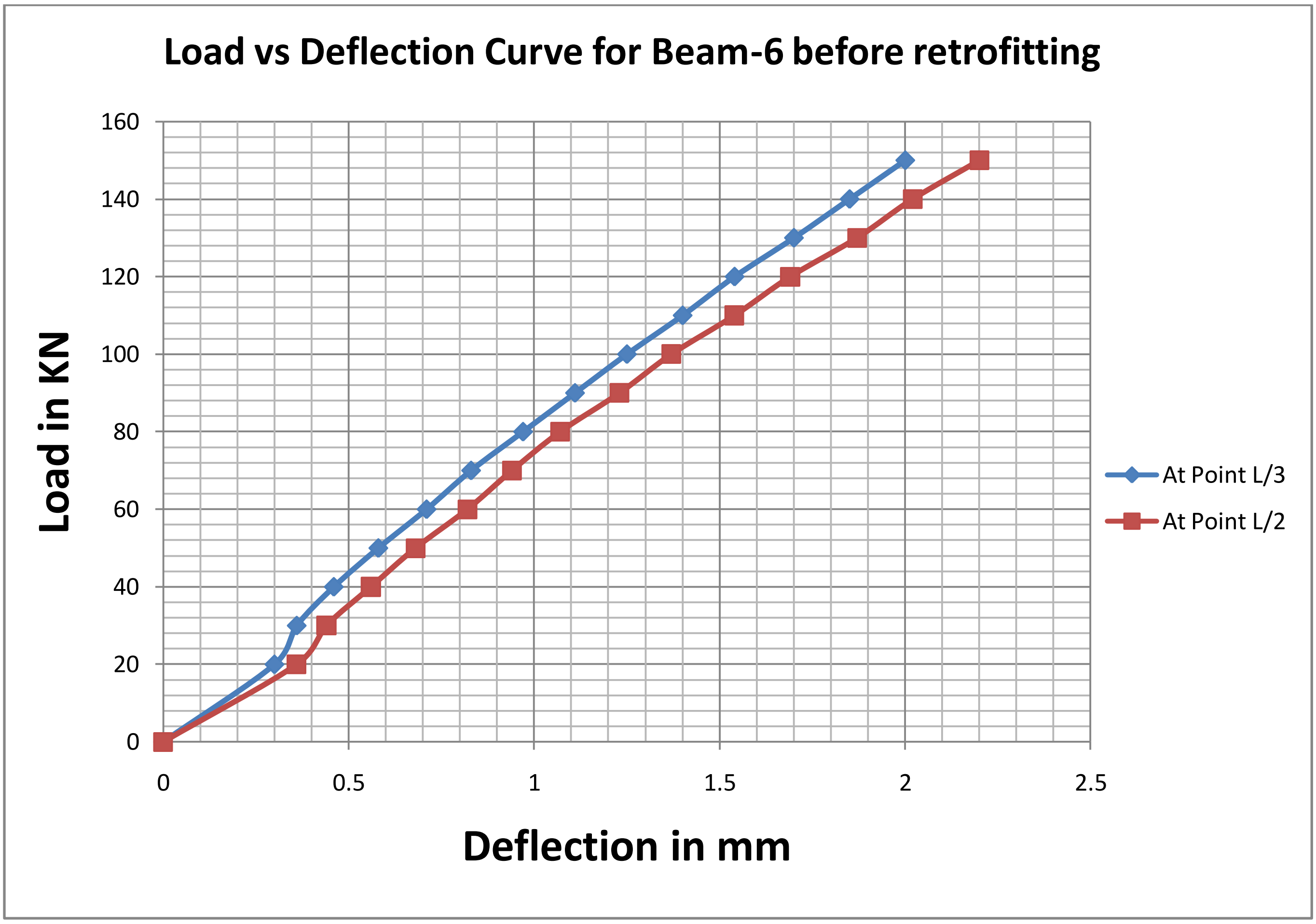


Fig. 4.6 Load vs. Deflection Curve for Beam 4 before retrofitting

Beam-6 is first loaded till hairline cracks are formed then it is retrofitted with U-Jacketed single layered GFRP for 1 m length in the middle portion. Two point static loading is applied on the beam and at the each increment of the load, deflection at L/3, L/2 and 2L/3 are taken with the help of dial gauges. Using this load and deflection data, load vs. deflection curve is plotted. No initial hairline cracks are visible due to the covering of GFRP. Later with the increase in loading values the crack propagated further under the GFRP. Beam-6 failed due to debonding of GFRP and then flexural failure of beam.

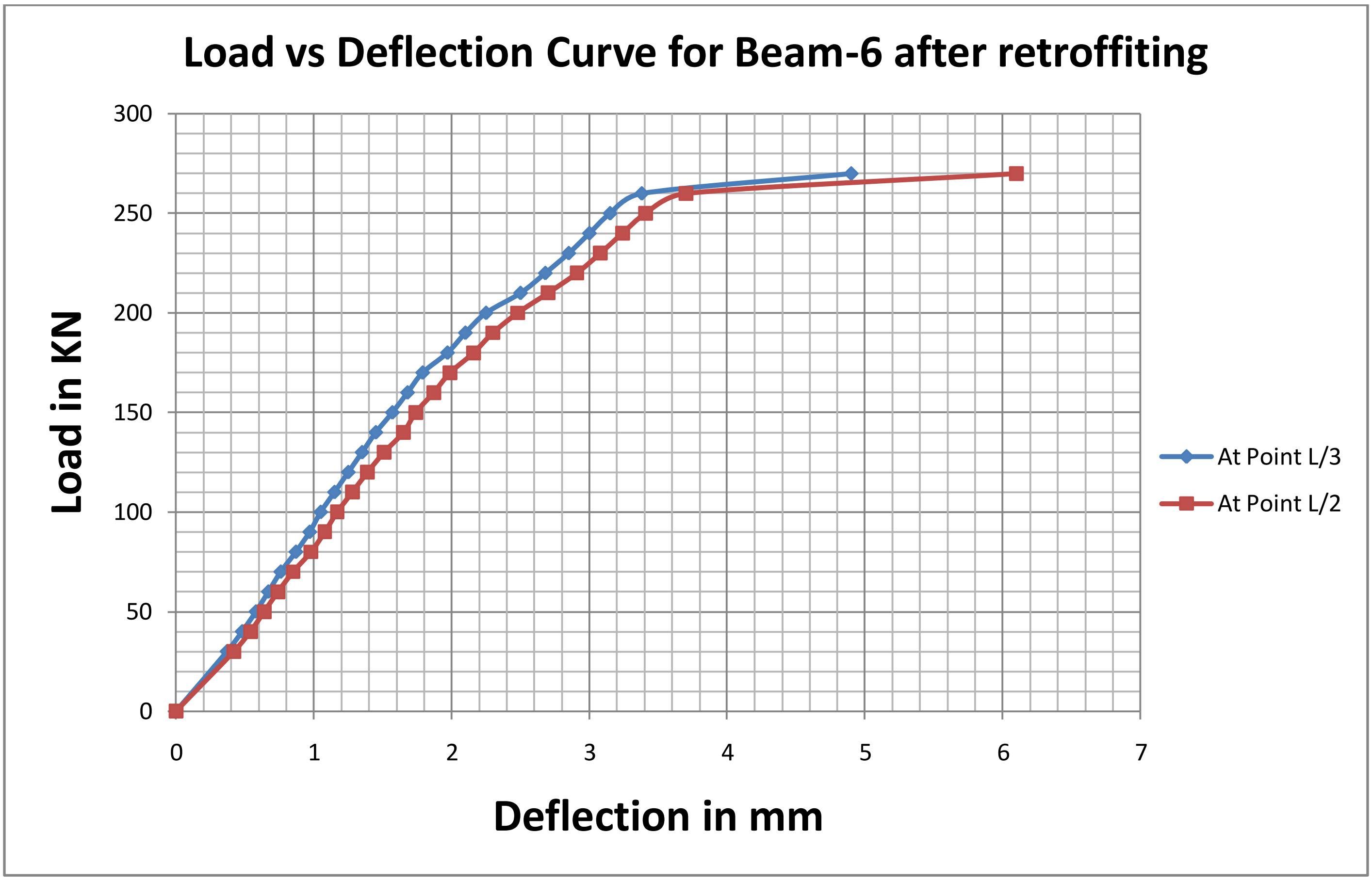


Fig. 4.7 Load vs. Deflection Curve for Beam 6 after retrofitting

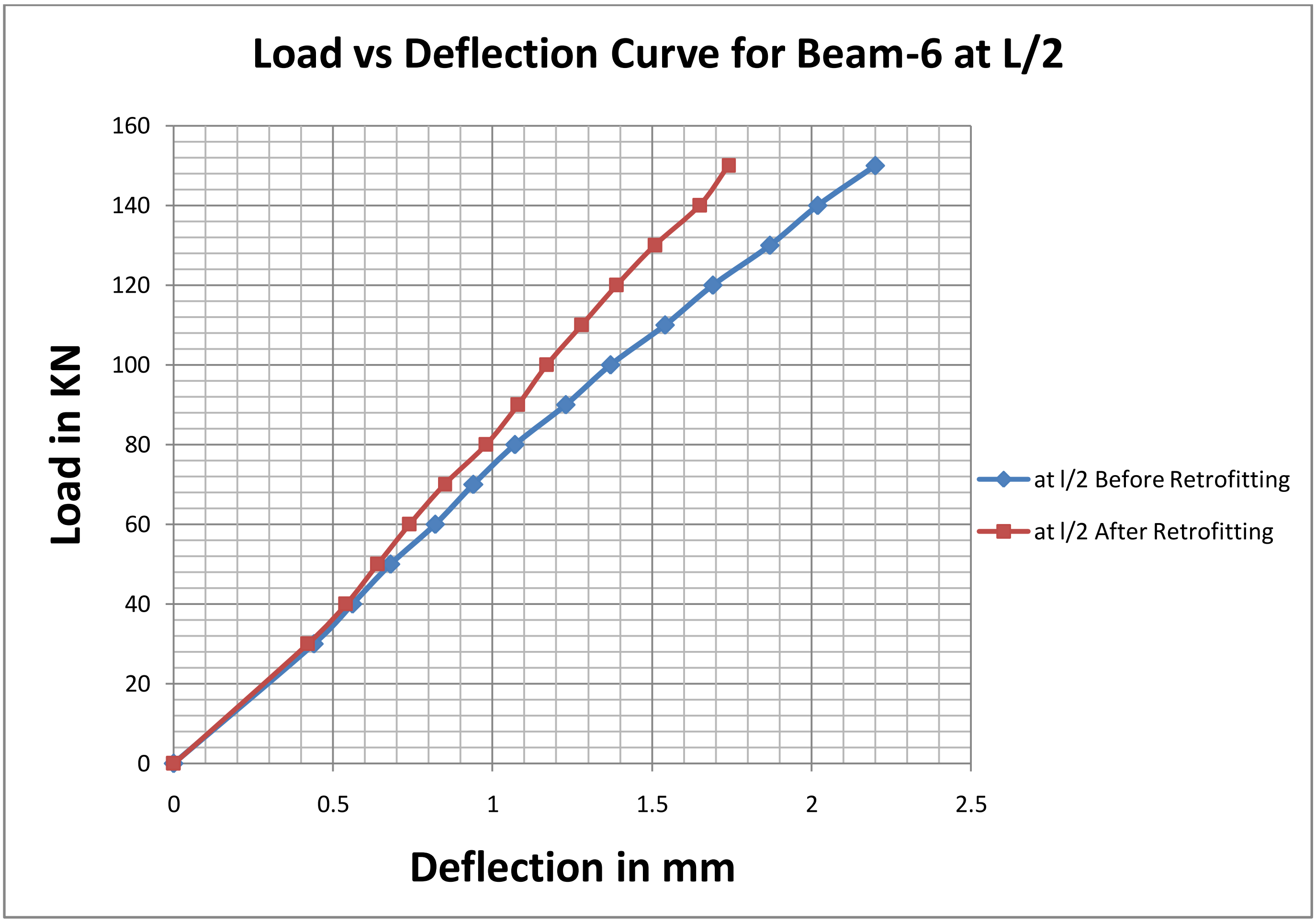


Fig. 4.8 Load vs. Deflection Curve for Beam 6 before and after retrofitting

Here it is studied that the deflection of the Beam 6 taken at the centre is controlled in the case of retrofitting.

