STRENGTHENING OF RC CONTINUOUS BEAM USING FRP SHEET

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Abstract:-**Strengthening structures through outside bonding of superior fibre bolstered polymer (FRP) composite is becoming very popular global for the duration of the past decade as it presents a more competitively priced and technically superior alternative to the conventional techniques in many situations as it gives high strength, low weight, corrosion resistance, excessive fatigue resistance, clean and speedy set up and minimal exchange in structural geometry. Although many in-situ RC beams are non-stop in construction, there were very restricted research paintings in the area of FRP strengthening of non-stop beams.**

In the prevailing have a look at an experimental investigation is done to observe the behavior of continuous RC beams under static loading. The beams are strengthened with externally bonded glass fiber reinforced polymer (GFRP) sheets. Different scheme of strengthening were employed. The application consists of fourteen continuous (-span) beams with ordinary dimensions equal to (150×200×2300) mm. The beams are grouped into two collections labeled S1 and S2 and every series have exceptional percentage of metal reinforcement. One beam from every collection (S1 and S2) changed into no longer reinforced and turned into considered as a manage beam, while all different beams from both the series were reinforced in diverse patterns with externally bonded GFRP sheets. The gift study examines the responses of RC non-stop beams, in terms of failure modes, enhancement of load capability and load deflection evaluation. The effects indicate that the flexural strength of RC beams may be considerably expanded by gluing GFRP sheets to the tension face. In addition, the epoxy bonded sheets advanced the cracking behavior of the beams by using delaying the formation of seen cracks and decreasing crack widths at higher load tiers. The experimental effects had been verified via using finite element approach.

KEYWORDS: continuous beam; flexural strengthening; GFRP; premature failure; debonding failure.

1. INTRODUCTION

1.1 GENERAL

A structure is designed for a specific duration and relying on the nature of the shape, its layout life varies. For a domestic building, this design life could be as low as twenty-five years, while for a public constructing, it could be fifty years. Deterioration in concrete systems is a first-rate venture faced through the infrastructure and bridge industries worldwide. The deterioration can be mainly because of environmental results, which incorporates corrosion of metallic, sluggish lack of electricity with ageing, repeated excessive depth loading, version in temperature, freeze-thaw cycles, touch with chemicals and saline water and publicity to extremely-violet radiations. As complete replacement or reconstruction of the structure may be value powerful, strengthening or retrofitting is an powerful manner to strengthen the equal.

The maximum famous strategies for strengthening of RC beams have concerned the usage of external epoxy-bonded metal plates. It has been discovered experimentally that flexural strength of a structural member can increase by way of using this method. Although metallic bonding approach is easy, fee-effective and efficient, it suffers from a severe trouble of decay of bond on the steel and concrete interphase because of corrosion of steel. Other common strengthening approach includes construction of metal jackets that is pretty effective from electricity, stiffness and ductility concerns. However, it increases usual move-sectional dimensions, leading to increase in self-weight of systems and is labour extensive. To get rid of these issues, metallic plate changed into replaced with the aid of corrosion resistant and light-weight FRP Composite plates. FRPCs assist to increase electricity and ductility without immoderate boom in stiffness. Further, such cloth may be designed to satisfy specific requirements by means of adjusting placement of fibers. So concrete participants can now be without problems and correctly reinforced the usage of externally bonded FRP composites.

By wrapping FRP sheets, retrofitting of concrete systems offer a extra reasonably-priced and technically superior alternative to the traditional strategies in many situations as it offers excessive electricity, low weight, corrosion resistance, high fatigue resistance, easy and fast installation and minimal exchange in structural geometry. FRP structures can also be used in regions with limited get right of entry to where traditional techniques might be impractical. However, due to loss of the right knowledge on structural conduct of concrete structures, using these materials for retrofitting the present concrete systems can not attain as much as the expectancy. Successful retrofitting of concrete structures with FRP wishes an intensive knowledge at the subject and to be had consumer-friendly technologies/ specific guidelines.

Beams are the essential structural individuals subjected to bending, torsion and shear in all form of structures. Similarly, columns also are used as numerous important factors subjected to axial load combined with/without bending and are utilized in all sort of structures.

Therefore, tremendous studies works are being done during world on retrofitting of concrete beams and columns with externally bonded FRP composites. Several investigators took up concrete beams and columns retrofitted with carbon fibre strengthened polymer (CFRP)/ glass fibre bolstered polymer (GFRP) composites in order to study the enhancement of strength and ductility, sturdiness, impact of confinement, instruction of layout guidelines and experimental investigations of those contributors.

1.2 FLEXURAL STRENGHTENING OF BEAMS

For flexural strengthening, there are numerous techniques which includes: segment expansion, metallic plate bonding, external publish tensioning technique, near-floor established (NSM) gadget and externally bonded (EB) gadget. While many techniques of strengthening structures are available, strengthening systems via outside bonding of advanced fibre-reinforced polymer composite (FRP) has grow to be very popular worldwide. During the past decade, their application on this subject has been growing because of the well-known blessings of FRP composites over different materials. Consequently, a awesome quantity of research, each experimental and theoretical, has been carried out at the behaviour of FRP-bolstered reinforced concrete (RC) systems. In this regard, the evolving generation of using carbon-bonded fibre- reinforced polymers (CFRP) for strengthening of RC beams has attracted an awful lot interest in current years.

1.3 ADVANTAGES OF FRP

Some of the main advantages of FRP can be listed below:
Low weight: The FRP is lots much less dense and therefore lighter than the equal extent of metallic. The decrease weight of FRP makes installation and managing considerably less complicated than metallic. These properties are specially essential when set up is finished in cramped locations. Other works like works on soffits of bridges and building floor slabs are achieved from man-get entry to structures in place of from complete scaffolding. The use of fibre composites does no longer extensively growth the load of the shape or the dimensions of the member. And because of their light weight, the transport of FRP substances has minimal environmental impact.
Mechanical energy: FRP can offer a most cloth stiffness to density ratio of 3.Five to five instances that of aluminum or metallic. FRP is so sturdy and stiff for its weight, it can out-perform the alternative materials.

Formability: The fabric can absorb irregularities inside the shape of the concrete surface. It can be molded to nearly any desired form. We can create or reproduction maximum shapes effortlessly.

Chemical resistance: FRP is minimally reactive, making it perfect as a defensive overlaying for surfaces in which chemical.

Joints: Laps and joints are not required.

Corrosion resistance: Unlike metal, FRP does not rust away and it can be used to make long-lasting structures.

Low protection: Once FRP is mounted, it requires minimum protection. The materials fibres and resins are long lasting if successfully detailed, and require little upkeep. If they may be broken in service, it's far exceedingly easy to repair them, by way of adding an additional layer.

Long life: It has high resistance to fatigue and has proven first-rate durability over the past 50 years.

Easy to use: The application of FRP plate or sheet material is like making use of wallpaper; as soon as it's been rolled on carefully to do away with entrapped air and extra adhesive it is able to be left unsupported. Fibre composite substances are available in very long lengths even as steel plate is generally restricted to 6 m.

These various factors in combination cause a considerably easier and quicker strengthening process than whilst the use of metallic plate.

1.4 SUITABILITY OF FRP FOR USES IN STRUCTURAL ENGINEERING

The power homes of FRPs together make up one of the primary motives for which civil engineers pick out them inside the design of systems. A material's electricity is ruled through its capability to sustain a load with out excessive deformation or failure. When an FRP specimen is examined in axial anxiety, the applied pressure in line with unit cross-sectional place (strain) is proportional to the ratio of exchange in a specimen's duration to its unique duration (stress). When the carried out load is eliminated, FRP returns to its unique form or period. In different phrases, FRP responds linear-elastically to axial stress. The response of FRP to axial compression is reliant at the relative percentage in quantity of fibres, the residences of the fibre and resin, and the interface bond strength. FRP composite compression failure happens while the fibres showcase excessive (frequently unexpected and dramatic) lateral or sides-manner deflection called fibre buckling. FRP's response to transverse tensile stress may be very lots dependent on the residences of the fibre and matrix, the interplay between the fibre and matrix, and the electricity of the fibre- matrix interface. Generally, however, tensile strength in this path is very bad. Shear stress is brought on in the plane of a place while outside loads generally tend to motive segments of a body to slip over each other. The shear power of FRP is tough to quantify. Generally, failure will arise within the matrix material parallel to the fibres. Among FRP's excessive strength homes, the maximum applicable features consist of notable sturdiness and corrosion resistance. Furthermore, their high energy-to-weight ratio is of sizeable benefit; a member composed of FRP can guide larger stay masses on account that its lifeless weight does not make contributions considerably to the masses that it ought to endure. Other capabilities consist of ease of installation, versatility, anti-seismic behaviour, electromagnetic neutrality, high-quality fatigue behaviour, and fire resistance. However, like maximum structural materials, FRPs have a few drawbacks that would create some hesitancy in civil engineers to apply it for all applications: excessive value, brittle behaviour, susceptibility to deformation beneath lengthy-term loads, UV degradation, photo-degradation (from exposure to mild), temperature and moisture consequences, loss of design codes, and most importantly, lack of understanding.