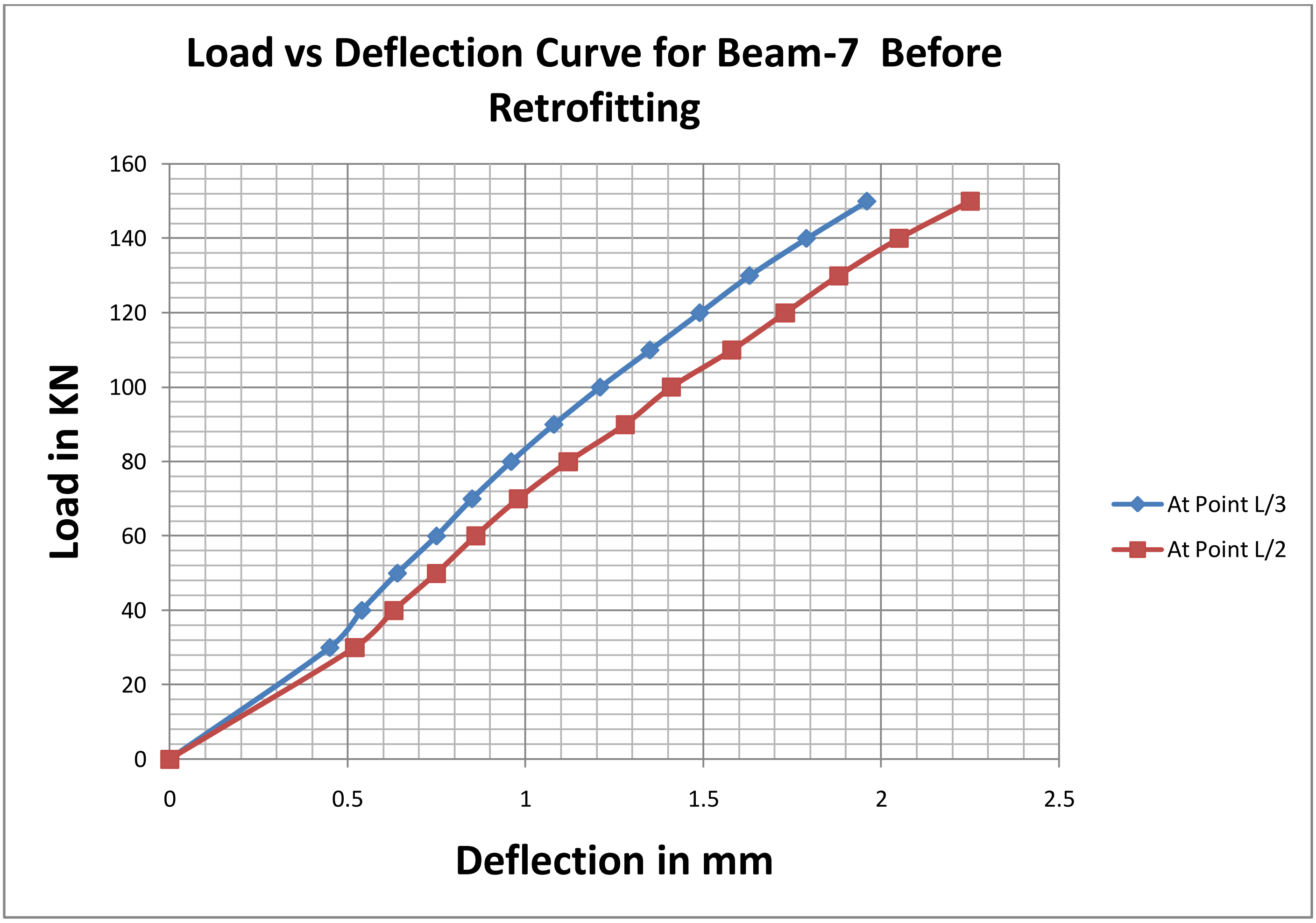


Fig.4.9 Load vs. Deflection Curve for Beam 7 before retrofitting

Beam-7 is first loaded till hairline cracks are formed then it is retrofitted with U-Jacketed single layered GFRP from end to end. Two point static loading is applied on the beam and at the each increment of the load, deflection at L/3, L/2 and 2L/3 are taken with the help of dial gauges. Using this load and deflection data, load vs. deflection curve is plotted. No initial hairline cracks are visible due to the covering of GFRP. Later with the increase in loading values the crack propagated further under the GFRP. In Beam-7 first fracture of GFRP took place at the center followed by debonding of the top portion in the center.

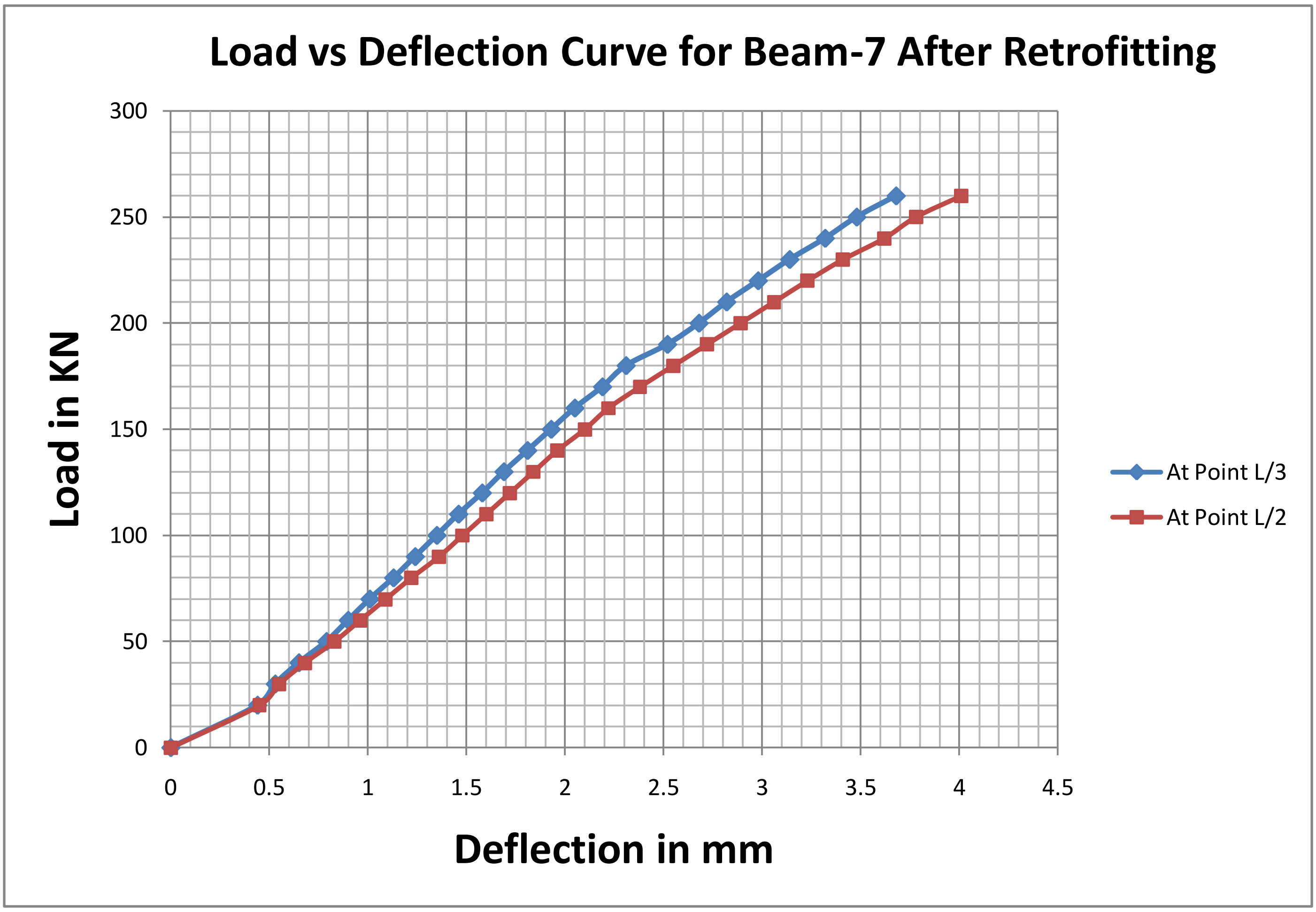


Fig. 4.10 Load vs. Deflection Curve for Beam 7 after retrofitting

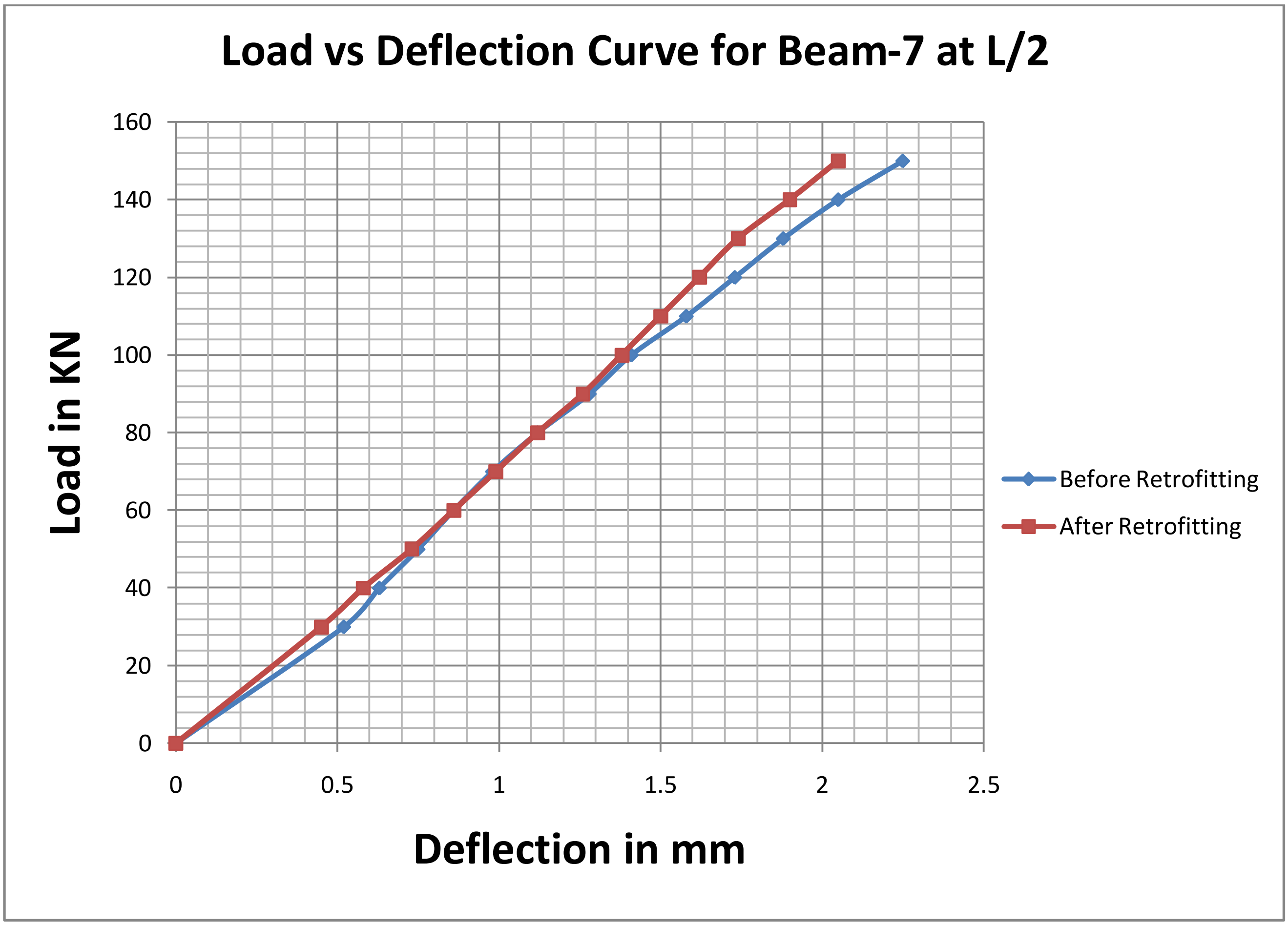


Fig. 4.11 Load vs. Deflection Curve for Beam 6 before and after retrofitting

Here it is observed that how the deflection of the Beam 7 taken at the centre is controlled after retrofitting.

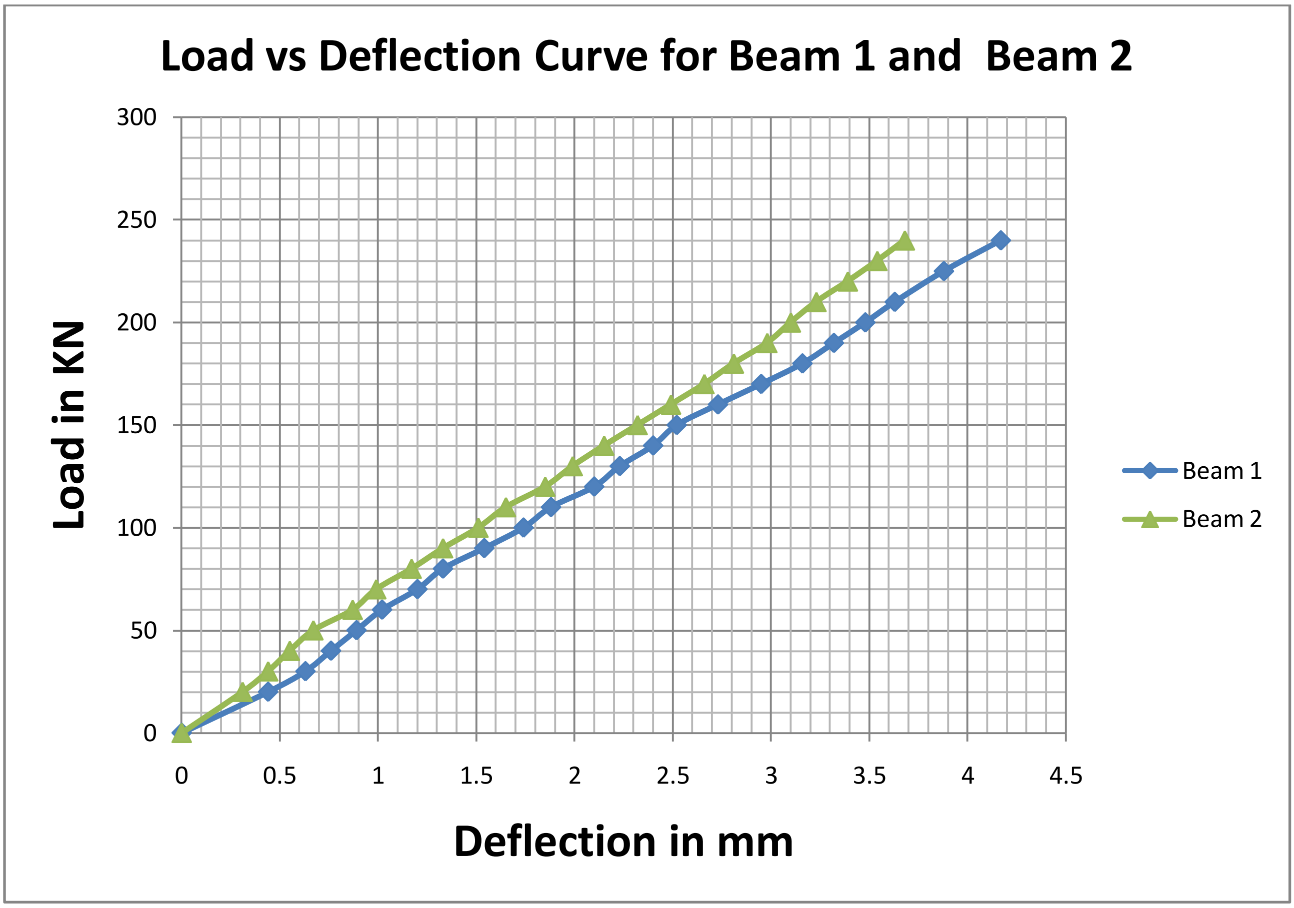


Fig.4.12 Load vs. Deflection Curve for Beam 1 and 2

From this figure it is observed that deflection in case of Beam-2 which has been strengthened with GFRP at the soffit is controlled to a certain extent with respect to the control Beam 1.And the ultimate load has also increased to a certain percentage which has been illustrated in the figure 4.28

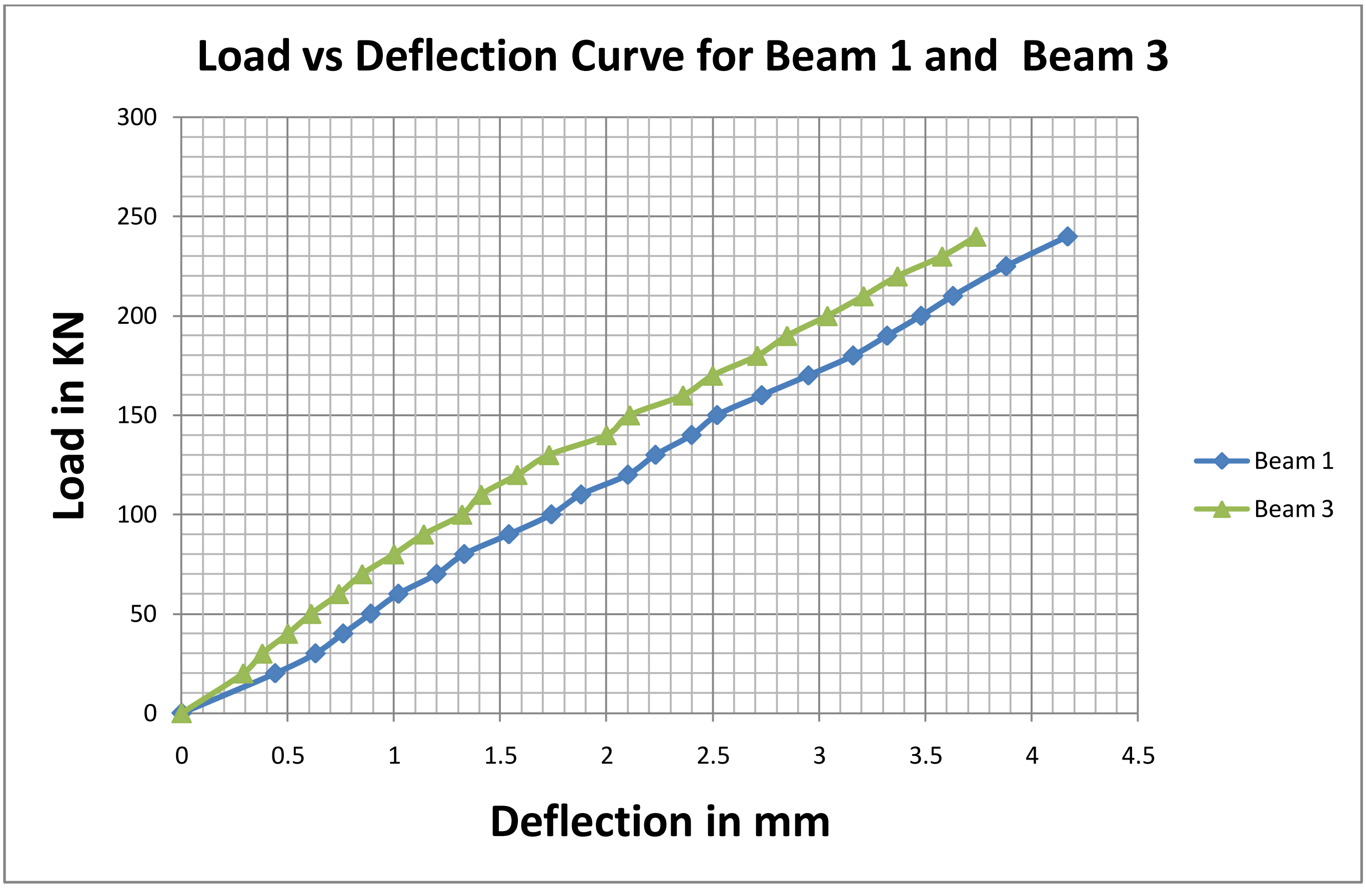


Fig. 4.13 Load vs. Deflection Curve for Beam 1 and Beam 3

From this figure 4.12 it is interpreted that deflection in case of Beam-3 which has been strengthened with GFRP only at the soffit but for the length L/3 to 2L/3 is controlled to a certain extent with respect to the Control Beam 1.And the ultimate load has also increased to a certain percentage which has been illustrated in the figure 4.28.

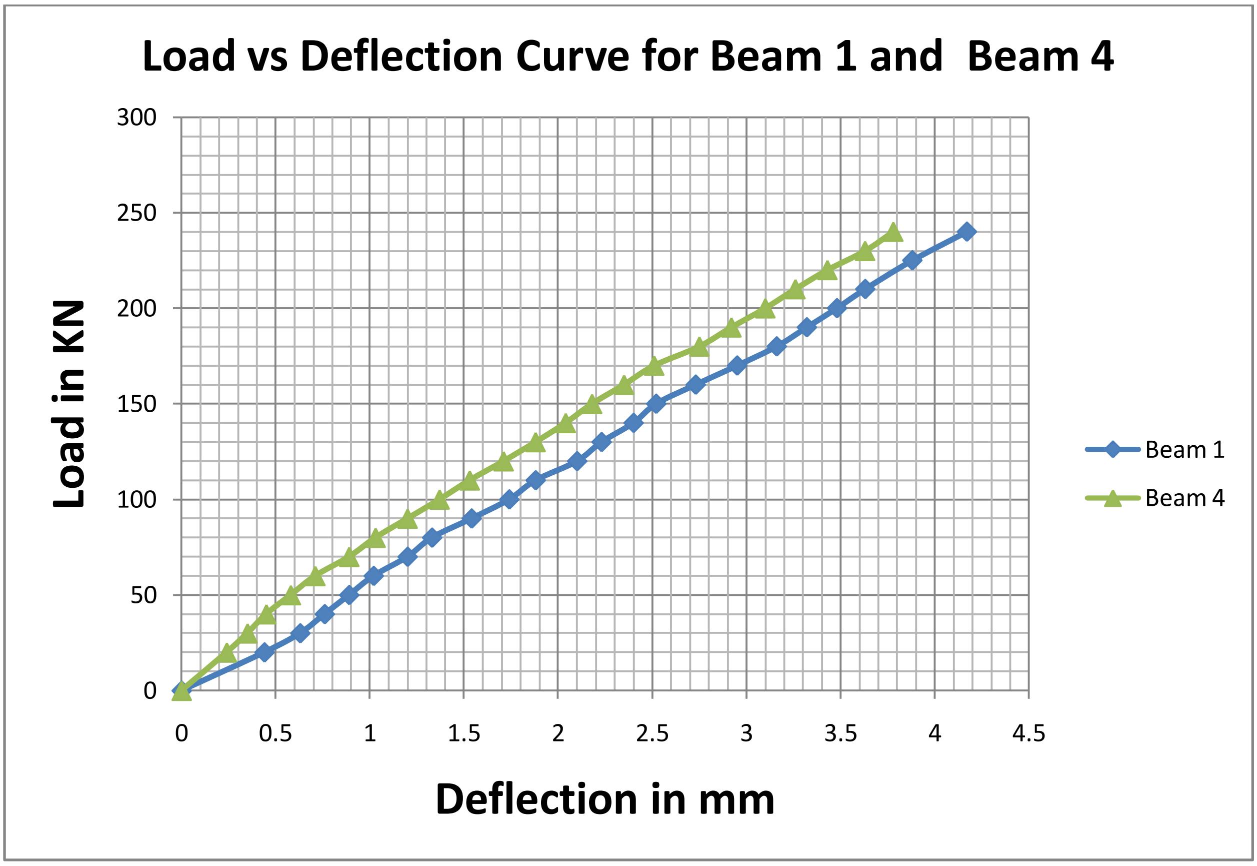


Fig. 4.14 Load vs. Deflection Curve for Beam 1 and Beam 4

It is observed from the fig 4.13 that deflection in case of Beam-4 which has been strengthened in the web part throughout its length with U-Jacketed single layered GFRP is controlled to a certain extent with respect to the Control Beam 1.And the ultimate load has also increased to a certain percentage which has been illustrated in the fig 4.28.