1.5 APPLICATIONS OF FRP COMPOSITES IN CONSTRUCTION

There are three broad divisions into which applications of FRP in civil engineering may be categorized: applications for brand new creation, restore and rehabilitation applications, and architectural programs. FRPs had been used widely by way of civil engineers inside the design of latest production. Structures which includes bridges and columns constructed completely out of FRP composites have verified notable sturdiness, and powerful resistance to results of environmental publicity. Pre-stressing tendons, reinforcing bars, grid reinforcement and dowels are all examples of the numerous diverse packages of FRP in new systems. One of the maximum commonplace makes use of for FRP includes the restore and rehabilitation of damaged or deteriorating structures. Several companies the world over are beginning to wrap damaged bridge piers to prevent crumble and steel-bolstered columns to enhance the structural integrity and to save you buckling of the reinforcement. Architects have additionally found the numerous packages for which FRP can be used. These include structures which include siding/cladding, roofing, flooring and walls.

1.6 DISADVANTAGES OF FRP

The major downside of externally strengthening structures with fibre composite substances is the threat of hearth, vandalism or accidental harm, except the strengthening is included. A unique concern for bridges over roads is the hazard of soffit reinforcement being hit by over-height cars.

A perceived downside of the usage of FRP for strengthening is the exceptionally excessive fee of the materials. However, comparisons have to be made on the idea of the whole strengthening workout; in positive instances the charges can be less than that of metallic plate bonding. A downside within the eyes of many clients might be the lack of enjoy of the techniques and definitely qualified body of workers to perform the paintings. Finally, a massive drawback is the shortage of general design requirements.

2. LITERATURE

This chapter affords a overview of literature on strengthening of RC concrete beams. This evaluation incorporates of literature on bolstered beam below kinds of support situation i.E. Definitely supported and constantly supported.

2.1 SIMPLY SUPPORTED BEAM

Grace et al. (1999) investigated the behaviour of RC beams bolstered with CFRP and GFRP sheets and laminates. They studied the influence of the range of layers, epoxy sorts, and strengthening pattern at the response of the beams. They found that each one beams skilled brittle failure, with considerable enhancement in electricity, therefore requiring a better thing of safety in design.

Experimental investigations, theoretical calculations and numerical simulations showed that strengthening the reinforced concrete beams with externally bonded CFRP sheets within the anxiety area notably expanded the electricity 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 additionally changed the behaviour of those beams under load and failure sample. Most regularly the bolstered beams failed in a brittle way, mainly due to the lack of connection between the composite cloth and the concrete. The impact of the surface education of the concrete, adhesive type, and concrete electricity on the general bond electricity is studied as well as traits of pressure transfer from the plate to concrete. They concluded that the surface education at the side of along side soundness of concrete should have an effect on the last bond power. Thereafter, Study on de-bonding issues in concrete beams externally bolstered with FRP composites are done by using many researchers.

Many investigators used externally bonded FRP composites to enhance the flexural strength of reinforced concrete members. To evaluate the flexural overall performance of the reinforced participants, it's miles vital to examine flexural stiffness of FRP bolstered members at different tiers, including pre-cracking, put up-cracking and submit-yielding. However, simplest few studied are centered on the strengthened concrete members bolstered underneath pre- loading or pre-cracking (Arduni & Nanni, 1997).

In another studies**, Hee Sun Kim (2011)** carried on experimental studies of 14 reinforced concrete (RC) beams retrofitted with new hybrid fibre bolstered polymer (FRP) system consisting carbon FRP (CFRP) and glass FRP (GFRP). The objective of this observe was to look at impact of hybrid FRPs on structural conduct of retrofitted RC beams and to analyze if exclusive sequences of CFRP and GFRP sheets of the hybrid FRPs have influences on development of strengthening RC beams. The beams are loaded with extraordinary magnitudes prior to retrofitting so as to investigate the effect of preliminary loading on the flexural behavior of the retrofitted beam. The predominant check variables are sequences of attaching hybrid FRP layers and magnitudes of preloads. Under loaded situation, beams are retrofitted with two or 3 layers of hybrid FRPs, then the weight will increase till the beams attain failure. Test effects finish that strengthening results of hybrid FRPs on ductility and stiffness of RC beams depend on orders of FRP layers.

2.2 CONTINUOUS BEAM

Although several research studies had been conducted on the strengthening of honestly supported strengthened concrete beams using outside plates, there is very less mentioned work on the behavior of bolstered non-stop beams. Moreover, most layout tips were developed for without a doubt supported beams with external FRP laminates. A crucial literature evaluate revealed that a minimal amount of studies paintings had been executed for addressing the possibility of strengthening the terrible second place of non-stop beam using FRP substances.

Grace et al., (2001) investigated the experimental overall performance of CFRP strips used for flexural strengthening within the terrible moment vicinity of a full-scale bolstered concrete beam. They considered classes of beams (I and II) for flexural strengthening. Category I beams have been designed to fail in shear and Category II beams have been designed to fail in flexure. Five complete scale concrete beams of each category have been tested. It turned into found that Category I beams failed by way of diagonal cracking with nearby debonding at the top of the beams, meanwhile Category II beams failed via delamination on the interface of the CFRP strips and the concrete floor, both with and without concrete-cover failure via way shear/tension delamination. When the beams failed, the CFRP strips were not harassed to their maximum capacity, which led to ductile screw ups in all of the beams. The maximum growth of load-sporting capacity because of strengthening turned into discovered to be 29% for Category I beams, and forty% for Category II beams with admire to corresponding manage beams.

More lately, **El-Refaie et al., (2003)** examined 5 reinforced concrete continuous beams bolstered in flexure with outside CFRP laminates. All beams had the same geometrical dimensions and internal metal reinforcement. The essential parameters tested were the location and form of the CFRP laminates. Three of the beams have been reinforced using one-of-a-kind lay-up arrangements of CFRP reinforcement, and one turned into reinforced the use of CFRP sheets. The overall performance of the CFRP reinforced beams was in comparison with a non- bolstered reference beam. It became determined that, peeling failure was the most important failure mode for all of the reinforced tested beams. It was observed that the longitudinal elastic shear stresses on the adhesive/concrete interface calculated at beam failure were close to the limiting price

advocated in (Concrete Society Technical Report fifty five, 2000). They additionally found that, bolstered beams at each sagging and hogging zone produced the highest load capacity.

Ashour et al., (2004) tested 16 strengthened concrete (RC) non-stop beams with exceptional preparations of inner metallic bars and outside CFRP laminates. All take a look at specimens had the identical geometrical dimensions and were categorised into three companies consistent with the amount of inner metal reinforcement. Each institution protected one non-bolstered control beam designed to fail in flexure. Three failure modes had been determined, particularly laminate rupture, laminate separation and peeling failure of the concrete cover attached to the composite laminate. The ductility of all strengthened beams became reduced in comparison with their respective reference beam. Additionally, simplified methods for estimating the flexural load capability and the interface shear stresses between the adhesive and the concrete cloth were offered. As in previous studies, they found that growing the CFRP sheet duration a good way to cowl the whole negative or superb moment zones did no longer save you peeling failure of the CFRP laminates.

Finally, **Majid Mohammed Ali Kadhim (2011)** centered on the conduct of the high power concrete continuous beam strengthened with carbon fibre-bolstered polymer (CFRP) sheet with one-of-a-kind CFRP sheet lengths. Three full-scale non-stop beams are analyzed under two points load, and the facts of evaluation are compared with the experimental information provided through other researchers. ANSYS program is used and the consequences acquired from analysis give excellent settlement with experimental records with admire to load–deflection curve, final power, and the crack styles. The period of CFRP sheet is changed within the terrible and high quality regions and the results showed that the closing electricity of the beam was reached while the price of Lsheet/Lspan reaches 1.0, and whilst the fee decreases, the remaining energy of beam also decreases a bit (1.Four%), but while it decreases much less than 0.6, the last electricity additionally decreases lots (15%).

From the above facts, it's miles, thus, clean that there lies a huge scope of studies inside the field of retrofitting of RC non-stop beam. Although a first-rate deal of studies has been performed on certainly supported reinforced concrete (RC) beams reinforced with Fibre Reinforced Polymer composites (FRP), a few works has been focused on non-stop beams.

2.3 OBJECTIVE AND SCOPE OF THE PRESENT WORK

The goal of this work is to carry out the research of externally bonded RC non-stop beams using FRP sheet.

In the present work, conduct of RC non-stop rectangular beams reinforced with externally bonded GFRP is experimentally studied. The beams are grouped into two series categorized S1 and S2. Each series have one-of-a-kind longitudinal and transverse metallic reinforcement ratios. All beams have the equal geometrical dimensions. These beams are tested up to failure by applying two factors loading to assess the enhancement of flexural power because of strengthening. A finite element version has been developed to observe the response of strengthened beams.

3. EXPERIMENTAL STUDY

The experimental look at consists of casting of fourteen huge scale non-stop (two-span) square bolstered concrete beams. All the beams susceptible in flexure are casted and tested to failure. The beams had been grouped into two series categorized S1 and S2. Each series had one of kind longitudinal and transverse steel reinforcement ratios which are noted in Table 3.6 and Table three.7 for S1 and S2 respectively. Beams geometry in addition to the loading and assist arrangements are illustrated in Figure 3.6. All beams had the identical geometrical dimensions: 150 mm wide × two hundred mm deep × 2300 mm lengthy.