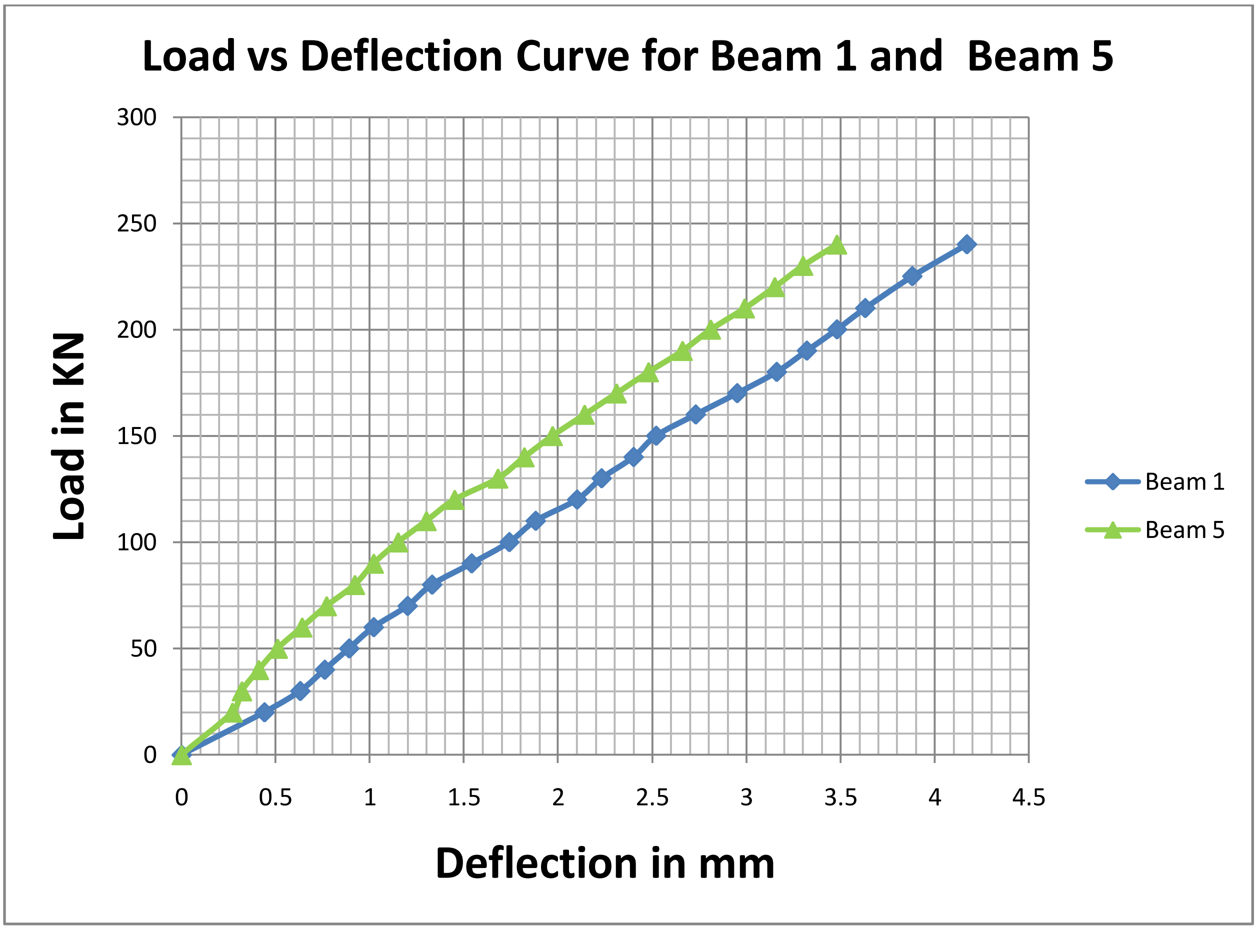


Fig. 4.15 Load vs. Deflection Curve for Beam 1 and Beam 5

From this figure it is studied that deflection in case of Beam-5 which has been strengthened in the web part for a length of 1 m length with U-Jacketed single layered GFRP is controlled to a much greater extent with respect to the Control Beam 1.And the ultimate load has also increased to a certain percentage which has been illustrated in the fig 4.28

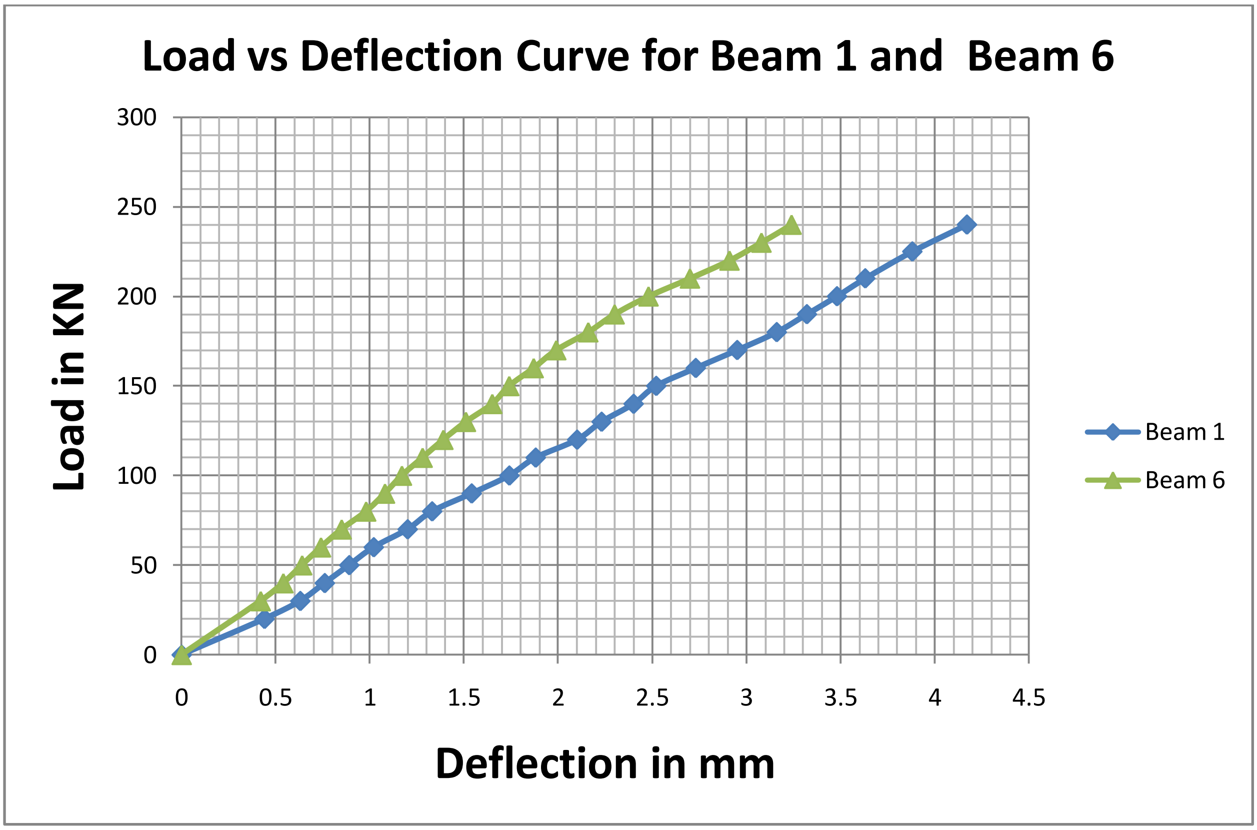


Fig. 4.16 Load vs. Deflection Curve for Beam 1 and Beam 6

From this figure 4.16 it is observed that deflection in case of Beam-6 which has been first cracked till initial hairline cracks appeared and then strengthened in its the web part for a length of 1 m length with U-Jacketed single layered GFRP is controlled to the maximum extent with respect to the Control Beam 1 and other strengthened beams .And the ultimate load has also increased to a certain percentage which has been illustrated in the fig 4.28.

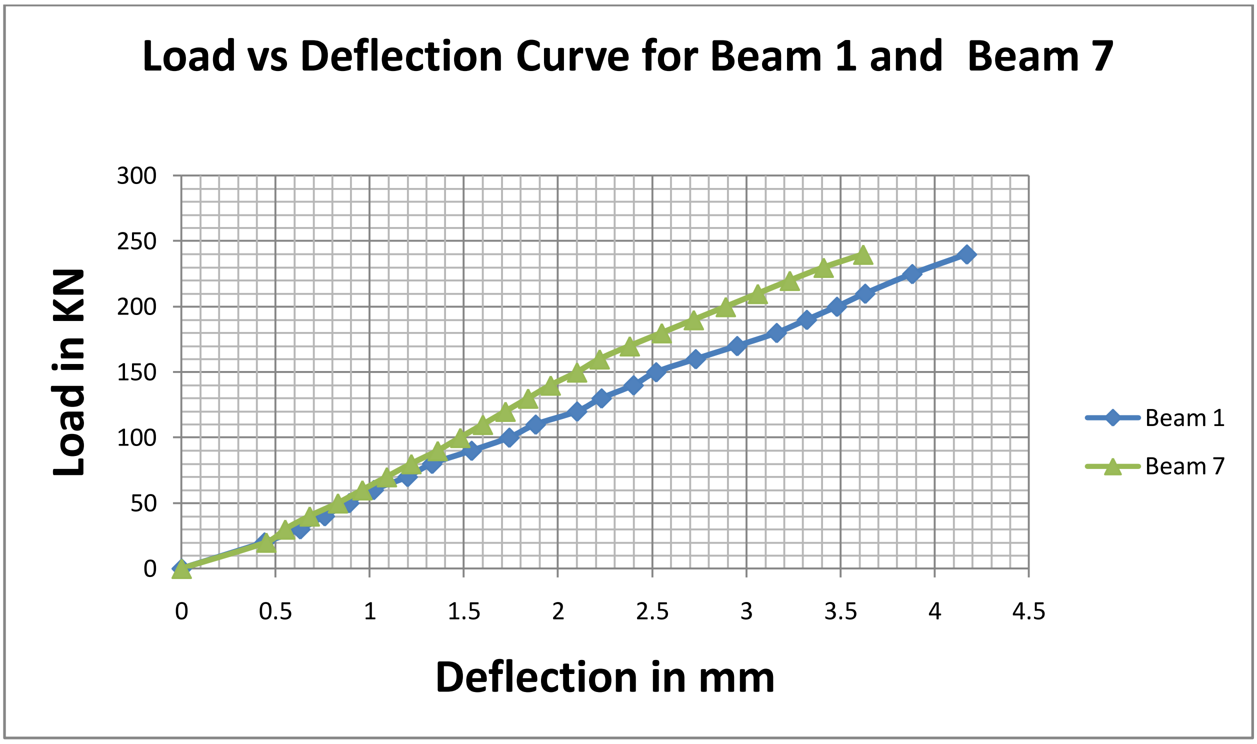


Fig. 4.17 Load vs. Deflection Curve for Beam 1 and Beam 7

From this figure 4.17 it can be interpreted that deflection in case of Beam-7 which has been first cracked till initial hairline cracks appeared and then strengthened in its the web part throughout its length with U-Jacketed single layered GFRP is controlled to the good extent with respect to the Control Beam 1 and but less than Beam-6 .And the ultimate load has also increased to a certain percentage which has been illustrated in the fig 4.28.

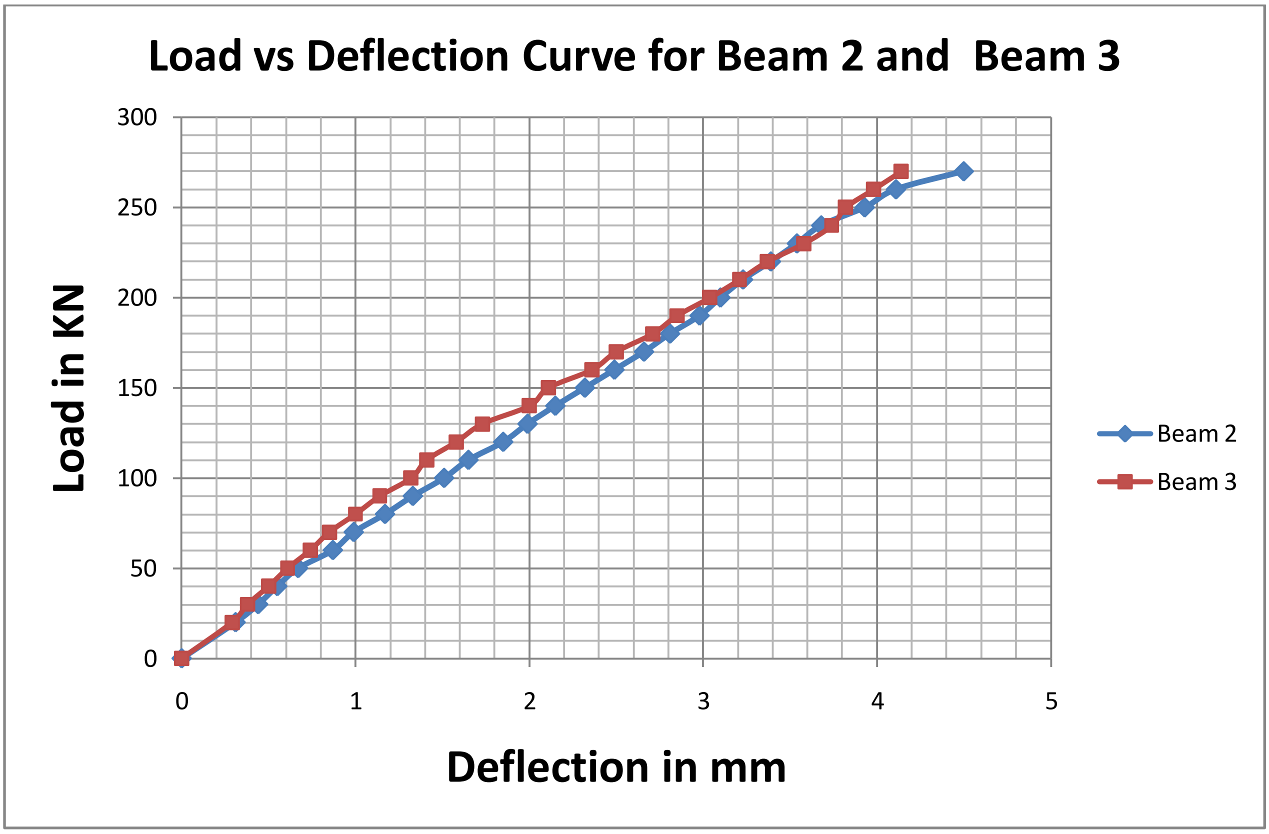


Fig. 4.18 Load vs. Deflection Curve for Beam 2 and Beam 3

Here in the figure 4.18 deflection of the beam 2 and beam 3 has been compared which shows that beam 3 which is strengthened with GFRP only at soffit for a length from L/3 to 2L/3 has less deflection as compared to beam 2 which is strengthened at soffit throughout its length.

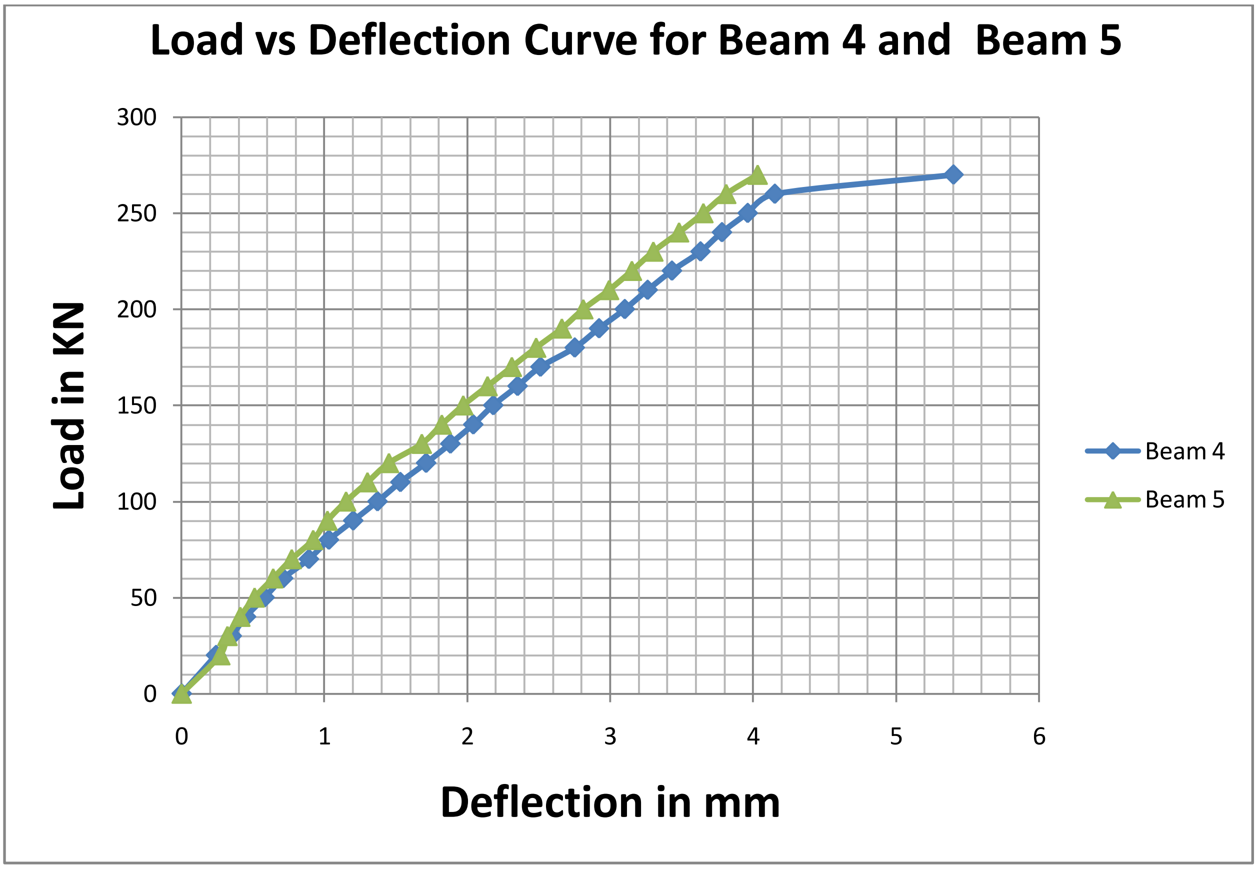


Fig. 4.19 Load vs. Deflection Curve for Beam 4 and Beam 5

Here in the figure 4.19 deflection of the beam 4 and beam 5 has been compared which shows that beam 5 which is strengthened with U-Jacketed single layered GFRP in the web part for a length of 1 m has less deflection as compared to beam 4 which is strengthened with U-Jacketed single layered GFRP in the web part throughout its length

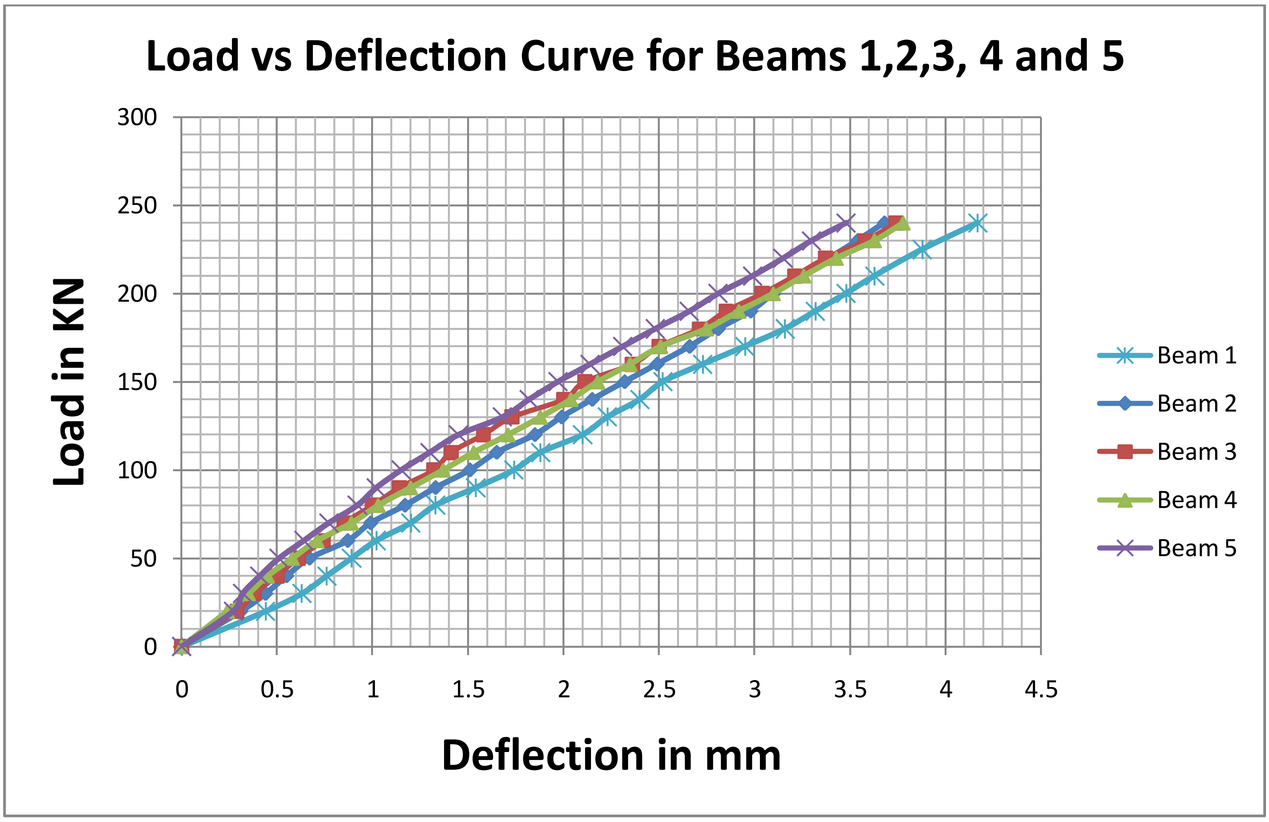


Fig. 4.20 Load vs. Deflection Curve for Beam 1, 2, 3, 4 and Beam 5

Here the all strengthened beam are compared with each other and also with the control Beam 1. And from the figure 4.20 it can be interpreted that beam 5 which is strengthened with U-Jacketed single layered GFRP in the web part for a length of 1 m has minimum deflection as compared to others.