One beam from each series (S1 and S2) turned into no longer reinforced and became considered as a manipulate beam, while all other beams from both the collection had been reinforced with externally bonded GFRP sheets. Experimental information on load, deflection and failure modes of each of the beams are acquired. The exchange in load carrying ability and failure mode of the beams are investigated for one-of-a-kind kinds of strengthening sample.

3.1 CASTING OF SPECIMEN

For undertaking test, the percentage of **1: 1.67: 3.33** is taken for cement, exceptional combination and direction combination. The blending is carried out by the use of concrete combination. The beams are cured for 28 days. For each beam six concrete dice specimens have been made at the time of casting and were saved for curing. The uniaxial compressive checks on produced concrete (one hundred fifty × 150 × a hundred and fifty mm concrete cube) had been carried out and the average concrete compressive electricity (fcu) after 28 days for each beam is shown in Table 3.5 and Table 3.6.

Table 3.1 Design Mix Proportions

Description	Cement	Sand (Fine Aggregate)	Course Aggregate	Water
Mix Proportion (by weight)	1	1.67	3.33	0.55
Quantities of materials (Kg/m^3)	368.42	533.98	1231.147	191.58

3.1.1 MATERIALS FOR CASTING

3.1.1.1 CEMENT

Portland Slag Cement (PSC) (Brand: Konark) 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.

(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

3.1.1.2 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.

3.1.1.3 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.

3.1.1.4 WATER

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

3.1.1.5 REINFORCING STEEL

All the beams were grouped into two collection categorized S1 and S2. Each series had one of kind longitudinal and transverse metal reinforcement ratios which can be noted in Table three.6 and Table 3.7.

Series S1 beams are reinforced with two eight mm diameter at the lowest, 12 mm diameter bars as top reinforcement all through the period and two 10 mm diameter bars at top anxiety region. To fortify the beam in shear, special diameter bars is used for stirrups, 10 mm diameter is used in the shear quarter of intermediate assist and 8mm diameter is used within the area of stop assist. The diameter version is given due to higher shear pressure in intermediate or non-stop help than give up guide. Series S2 beams have been bolstered with excessive-yield Strength Deformed bars of 10 mm diameter at the lowest and two 10 mm diameter bars at top anxiety region, 6 mm bars were used as hanger bars, closed stirrups of 8 mm diameter excessive-yield Strength Deformed bars at 100 mm centres were furnished to save you shear failure.

Three bars of every diameter rods had been examined in tensile and the measured common yield power is averaged and shown in Table 3.3. The modulus of elasticity of steel bars was 2 × 105 MPa.

Table 3.2 Tensile Strength of the bars

Diameter of the reinforcement (mm)	Tensile strength (MPa)
8	523
10	429
12	578

3.1.2 DETAILING OF REINFORCEMENT

For the same series of continuous reinforced concrete beams, same arrangement for flexure and shear reinforcement is made.

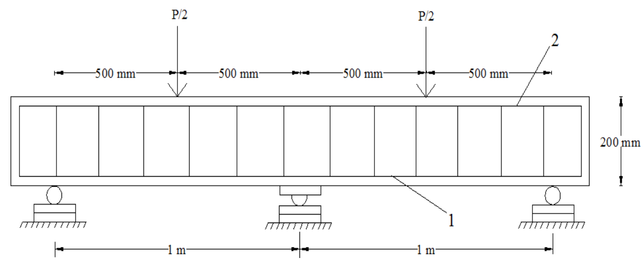


Figure 3.1 Detailing of reinforcement 1, 2 – top and bottom steel reinforcement

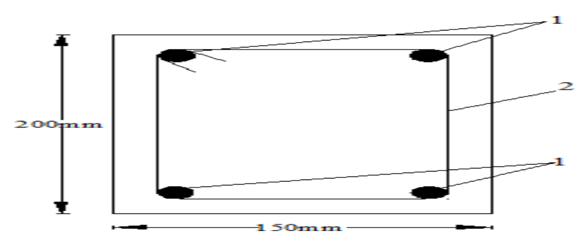


Figure 3.2 Cross section: 1 – Longitudinal rebars, 2 – close stirrups

3.1.3 FORM WORK

Figure 3.3 Steel Frames Used For Casting of Beam

3.1.4 MIXING OF CONCRETE

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

3.1.5 COMPACTION

Needle vibrator become used for correct Compaction and care is taken to avoid displacement of the reinforcement cage inside the form paintings. Then the surface of the concrete is leveled and smoothened by means of metallic trowel and timber go with the flow.

3.1.6 CURING OF CONCRETE

Curing is achieved to prevent the loss of water that is important for the manner of hydration and therefore for hardening. Here curing is accomplished by means of spraying water at the jute baggage unfold over the surface for a duration of 28 days.

3.2 STRENGTHENING OF BEAMS

At the time of bonding of fiber, the concrete floor is made hard using a rough sand paper texture and then wiped clean with an air blower to take away all dirt and debris. The fabric are cut in keeping with the size and after that the epoxy resin is blended in accordance with producer's instructions. The blending is accomplished in a plastic box (one hundred parts through weight of Araldite LY 556 to ten parts by way of weight of Hardener HY 951). After the uniform blending, the epoxy resin is implemented to the concrete floor. Then the GFRP sheet is positioned on top of epoxy resin coating and the resin is squeezed through the roving of the cloth with the roller. Air bubbles entrapped at the epoxy/concrete or epoxy/cloth interface are eliminated. This operation is done at room temperature. Concrete beams reinforced with glass fiber cloth are cured for as a minimum 7 days at room temperature earlier than checking out.

Figure 3.5 Roller used for the removal of air bubble

3.3 EXPERIMENTAL SETUP

The beams are examined within the loading frame of the "Structural Engineering" Laboratory of National Institute of Technology, Rourkela. The checking out procedure for the all the specimen is equal. The two-point loading arrangement is used for testing of beams. Two-point loading is conveniently furnished by means of the association proven in Figure 3.6.

The load is transmitted thru a load cellular and spherical seating directly to a spreader beam. The spreader beam is hooked up on rollers seated on steel plates bedded on the test member with cement if you want to provide a clean leveled surface. The check member is supported on curler bearings appearing on similar spreader plates. The specimen is placed over the 2 metallic rollers bearing leaving one hundred fifty mm from the ends of the beam. The last one thousand mm is divided into identical elements of 500 mm. Two dial gauges are placed simply under the center of the mid span of the beam i.E. Just below the burden factor for recording the deflection of the beams.

Figure 3.6 Experimental setup

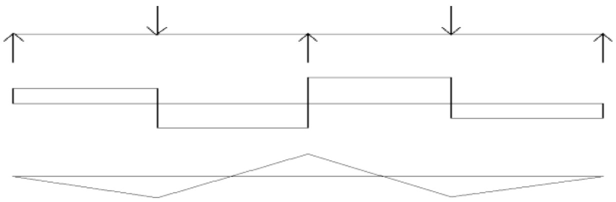


Figure 3.7 Continuous beam (a) Shear Force Diagram (b)Bending Moment Diagram

3.4 FABRICATION OF GFRP PLATE

There are two basic approaches for moulding: hand lay-up and spray-up. The hand lay-up method is the oldest and simplest fabrication technique. The manner is maximum common in FRP marine construction. In hand lay-up method, liquid resin is placed at the side of FRP in opposition to completed floor. Chemical response of the resin hardens the fabric to a strong mild weight product. The resin serves as the matrix for glass fiber as concrete acts for the metal reinforcing rods.

The following constituent materials were used for fabricating plates:

1. Glass Fiber
2. Epoxy as resin
3. Diamine as hardener as (catalyst)
4. Polyvinyl alcohol as a releasing agent

A plastic sheet was kept on the plywood platform and a skinny movie of polyvinyl alcohol was applied as a releasing agent by using the usage of spray gun. Laminating begins with the software of a gel coat (epoxy and hardener) deposited inside the mold by brush, whose primary reason turned into to offer a clean external floor and to guard fibers from direct publicity from the surroundings. Steel curler became carried out to get rid of the air bubbles. Layers of reinforcement have been carried out and gel coat changed into implemented by way of brush. Process of hand lay-up is the continuation of the above process before gel coat is hardened. Again a plastic sheet changed into applied by using making use of polyvinyl alcohol within the sheet as releasing agent. Then a heavy flat metal rigid platform was saved pinnacle of the plate for compressing purpose. The plates had been left for minimum 48 hours earlier than transported and reduce to actual form for testing.

Plates of 2 layers, 4 layers, 6 layers and 8 layers were casted and six specimens from each thickness were tested.