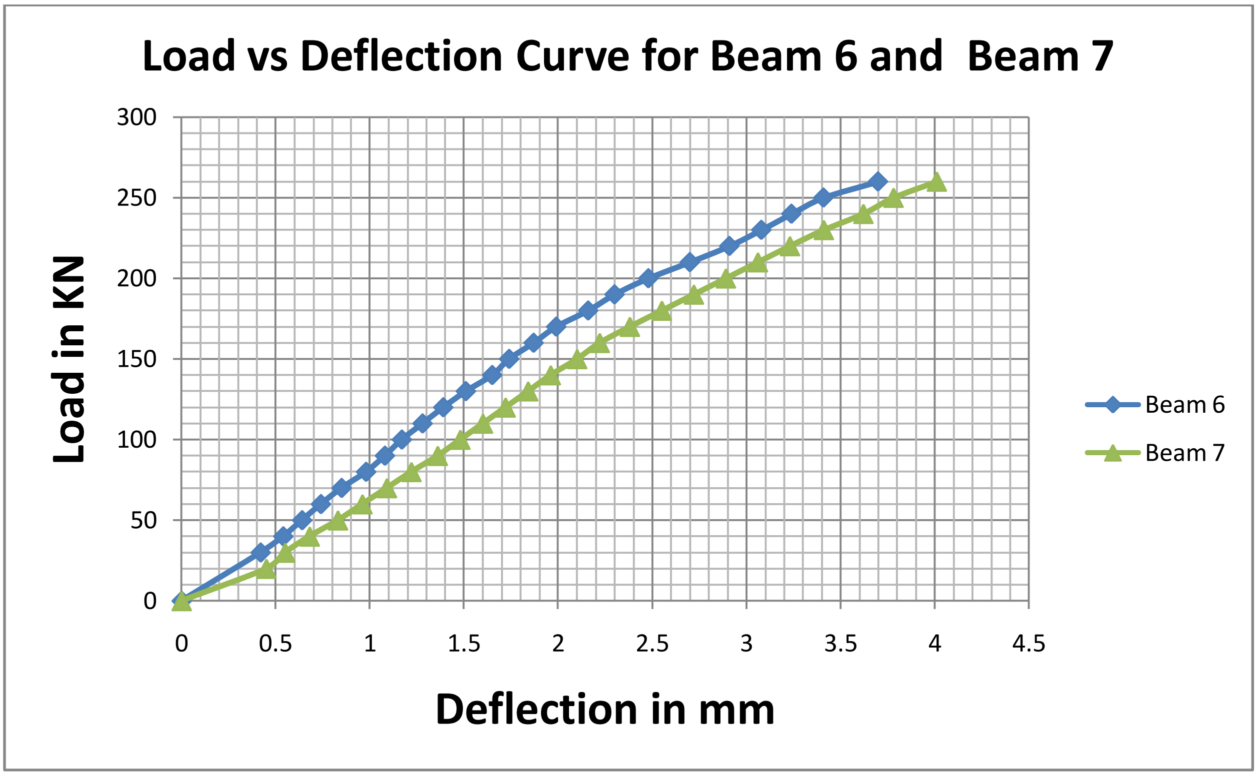


Fig. 4.21 Load vs. Deflection Curve for Beam 6 and Beam 7

Here the beam 6 and beam 7 are compared which are retrofitted after initial hairline cracks are observed. And from the graph we can interpret that deflection in the case of beam 6 is less than that of beam 7.

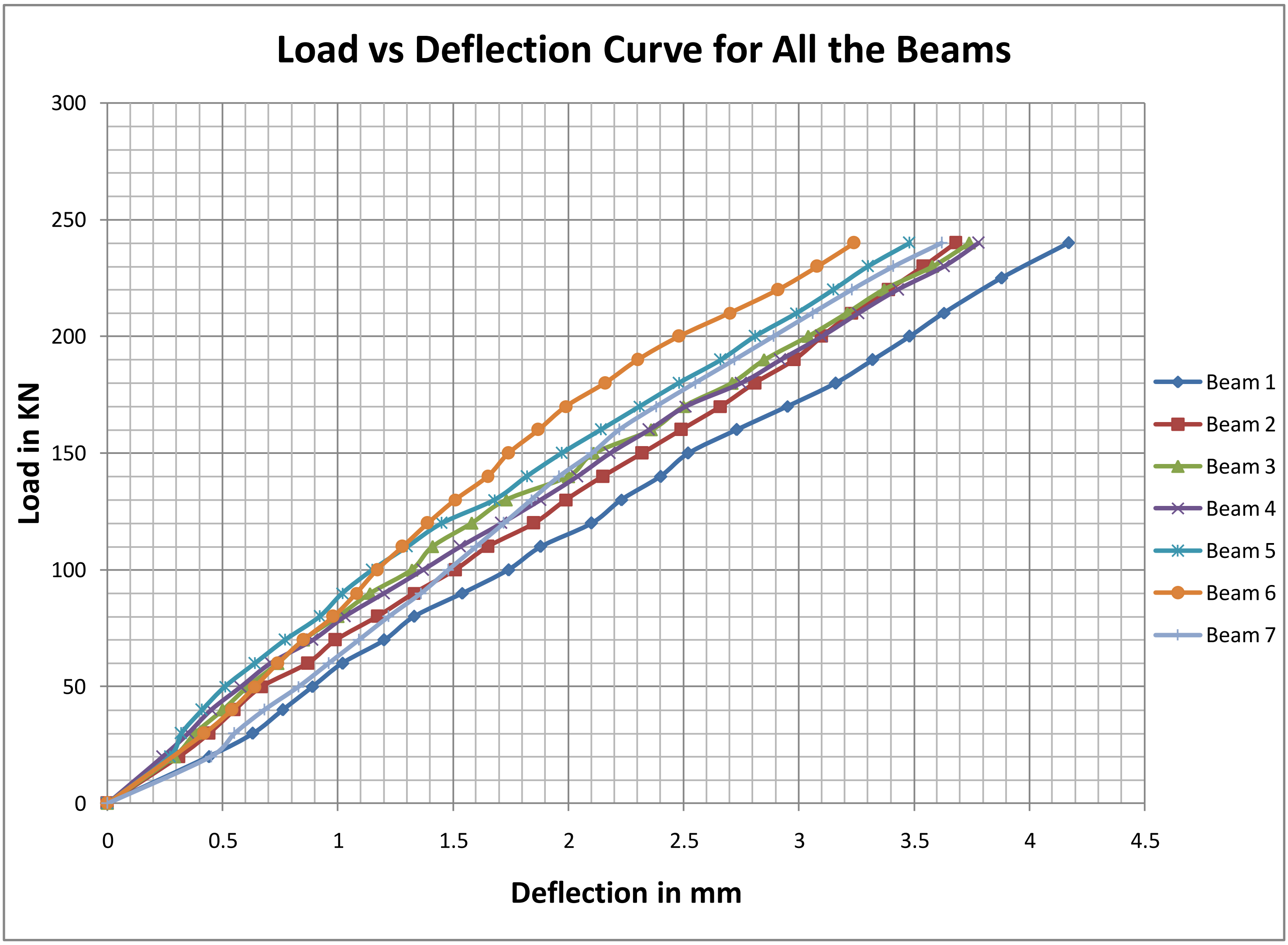


Fig. 4.22 Load vs. Deflection Curve for all the Beams

Here all the beams are compared with respect to their deflection and load data. And it can be interpreted that beam 6 which is retrofitted with U-Jacketed single layered GFRP sheet for a length of 1 m in the middle section of the beam where most of the cracks are occurring has minimum deflection value as compared to others.

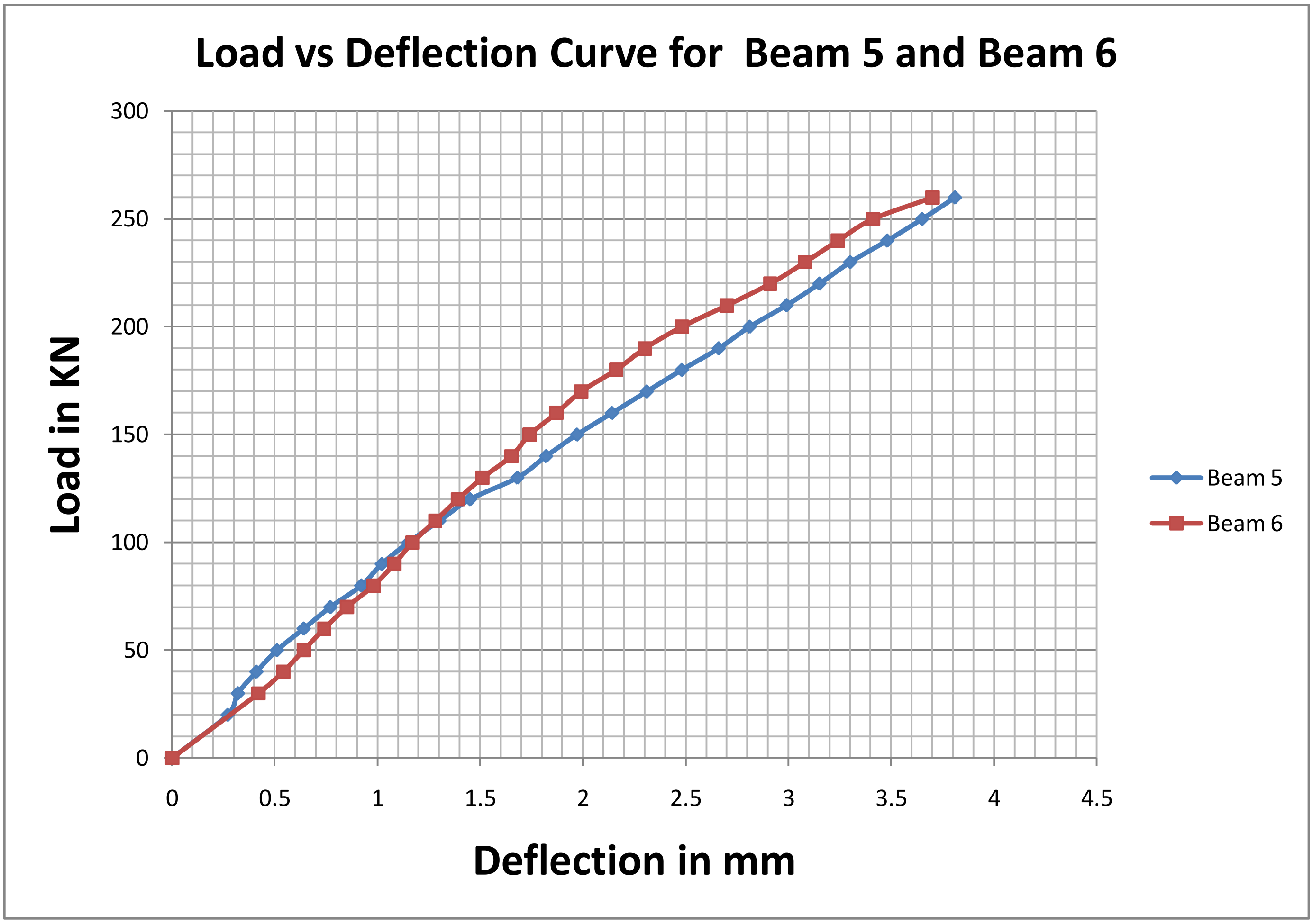


Fig. 4.23 Load vs. Deflection Curve for Beam 5 and Beam 6

Here beam 5 and beam 6 are compared. Both the beams are having same wrapping pattern but beam 5 is strengthened from the starting where as beam 6 is retrofitted with U-Jacketed single layered GFRP sheet for a length of 1 m in the middle section after initial hairline cracks are formed. And from the figure 4.23 it can be interpreted that in the initial stages of loading the retrofitted beam i.e. beam no 6 is showing more deflection but as the load value is increased the retrofitted beam is showing less deflection.