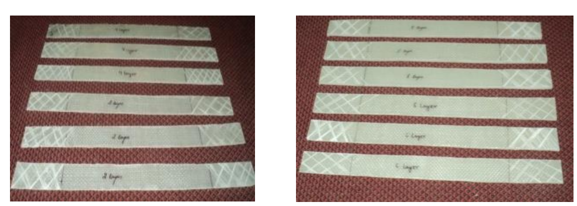


Figure 3.8 Specimens for tensile testing

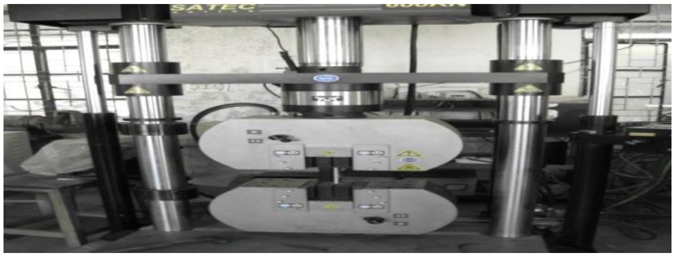


Figure 3.9 Experimental set up of INSTRON 1195

Figure 3.10 Specimen failure after tensile test

Table 3.3 Size of the specimens for tensile test

No. of layers	Length (cm)	Width (cm)	Thickness (cm)
2	15	2.3	0.1
4	15	2.3	0.25
6	15	2.3	0.3
8	15	2.3	0.45

3.5 DETERMINATION OF ULTIMATE STRESS, ULTIMATE LAOD AND YOUNG'S MODULUS

The ultimate stress, final load and younger's modulus changed into decided experimentally by appearing unidirectional tensile check at the specimens reduce in longitudinal and transverse course. The dimensions of the specimens are proven in Table 3.Four. The specimens have been reduce from the plates by means of diamond cutter or by using hex noticed. After reducing via hex noticed, it changed into polished inside the sprucing system.

For measuring the young's modulus, the specimen is loaded in INSTRON 1195 typical tensile test gadget to failure with a encouraged price of extension. Specimens had been gripped in the higher jaw first after which gripped inside the movable lower jaw. Gripping of the specimen need to be proper to prevent slippage. Here, it's far taken as 50 mm from each side. Initially, the stain is saved zero. The load in addition to extension changed into recorded digitally with the help of the load cell and an extensometer respectively. From these facts, pressure as opposed to stain graph become plotted, the preliminary slope of which offers the Young's modulus. The closing strain and the last load had been obtained on the failure of the specimen. The common value of each layer of the specimens is given in the Table 3.5.

Table 3.4 Result of the specimens

Thickness of the specimen	Ultimate stress (MPa)	Ultimate Load (N)	Young's modulus(MPa)
2 Layers	172.79	6200	6829.9
4 Layers	209.09	9200	7788.5
6 Layers	236.23	12900	7207.4
8 Layers	253.14	26200	7333.14

3.6 TESTING OF BEAMS

All the fourteen beams are examined separately. All of them are examined inside the above arrangement. The sluggish boom in load and the deformation inside the dial gauge reading are taken at some stage in the check. The load at which the first visible crack is developed is recorded as cracking load. Then the load is implemented until the remaining failure of the beam. The deflections at midpoint of each span are taken for all beams with and without GFRP and are recorded with admire to increase of load. The records provided in this bankruptcy were interpreted and mentioned within the subsequent chapter to attain a conclusion.

Table 3.5 Details of the Test Specimens for Series S1

Designation of Beams	f_{cu} (MPa)	Main Longitudinal steel		Positive moment strengthening		Negative moment strengthening	
		Top	*Bottom*	*No. of layers*	*Strengthened length(m)*	*No. of layers*	*Strengthened length(m)*
CB1	22.67	2-12 2-10*	2-8	-	-	-	-
SB1	23.3	2-12 2-10*	2-8	2	0.88m	6	0.88m
SB2	25.82	2-12 2-10*	2-8	1			
SB3	23.85	2-12 2-10*	2-8	2			
SB4	24.46	2-12 2-10*	2-8	3			
SB5	24.68	2-12 2-10*	2-8	4			
SB6	22.86	2-12 2-10*	2-8	4			
SB7	25.3	2-12 2-10*	2-8	2		4	
SB8	25.13	2-12 2-10*	2-8	3		6	
SB9	23.9	2-12 2-10*	2-8	2			

**provided at top tension zon*

Table 3.6 Details of the Test Specimens for Series S2

Designation of Beams	f_{cu} (MPa)	Main Longitudinal steel		Positive moment strengthening		Negative moment strengthening	
		Top	*Bottom*	*No. of layers*	*Strengthened length(m)*	*No. of layers*	*Strengthened length(m)*
CB2	25.34	2-6, 2-10*	2-10	0	-	0	-
TB1	24.5	2-6, 2-10*	2-10	2	0.88m	6	0.88m
TB2	23.51	2-6, 2-10*	2-10	2			
TB3	25.61	2-6, 2-10*	2-10	4			

3.6.1 BEAM-1

CONTROL BEAM (CB1)

The control beam, CB1, failed inside the RC traditional flexural mode due to yielding of inner tensile metal reinforcement. The huge flexural cracks have been befell at mid-span and valuable assist. These cracks had been properly extended to the compressive regions.

Figure 3.11 Experimental Setup of the CB1

Figure 3.12 Flexural failure of CB1

3.6.2 BEAM-2 CONTROL BEAM (CB2)

The control beam, CB2 also failed in flexural failure as

shown in Figure 3.13.

Figure 3.13 Control Beam, CB2 after failure

3.6.3 BEAM-3

STRENGHENED BEAM 1 (SB1)

The beam became strengthened with the aid of applying layers of FRP below the beam (width= 150 mm) from assist to assist and 6 layers of FRP above the crucial guide (width= one hundred fifty mm) between two load factors as proven in Figure 3.14. The reinforced beam SB1, confirmed crack at a load of a hundred and ten KN and failed through debonding failure in which the FRP sheet changed into separated without concrete cowl and the final failure took place at 320KN as proven in Figure 3.15. The rupture of FRP sheet turned into unexpected and accompanied with the aid of a noisy noise indicating a rapid release of strength and a total loss of load capacity.

Figure 3.14 Experimental Setup of the Beam

Figure 3.15 Debonding failure of FRP

Figure 3.16 Magnified view of the failure of the beam

3.6.4 BEAM-4

STRENGHENED BEAM 2 (SB2)

Single layer of U-wrap was carried out at the beam to prevent flexural failure. Tensile rupture of FRP occurred on the mid segment of each left and proper span at lower loads and because the load extended, the beam failed in debonding with concrete cover as shown in Figure 3.17 and shear crack changed into developed beneath the FRP layer as shown in Figure 3.18.

Figure 3.17 Tensile rupture of FRP at mid section of right span at lower value of load

Figure 3.18 Ultimate failure of beam by debonding of FRP with concrete cover

3.6.5 BEAM-5

STRENGHENED BEAM 3 (SB3)

U- Jacketed double Layered GFRP turned into implemented to decorate the load capability as proven within the Figure3.19. By strengthening the RC beam using GFRP sheet, the cracking of the beam may be not on time and flexural capacity may be elevated. The reinforced beam failed in debonding of FRP sheet (Figure 3.20).

Figure 3.19 U-jacketed GFRP wrapped on the Beam SB3

Figure 3.20 Debonding failure of FRP

3.6.6 BEAM-6
STRENGHENED BEAM 5 (SB4)

To prevent debonding, one layer of complete U-wrap was provided above the FRP of two layers which was applied at the soffit of the beam (width =150 mm) and one layer of U-strip of width 10 cm was applied over 6 layers FRP above the central support. Complete U-wrap took extra load and prevented the debonding, the failure mode was To prevent debonding, one layer of complete U-wrap changed into furnished above the FRP of two layers which become implemented on the soffit of the beam (width =150 mm) and one layer of U-strip of width 10 cm became implemented over 6 layers FRP above the valuable support. Complete U-wrap took more load and averted the debonding, the failure mode was tensile rupture and because the U-strip could not save you debonding of higher layer of FRP as it got ruptured at higher load value.tensile rupture and as the U-strip could not prevent debonding of upper layer of FRP as it got ruptured at higher load value.

Figure 3.21 Strengthening pattern of beam SB4

Figure 3.22 Crack pattern after initial loading

Figure 3.23 Failure of the beam by tensile rupture

3.6.7 BEAM-7
STRENGHENED BEAM 5 (SB5)

Same arrangement of FRP changed into made as SB4 and to enhance the capacity of beam SB4, two layers of entire U-wrap became furnished in vicinity of one layer and layers of U-strip of width 10 cm turned into carried out in preference to one layer..

Figure 3.24 Cracking pattern

at lower load value

Figure 3.25 Rupture of GFRP sheet at mid section of the right span

3.6.8 BEAM-8
STRENGHENED BEAM 6 (SB6)

Above the U- Jacketed double Layered GFRP, greater two layers of FRP however half of the width of the primary layers, become implemented on the flexural quarter to save you the flexural failure. In this example, as opposed to tensile rupture, debonding failure occurred as proven in Figure 3.27.

Figure 3.26 Debonding of FRP and cracking pattern above

central support of the beam

Figure 3.27 Debonding failure of Strengthened beam SB6

3.6.9 BEAM-9
STRENGHENED BEAM 7 (SB7)

The depth of the neutral axis was found out and the GFRP was provided up to the Neutral axis from the tension face. Here, shear crack was found and debonding occurred as shown in Figure 3.29.

Figure 3.28 Strengthening pattern of SB7

Figure 3.29 Shear crack in the left span

Figure 3.30 Magnified view of shear crack and debonding of GFRP

3.6.10 BEAM-10

STRENGHENED BEAM 8 (SB8)

The no. of FRP layers was increased here as compared to SB7 to examine the changes in load capacity or the failure pattern. The failure mode of the beam was debonding as shown in Figure 3.32.