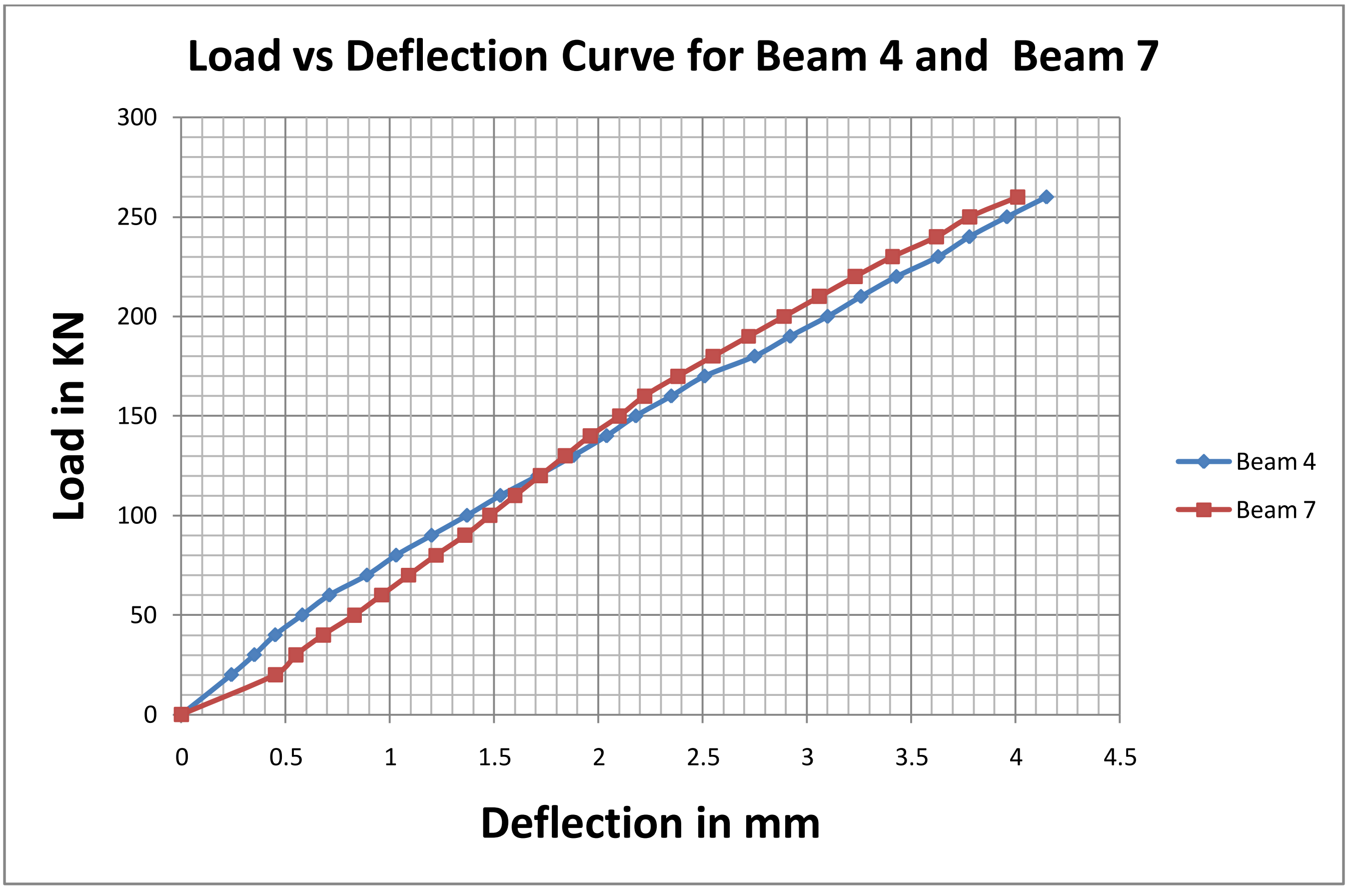


Fig. 4.24 Load vs. Deflection Curve for Beam 4 and Beam 7

Here beam 4 and beam 7 are having same wrapping pattern but beam 4 is strengthened from the starting where as beam 6 is retrofitted with U-Jacketed single layered GFRP sheet throughout its length after initial hairline cracks are formed. And from the figure 4.24 it can be interpreted that in the initial stages of loading the retrofitted beam i.e. beam no 7 is showing more deflection but as the load value is increased the retrofitted beam is showing less deflection.

Following are the values of deflection Control Beam 1 obtained from finite element modelling and IS code method using Matlab program and it is compared with experimental values in the graphs.

Table 4.1 Deflection of Control Beam 1 from different methods

LOAD (in KN)	FEM (in mm)	IS CODE (in mm)	EXPERIMENTAL (in mm)
20	0.294	0.156	0.440
30	0.427	0.338	0.630
40	0.560	0.52	0.760
50	0.693	0.701	0.890
60	0.826	0.882	1.020
70	0.958	1.063	1.200
80	1.091	1.244	1.330
90	1.224	1.425	1.540
100	1.357	1.606	1.740
110	1.490	1.787	1.880
120	1.622	1.968	2.100
130	1.755	2.149	2.230
140	1.888	2.329	2.400
150	2.021	2.510	2.520
160	2.154	2.691	2.730
170	2.2867	2.872	2.950
180	2.419	3.053	3.160
190	2.552	3.234	3.320
200	2.685	3.415	3.480
210	2.818	3.595	3.630
220	2.950	3.776	3.840
230	3.083	3.957	3.970
240	3.216	4.138	4.170

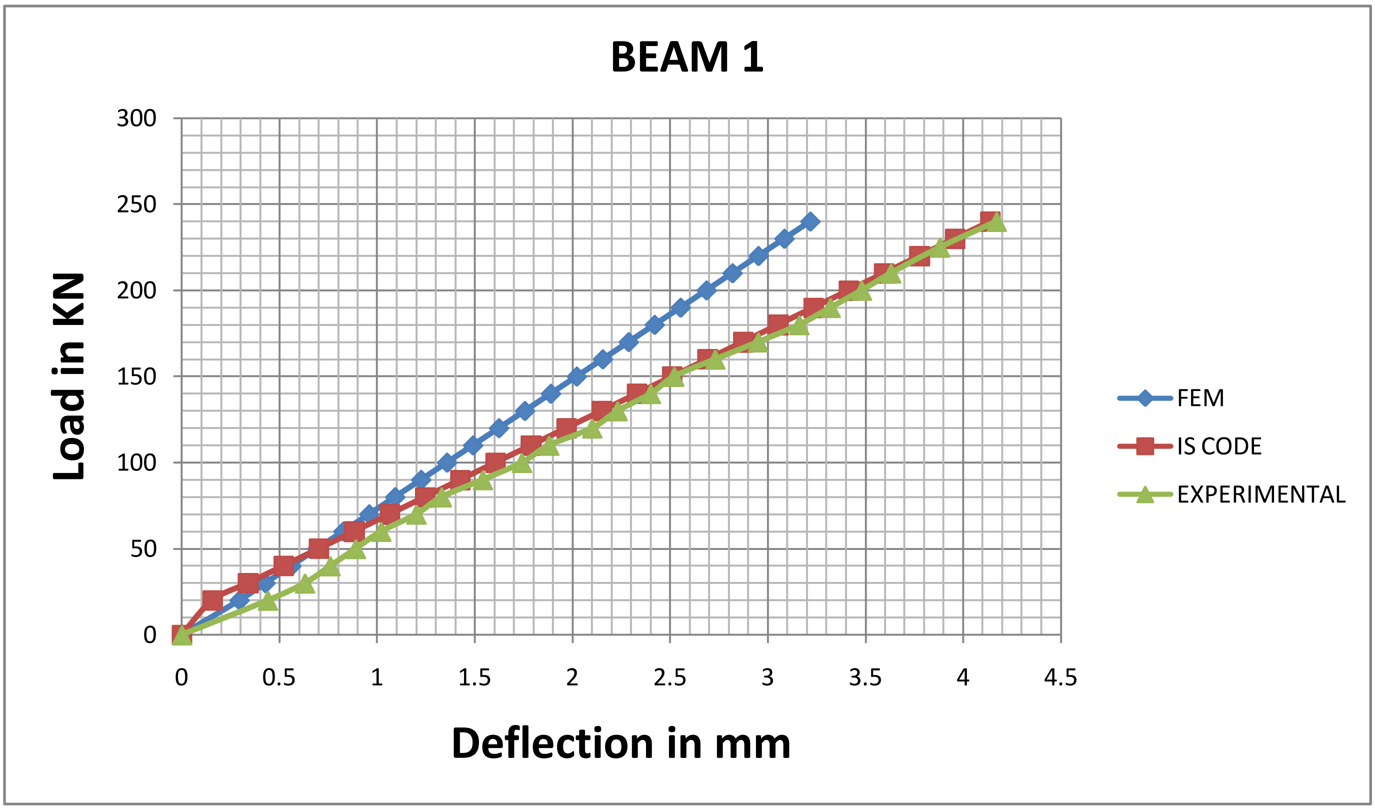


Fig. 4.25Load vs. Deflection Curve for Control Beam 1 using different methods

From the figure 4.25 it is observed that the central deflection of the T-beam obtained from experiment compare well with the results using IS Code method. It is also observed from the figure that at lower loads the values of Finite element model is closer to the experimental values but at higher loads it is varying because here linear finite element modeling have been used to find the deflections.

Following are the values of deflection for Beam 2 obtained from finite element modelling and IS code method using Matlab program and it is compared with experimental values in the graphs

Table 4.2 Deflection of Beam 2 from different methods

LOAD (in KN)	FEM (in mm)	IS code (in mm)	EXPERIMENTAL (in mm)
20	0.289	0.154	.310
30	0.420	0.334	.440
40	0.551	0.513	.550
50	0.682	0.691	.670
60	0.812	0.869	.870
70	0.943	1.047	.990
80	1.074	1.226	1.170
90	1.205	1.404	1.330
100	1.335	1.582	1.510
110	1.466	1.760	1.650
120	1.597	1.938	1.850
130	1.728	2.116	1.990
140	1.858	2.294	2.150
150	1.989	2.472	2.320
160	2.120	2.650	2.490
170	2.251	2.828	2.660
180	2.381	3.006	2.810
190	2.512	3.184	2.980
200	2.643	3.363	3.100
210	2.774	3.541	3.230
220	2.904	3.719	3.390
230	3.035	3.897	3.540
240	3.166	4.075	3.680