Figure 3.31 Strengthening pattern of SB8

Figure 3.32 Failure of SB8 by debonding of GFRP

3.6.11 BEAM-11

STRENGHENED BEAM 9 (SB9)

To prevent debonding of FRP, metallic bolt system changed into introduced. The holes within the beam had been made while casting of the beam and after making use of FRP sheet to the beam the metallic bolts have been inserted into the hole and have been tightened after placing the steel plate after the FRP. Anchoring plate, because of high compressive pressure were given buckled as proven in Figure 3.34.

Figure 3.33 Strengthening and anchoring pattern of SB9

Figure 3.34 Failure pattern of SB9

Figure 3.35 Magnified view of Debonding

3.6.12 BEAM-12

TB1

The reinforced beam confirmed crack at a load of a hundred and 10KN and failed by means of debonding failure in which the FRP sheet changed into separated with out concrete cowl at 224 KN that is shown in Figure 3.37. The rupture of FRP sheet become unexpected and accompanied by using a loud noise indicating a rapid launch of energy and a complete loss of load potential. By strengthening the RC beam using GFRP sheet, the cracking of the beam can be behind schedule and flexural potential may be expanded. *Figure 3.36 Top FRP of Beam TB1 before Testing*

Figure 3.37 FRP sheet separations without concrete

3.6.13 BEAM-13

TB2

Full double layered U-wrap was applied and six layers of FRP above the central support. The ultimate failure load was 298 KN.

Figure 3.38 Experimental set up and strengthening pattern of TB2

Figure 3.39 Failure of the beam by tensile rupture

3.6.14 BEAM-14
TB3

Above the U- Jacketed double Layered GFRP, extra two layers of FRP but half of the width of the first two layers, turned into applied on the flexural crack quarter to prevent the flexural failure. In this situation, rather than tensile rupture, debonding failure took place as shown in Figure 3.41 and the failure load turned into 326 KN.

Figure 3.40 Strengthened beam TB3

Figure 3.41 Failure of beam TB3

4. TEST RESULTS AND DISCUSSIONS

The beams were loaded with a focused load on the middle of each span and the received experimental results are supplied and mentioned ultimately in terms of the found mode of failure and cargo-deflection curve. The crack patterns and the mode of failure of every beam are also described on this bankruptcy. All the beams are examined for his or her last strengths and it's far observed that the manage beam had less load carrying ability than the reinforced beam. Two sets of beams i.E. S1 and S2 were examined and one beam from every collection became examined as un-reinforced control beam and relaxation beams had been reinforced with various styles of FRP sheets. The extraordinary failure modes of the beams have been located for each the collection S1 and S2 as shown in Table 4.1 and Table 4.2.

4.1 EXPERIMENTAL RESULTS
4.1.1 FAILURE MODES
4.1.1.1 CONTROL BEAM

The control beam CB1 and CB2 failed completely in flexure. The failure started first at the tension zone and then propagated towards the compression zone and finally failed in flexure.

4.1.1.2 STRENGTHENED BEAM

Generally, the rupture of FRP sheet was sudden and followed by using a loud noise indicating a rapid launch of strength and a complete lack of load ability. For all of the bolstered beams, the failure modes for Series S1 and S2 are described in Table 4.1 and Table 4.2.

The following failure modes were examined for all the tested beams:

- Flexural failure
- Debonding failure (with or without concrete cover)
- Tensile rupture

Rupture of the FRP laminate is believed to occur if the stress inside the FRP reaches its layout rupture pressure before the concrete reaches its most usable stress. GFRP debonding can arise if the pressure within the FRP cannot be sustained by means of the substrate. In order to save you debonding of the GFRP laminate, a issue ought to be positioned at the strain level evolved in the laminate.

Table 4.1 Experimental Results of the Tested Beams for Series S1

Designation of Beams	Failure Mode	P_u (KN)	$\lambda =$ Pu(strengthened beam) Pu(Control beam)
CB1	Flexural failure	260	1
SB1	Debonding failure without concrete cover	320	1.23
SB2	Tensile rupture	325	1.25
SB3	Debonding failure without concrete cover	334	1.28
SB4	Tensile rupture	370	1.42
SB5	Tensile rupture	380	1.46
SB6	Debonding failure without concrete cover	415	1.59
SB7	Debonding failure	332	1.27
SB8	Debonding failure without concrete cover	345	1.32
SB9	Debonding failure	421	1.61

Table 4.2 Experimental Results of the Tested Beams for Series S2

Designation of Beams	Failure Mode	P_u (KN)	$\lambda =$ Pu(strengthened beam) Pu(Control beam)
CB2	Flexural failure	200	1
TB1	Debonding failure	224	1.12
TB2	Tensile rupture	298	1.49
TB3	Debonding of FRP	326	1.68

4.1.2 LOAD DEFLECTION AND LOAD CARRYING CAPACITY

The GFRP strengthened beams and the manage beams are tested to find out their remaining load sporting capacity. The deflection of every beam underneath the load factor i.E. On the midpoint of each span position is analyzed. Mid-span deflections of every reinforced beam are as compared with the manage beam. It is noted that the behavior of the flexure deficient beams while bonded with GFRP sheets are higher than the manage beams. The mid-span deflections of the beams are decrease while bonded externally with GFRP sheets. The stiffness of the strengthened beams turned into better than that of the control beams. Increasing the numbers of GFRP layers typically reduced the mid span deflection and extended the beam stiffness for the same cost of applied load. The use of GFRP sheet had effect in delaying the boom of crack formation.

The closing failure load for all the examined beams are summarized in Table 4.1 and Table four.2. The remaining load enhancement ratio (λ), that is the ratio of the ultimate load of the externally bolstered beam to the manage beam, is supplied in Table four.1 and Table 4.2. From the 2 tables it's miles observed that, addition of GFRP layers elevated the final load ability and through introducing the anchoring gadget, the enhancement of load capability can be performed.

4.1.2.1 STRENGTHENED BEAM OF S1 SERIES

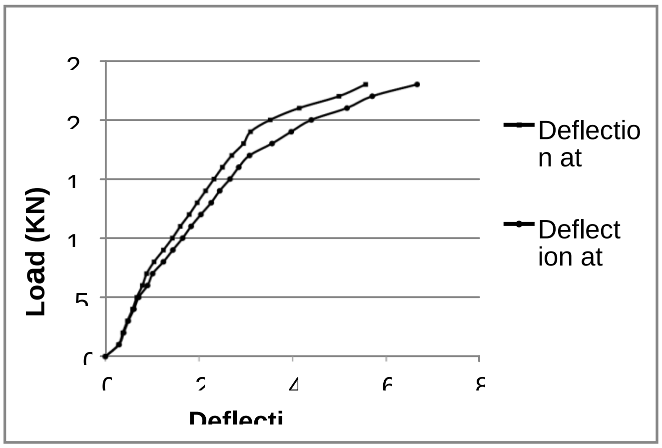


Figure 4.1 Load versus Deflection Curve for CB1

Beam 1 was taken as the manipulate beam (CB1) which is vulnerable in flexure and no strengthening became completed to this beam. Two point static loading turned into applied on the beam and on the every increment of the load, deflection at midpoint of every span have been concerned about the help of dial gauges. Using this load and deflection records, load vs. Deflection curve became plotted.

At the load of 70 KN initial hairline cracks appeared. Later with the increase in loading values the crack propagated further. The Beam CB1 failed completely in flexure at the load of 260 KN.

Beam-2, SB1 is bolstered through applying GFRP at the soffit from aid to aid and at the pinnacle between two load factors. At the midpoint of every span, deflection values were taken and cargo as opposed to deflection curve was plotted. The deflection values are much less than that of the control beam for the identical load price. At the weight of one hundred ten KN initial hairline cracks appeared. Later with the increase in loading values the crack propagated further. At lower load, debonding of FRP with out concrete cowl befell and SB1 ultimately failed in concrete crushing with an final load of 320 KN.

Beam-3, SB2 is bolstered with U-wrap from aid to help distance and at the pinnacle of the beam among the 2 load points. The deflection values are much less than that of the control beam for the identical load cost. No preliminary hairline cracks have been seen due to the protecting of GFRP. Later with the increase in loading values the crack propagated similarly below the GFRP. Tensile rupture befell at decrease load and because the load extended, debonding of the FRP came about with concrete cowl and sooner or later the beam failed in shear and the failure load become 325 KN.

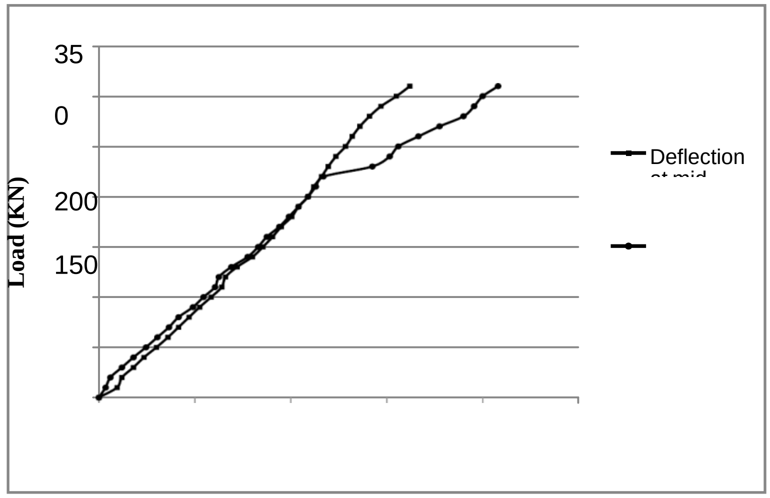


Figure 4.4 Load versus Deflection Curve for SB3

Beam-4, SB3 is bolstered with U-wrap from help to help distance, but the layers have been multiplied and at the pinnacle of the beam among the two load factors. The beam failed in debonding of FRP without concrete cover. The deflection values are remarkably less than that of the control beam and beam SB1 for the same load price. The cracking load changed into one hundred twenty KN and the failure load become 334 KN.

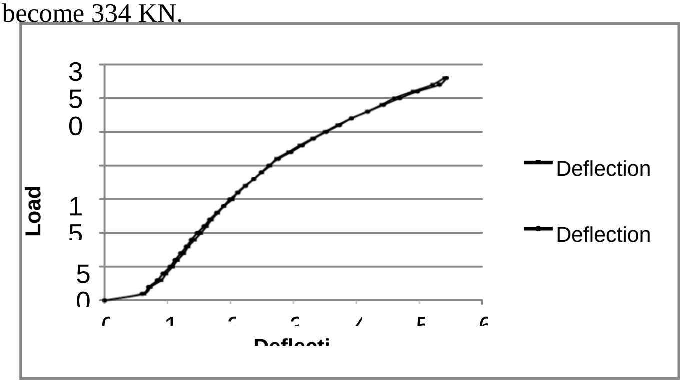


Figure 4.5 Load versus Deflection Curve for SB4

Beam-5, SB4 is bolstered presenting FRP at the soffit of the beam from assist to help distance and U-wrap above it, and on the pinnacle of the beam between the 2 load points and U-strip above it. Tensile rupture of FRP without concrete cowl came about and later with the growth in loading values the crack propagated similarly below the GFRP and beam failed in flexure. The failure load of SB4 changed into 370 KN. The deflection values are once more remarkably much less than that of the manipulate beam for the same load value.

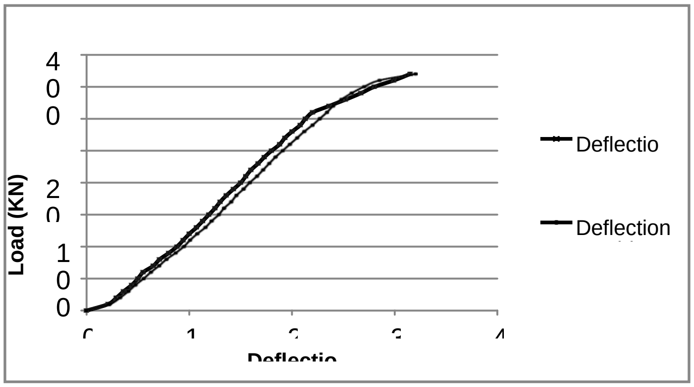


Figure 4.6 Load versus Deflection Curve for SB5

Beam-6, SB5 is bolstered imparting FRP at the soffit of the beam from guide to aid distance and U-wrap above it, and at the pinnacle of the beam between the two load factors and U-strip above it. Here the numbers of FRP layers of U-wrap and U-strip have been multiplied. Tensile rupture of FRP with out concrete cowl occurred at lower load value and later with the growth in loading values the crack propagated in addition beneath the GFRP and beam failed in flexure. The deflection values are much less than that of the control beam for the equal load cost. The failure load of SB5 changed into 380 KN. The final load of this beam changed into better than the beam SB4, which become having same sample of FRP wrapping.

Beam-7, SB6 is bolstered supplying U-wrap FRP from assist to support distance and U-wrap FRP of half of of the width above it, and on the pinnacle of the beam among the 2 load factors. Debonding of FRP with out concrete cowl took place first and later with the growth in loading values the crack propagated further underneath the GFRP and beam failed in flexure. The deflection values are pretty much less than that of the control beam for the same load price. The failure load of SB6 changed into 415 KN.

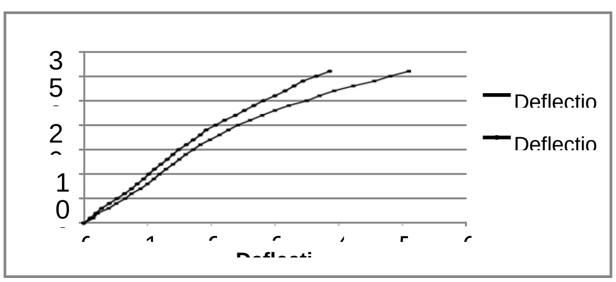
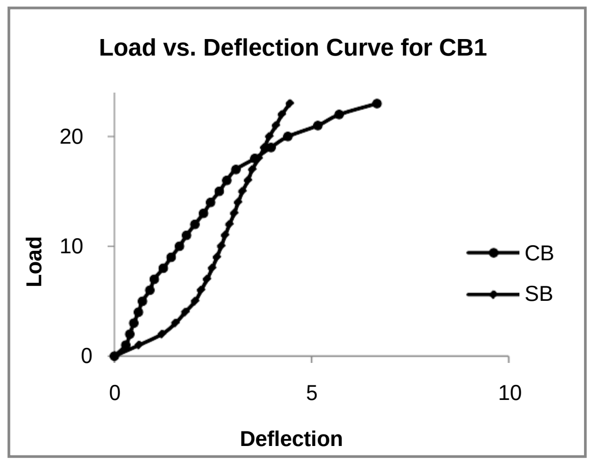


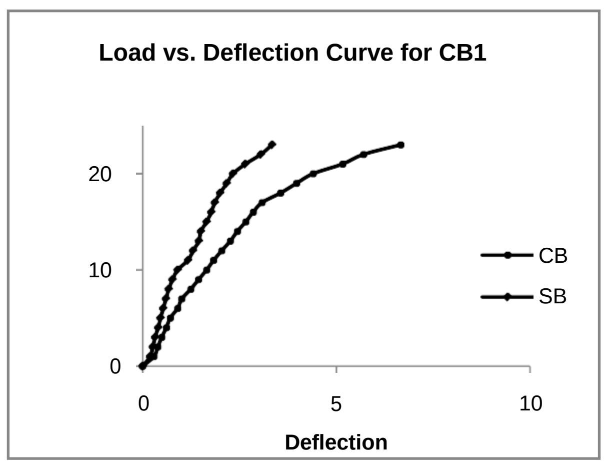
Figure 4.8 Load versus Deflection Curve for SB7

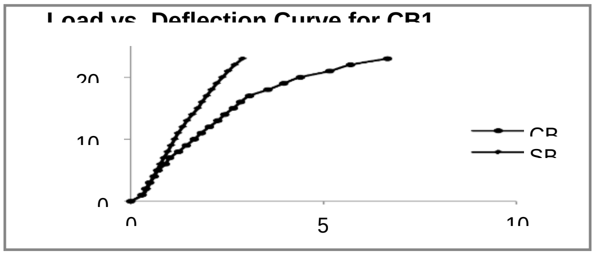
Beam-8, SB7 is strengthened presenting U-wrap FRP from support to assist distance as much as Neutral axis and U-wrap FRP on the pinnacle of the beam between the 2 load points up to Neutral axis. Debonding of FRP with out concrete cowl came about, with the boom in loading values the shear crack advanced and propagated and beam failed in shear. The deflection values are quite much less than that of the manage beam for the identical load value. The failure load of SB7 became 332 KN.

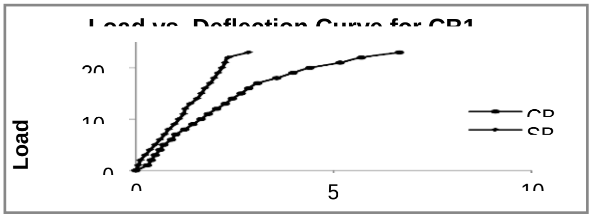
Beam-9, SB8 is bolstered offering U-wrap FRP from help to support distance up to Neutral axis and U-wrap FRP on the pinnacle of the beam among the two load points as much as Neutral axis. Here the layers of the U-wrap have been improved. Beam failed in debonding of FRP with out concrete cowl. Here also, the deflection values are quite much less than that of the manage beam for the same load price. The failure load of SB8 became 345 KN.

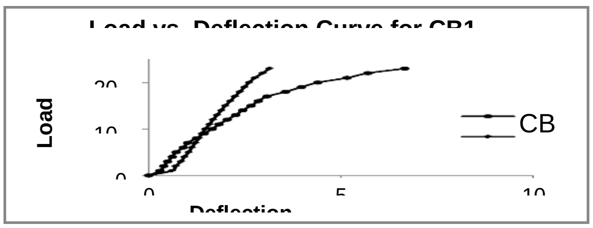
Beam-10, SB9 is reinforced as beam SB6, i.E. U-wrap FRP from assist to support distance and U-wrap FRP of 1/2 of the width above it, and at the top of the beam between the 2 load factors. Here, to save you debonding failure anchoring gadget was delivered. It took extra load than the corresponding beam SB6 and as much as a few load values it averted the debonding failure. It avoided the debonding failure up to a point and eventually failed in flexure. The deflection values are pretty much less than that of the control beam for the identical load value. The failure load of SB9 turned into 421 KN.