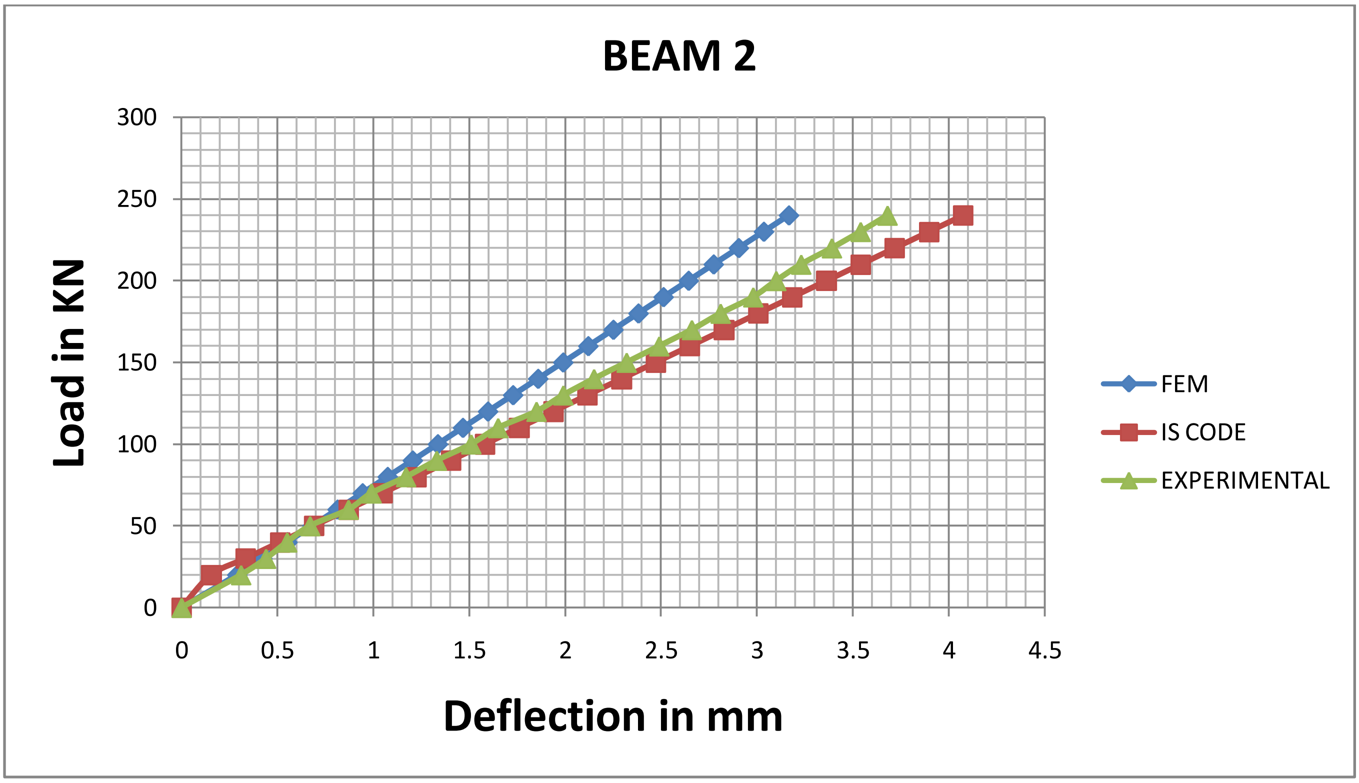


Fig. 4.26 Load vs. Deflection Curve for Beam 2 using different methods

It can be studied from the figure 4.26 that the values of deflection obtained from finite element modeling at lower loads are agreeing with experimental values. But at higher load it is varying as linear finite element modeling is used. Further it can be observed that the experimental values of deflection are agreeing with value of deflection found by IS code method. Here the values of deflection in case of FEM are lower than IS code because in case of FEM total gross section is considered where as in IS code method crack section is considered.

4.4 ULTIMATE LOAD CARRING CAPACITY

The load carrying capacity of the control beams and the strengthen beam are plotted below. It is observed that beam 4 is having the max load carrying capacity.

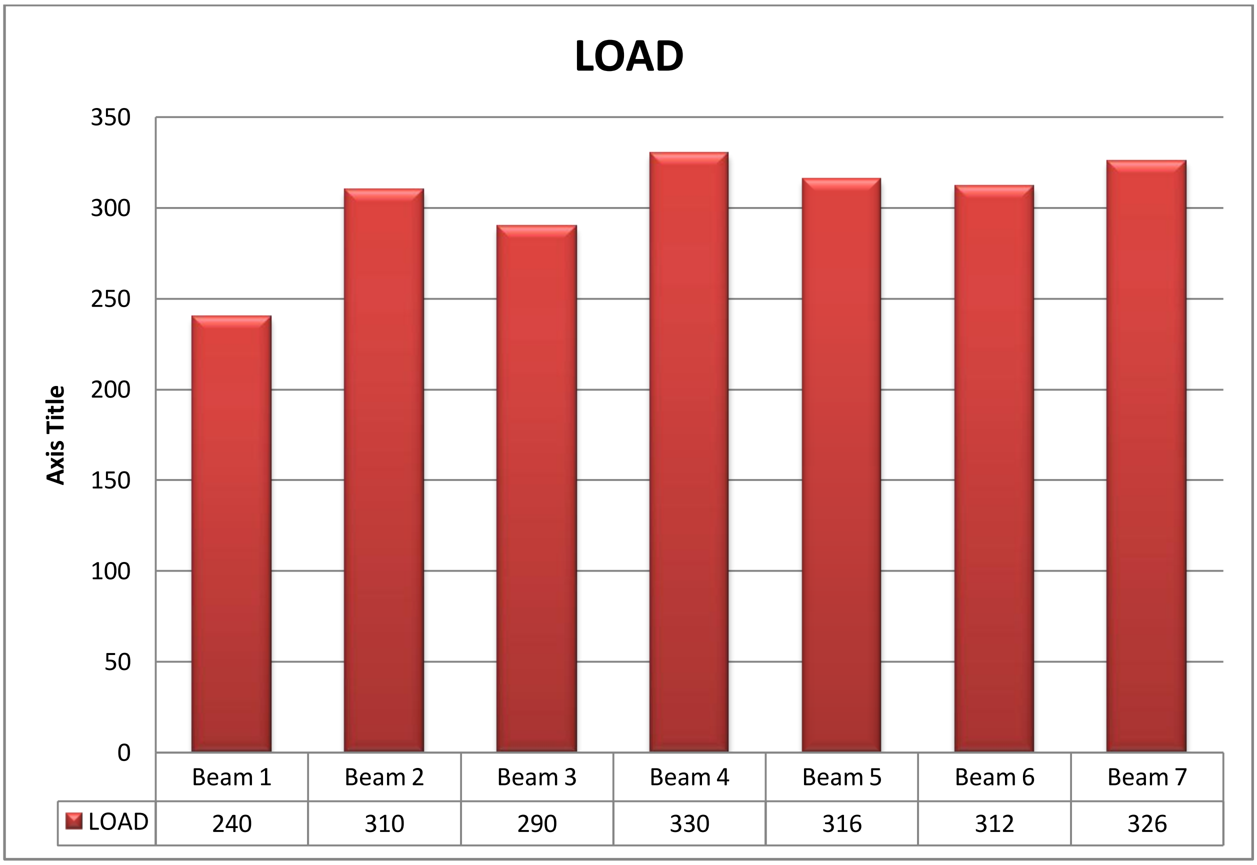


Fig. 4.27 Ultimate Load carrying capacity

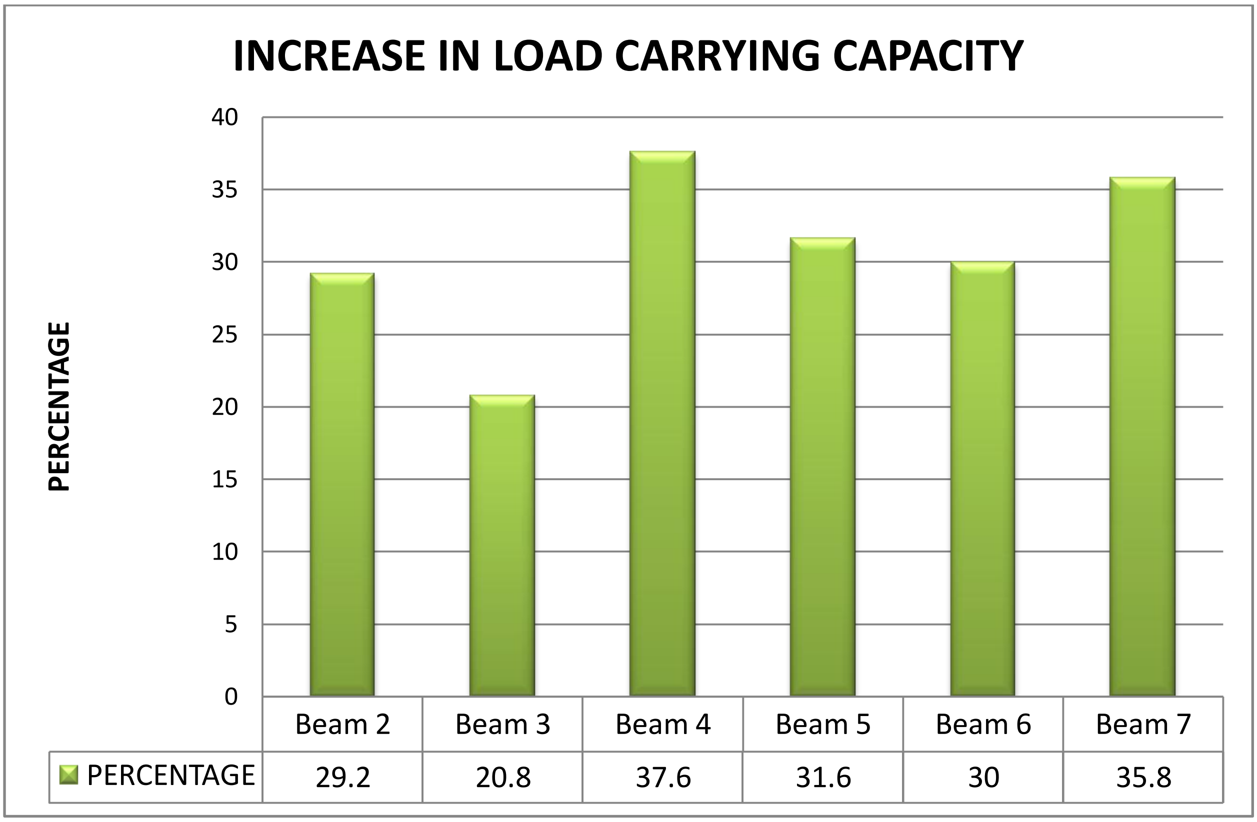


Fig. 4.28 Percentage increase in the Ultimate Carrying capacity w.r.t Control Beam 1

From the above figure we can observe the amount of increase in the flexural strength for each strengthened beam with respect to the Control Beam 1

CHAPTER 5

CONCLUSIONS

5.1 CONCLUSIONS

The present experimental study is done on the flexural behavior of reinforced concrete T-beams strengthened by GFRP sheets. Seven reinforced concrete (RC) T-beams weak in flexure having same reinforcement detailing are casted and tested. From the test results and calculated strength values, the following conclusions are drawn:

1. The ultimate load carrying capacity of all the strengthen beams were enhanced as compared to the Control Beam1.
2. Initial flexural cracks appear for higher loads in case of strengthened beams.

3. The load carrying capacity of the strengthened Beam 4 was found to be maximum of all the beams. It increased up to 37.5 % more than the control beam 1, 6.5% more than strengthened beam 2 and 4.4 % more than the strengthened beam 5.

4. Beam 6 which was retrofitted in the web part only for 1 m length in the center showed minimum deflection values on same loads as compared to other strengthened beams and the control beam.

5. Beam 4 and Beam 7 were giving the best results in terms of load carrying capacity and deflection respectively. And both are having same wrapping pattern of GFRP which is bonded in the web part throughout its length.

6. Flexural strengthening in the web part throughout the length of the beam increases the ultimate load carrying capacity, but it is comparatively close to ultimate load carrying capacity of the beam strengthen at the soffit only so it is economically effective to use strengthen only at the soffit of the beam and the cracks developed were not visible in the former because of the side wrapping so it gives less warning compared to the beams strengthen only at the soffit of the beam

5.2 SCOPE OF THE FUTURE WORK

It promises a great scope for future studies. Following areas are considered for future research:

- ✓ Strengthening of beam weak in shear.
- ✓ Effect on torsional strength due to retrofitting
- ✓ Developing a non linear finite element model for the analysis of the strengthened RC T-Beams using various configuration of FRP strengthening.
- ✓ Variation of beam dimension.
- ✓ Strengthening of T- Beam with different type of FRP (like Carbon fiber reinforced polymer),.

CHAPTER 6

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