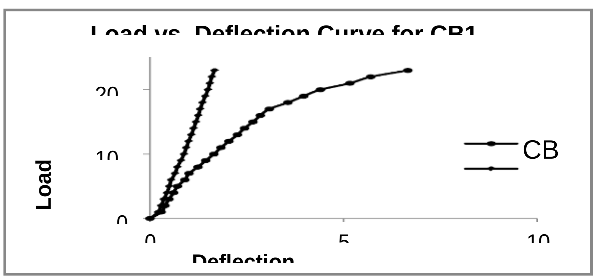


Figure 4.12 Load versus Deflection Curve for CB1, SB2, SB3

In SB2 one layer and in SB3 two layers of U-wrap have been provided to bolster the beams. The midpoint deflections have been as compared with the manage beam and proven in Figure 4.12 from in which it is able to be concluded that the deflection cost is reducing with the aid of strengthening the beams and with the aid of growing the layers of GFRP, the stiffness of beam increases barely.In SB4, one layer of U-wrap and U-strip and in SB5, two layers of U-wrap and two layers U-strip was provided to strengthen the beams. The midpoint deflection was compared with the control beam and shown in Figure 4.13.

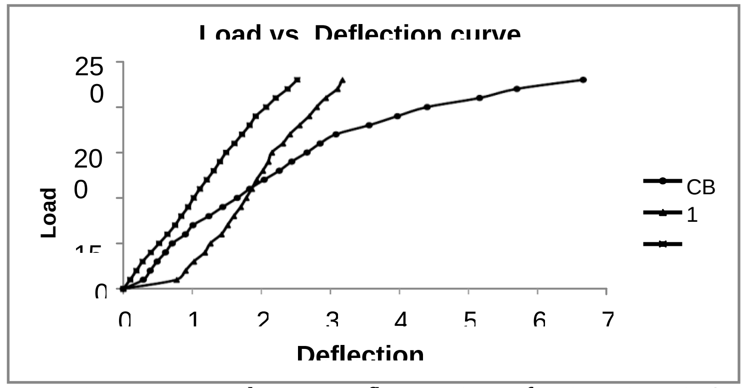


Figure 4.14 Load versus Deflection Curve for CB1, SB7, SB8

In SB7, two and four layers of U-wrap GFRP had been provided below and above the Neutral axis respectively and in case of SB8 the GFRP layers were improved to a few and 6 respectively. The midpoint deflections of SB1 and SB8 were compared to CB1 and from the plotted graphs and it is concluded that, by growing the GFRP layers the stiffness of the beam may be expanded.

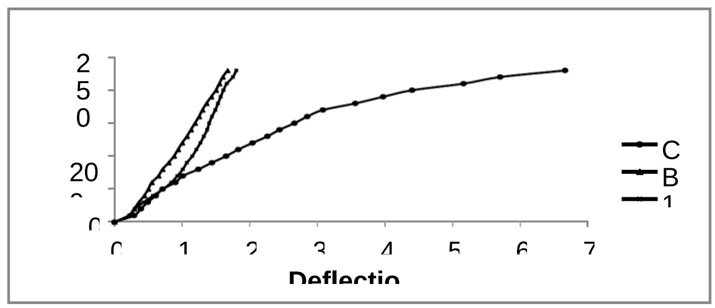


Figure 4.15 Load versus Deflection Curve for CB1, SB7, SB8

In SB9, Steel bolts were used to prevent the debonding failure of FRP. Here, the load capacity of SB9 was higher than SB6, the deflection values were less than CB1 as shown in Figure 4.15.

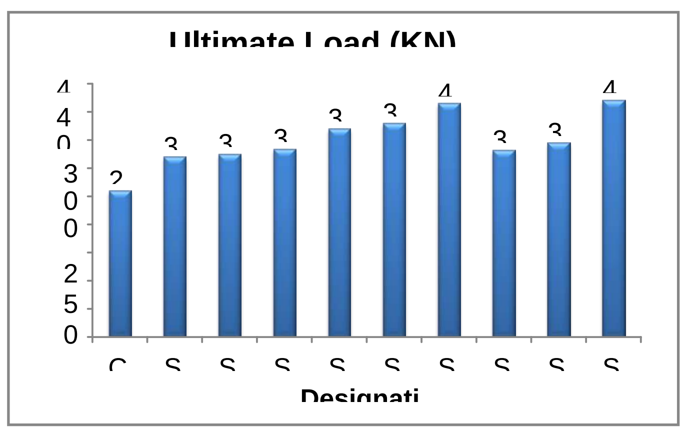


Figure 4.16 Ultimate Load Capacity of Series S1 beams

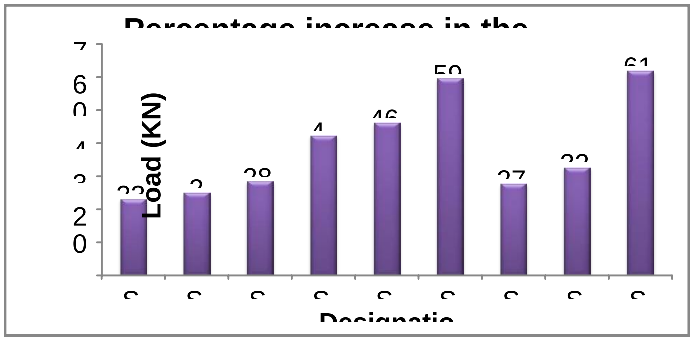


Figure 4.17 Percentage increase in the Ultimate Load Carrying capacity of strengthened beams of S1 w.r.t CB1

From Figure 4.16, it's miles concluded that the load capability of SB9 beam is highest and SB6 beam has second highest load ability amongst all of the strengthened beams of Series S1. The percent boom of load capability of all the beams are calculated and are drawn in Figure 4.17 from which it could be concluded that, by software of GFRP to the beams the load capacity may be superior. Strengthened beam SB6 and SB9 offers the most percentage growth of load potential.

4.1.2.2 STRENGTHENED BEAM OF S2 SERIES

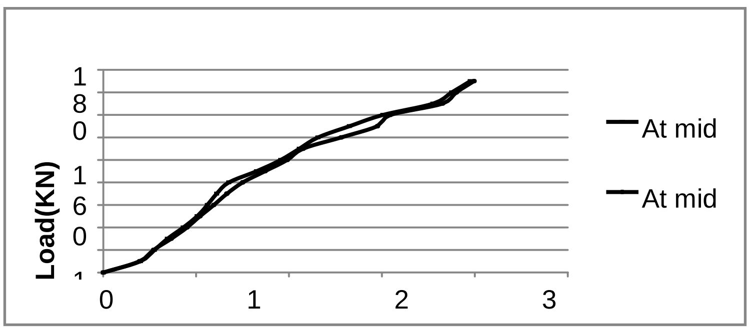


Figure 4.18 Load versus Deflection Curve for CB2

Beam 11, Control Beam for set S2, CB2, to which no outside strengthening become furnished, two factor static loading turned into applied and at the every increment of the burden, deflections at midpoint of each span were curious about the help of dial gauges. Using this load and deflection statistics, load vs. Deflection curve changed into plotted. At the burden of a 110 KN initial hairline cracks appeared and the beam failed in flexure with an ultimate load cost of 200KN

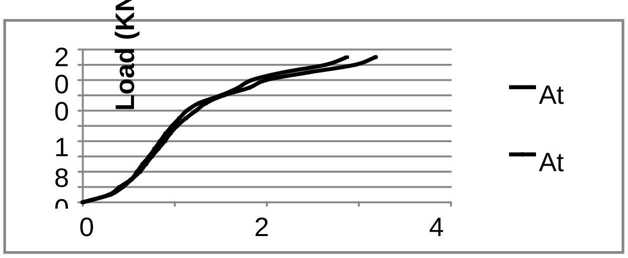


Figure 4.19 Load versus Deflection Curve for TB1

Beam-12, TB1 is strengthened on the soffit from guide to assist and at the top between two load factors. At the midpoint of every span, deflection values were taken and cargo as opposed to deflection curve became plotted. The deflection values are less than that of the manipulate beam for the same load value. At lower load fee, debonding of FRP with out concrete cowl occurred and TB1 sooner or later failed in concrete crushing. At the weight of 120 KN initial hairline cracks regarded. Later with the increase in loading values the cracks propagated further and the beam failed with an remaining load of 224 KN.

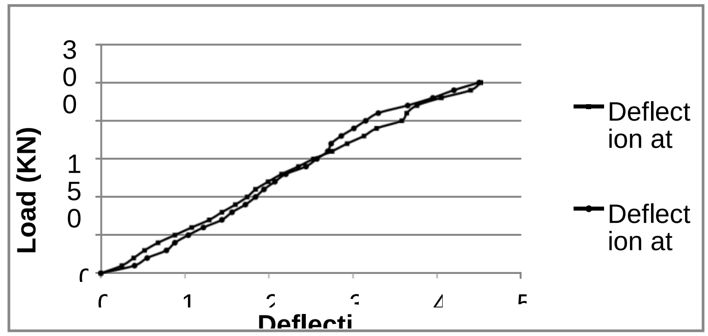


Figure 4.20 Load versus Deflection Curve for TB2

Beam-13, TB2 is reinforced with U-wrap from help to aid distance and on the top of the beam among the 2 load factors but the layers of U-wrap turned into extended right here. The deflection values are less than that of the manage beam for the same load cost. The beam failed in tensile rupture followed via flexural failure. The cracking load turned into 210 KN and the failure load changed into 298 KN.

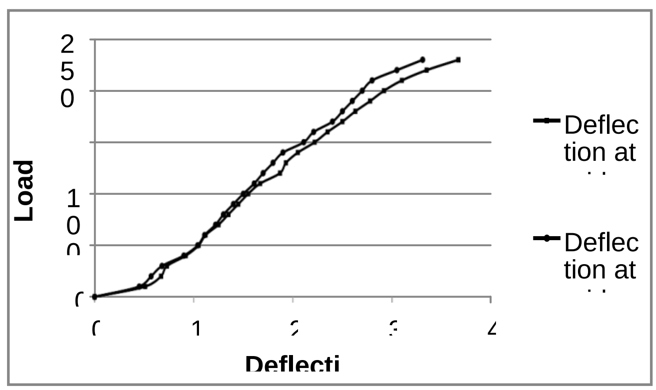


Figure 4.21 Load versus Deflection Curve for TB3

Beam-14, TB3 failed in debonding of FRP with out concrete cover observed via shear crack. The deflection values are remarkably much less than that of the manage beam, CB2 and bolstered beam TB1 for the equal load value. The failure load became 326 KN.

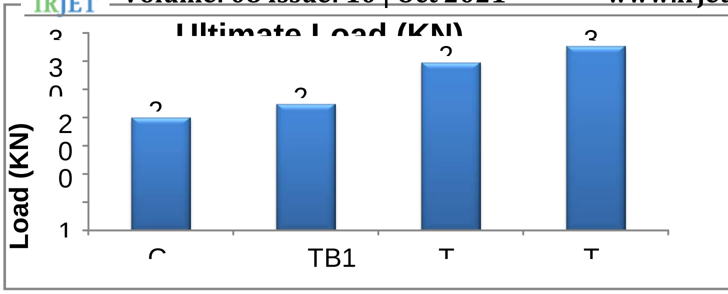


Figure 4.23 Ultimate Load (KN) Capacity of Series S2 beams

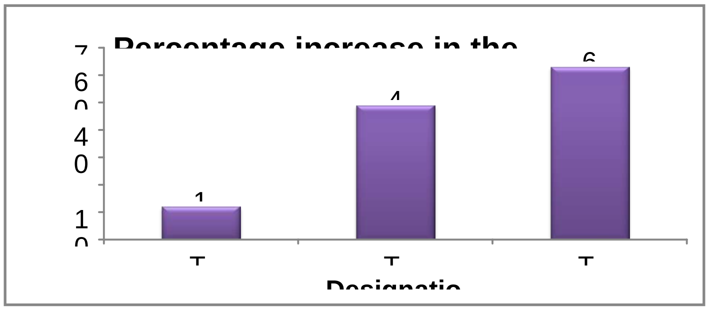


Figure 4.24 Percentage increase in the Ultimate Load Carrying capacity of strengthened beams of S2 w.r.t CB2

The load potential and the share increase of all of the bolstered beams of collection S2 are discussed right here and from Figure 4.23 and Figure 4.24, it's miles found that beam TB3 has the maximum load potential and most percent boom of load sporting ability respectively.

5. FINITE ELEMENT ANALYSIS

Finite element method (FEM) is a numerical approach for solving a differential or critical equation. It has been implemented to a number of physical troubles, in which the governing differential equations are to be had. The approach basically includes assuming the piecewise continuous feature for the solution and obtaining the parameters of the features in a way that reduces the error in the solution.

5.1 FORMULATION

The governing equation for beam is given in Equation 5.1.

$$M = \frac{d^2y}{dx^2}EI \;\ldots\ldots\ldots\ldots\ldots..(5.1)$$

The displacement field v(x) assumed for the beam element should be such that it takes on the values of deflection and the slope at either end as given by the nodal values v_i, ϕ_i, v_j, ϕ_j.

The v(x) can be given by,

$$v(x) = c_0 + c_1x + c_2 x^2 + c_3x^3 \;\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(5.2)$$

In fixing the differential equations via integration, there can be constants of integration that have to be evaluated by means of using the boundary and continuity situations. The variables whose values are to be determined are approximated with the aid of piecewise continuous polynomials. The coefficients of those polynomials are received by means of minimizing the full capability energy of the system. In FEM, typically, these coefficients are expressed in phrases of unknown values of number one variables. Thus, if an element has were given n nodes, the displacement discipline u may be approximated as,

$$u = \sum N_i\, u_i$$
(5.3)

where u_i are the nodal displacements in x-direction and N_i are the shape functions, which are functions of coordinates.

Shape functions or interpolation functions N_i are used in the finite element analysis to interpolate the nodal displacements of any element to any point within each element.

The beam element has modulus of elasticity E, moment of inertia I, and length L. Each beam element has two nodes and is assumed to be horizontal as shown in Figure 5.1. The element stiffness matrix is given by the following matrix, assuming axial deformation is neglected.

$$K = \frac{EI}{L^2}\begin{bmatrix} 12 & 6L & -12 & 6L \\ 6L & 4L^2 & -6L & 2L^2 \\ -12 & -6L^2 & 12 & -6L^2 \\ 6L & 2L^2 & -6L & 4L^2 \end{bmatrix}$$
(5.4)

It is clear that the beam detail has 4 levels of freedom: at every node (a transverse displacement and a rotation). The signal convention used is that the displacement is effective if it points upwards and the rotation is tremendous if it's far counter clockwise. Consequently for a structure with n nodes, the global stiffness matrix K could be of size 2 n × 2 n (when you consider that we've got two ranges of freedom at every node). Once the worldwide stiffness matrix K is obtained we have the subsequent shape equation

$$[K]\{U\}=\{F\} \qquad (5.5)$$

where U is the global nodal displacement vector and F is the global nodal force vector.

First the boundary conditions are applied manually to the vectors U and F. Then the matrix (5.5) is solved by partitioning and Gaussian elimination. Finally once the unknown displacements and reactions are found, the nodal force vector is obtained for each element as follows:

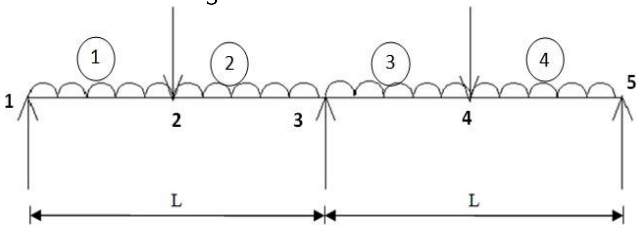
$$\{f\}=[k]\{u\} \quad \ldots\ldots\ldots\ldots \quad (5.6)$$

where {f} is the 4 × 1 nodal force vector in the element and u is the 4 × 1 element displacement vector. The first and second elements in each vector {u} are the transverse displacement and rotation, respectively, at the first node, while the third and fourth elements in each vector {u} are the transverse displacement and rotation, respectively, at the second node.

5.2 VALIDATION OF EXPERIMENTAL VALUE

In the experimental work, the tested beams consist of two spans of each 1000 mm as shown in Figure 5.1 is discritized as shown in Figure 5.2.

Figure 5.1 Continuous beam



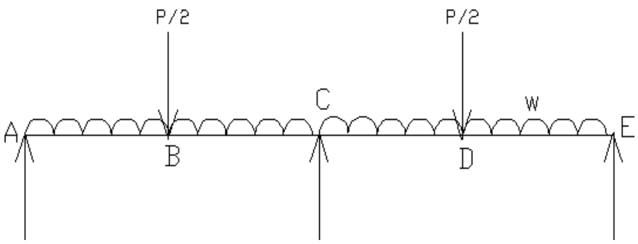


Figure 5.2 Finite element model

Figure 5.3 Beam element forces

The following sign convention is considered for the deflection calculation.

a) *x* is +ve towards right

b) *y* is +ve upwards

c) Anticlockwise slopes are +ve

d) Sagging BM are +ve

Four element mesh is taken as shown in Figure 5.2. Subdividing the span AC into two elements with a node at the load point has the advantage that, the nodal forces can be specified very easily. The meshing has also ensured that all elements are of uniform size, for easy hand calculation. Following the standard procedure, the global stiffness matrix and force vector is obtained as below,

$[K]_{10\times10}\{U\}_{10\times1} = \{F\}_{10\times1}$ ………(5.7)

Since there are five nodes and two d.o.f. per node, the global stiffness matrix is of size (10×10) and {F} is a column vector of size (10×1). The boundary conditions stipulate that the vertical deflection be zero at node 1, 5 and 9.

Boundary conditions are the known values of deflection and slope at specified values of *x*. Here the following boundary conditions are used for the exact analysis of the continuous beam.

At x = 0; y=0 At x= L; y= 0 At x= 2L; y=0

Thus reduced set of equations involving unknown nodal d.o.f. is obtained in matrix form as,

$\{f\}_{7\times1} = [k]_{7\times7}\{u\}_{7\times1}$ ……. (5.8)

Solving the Equation 5.8, the nodal displacement is found out.

The experimental and numerical load-deflection curves obtained for the control beam, CB1 are illustrated in Figure 5.4.

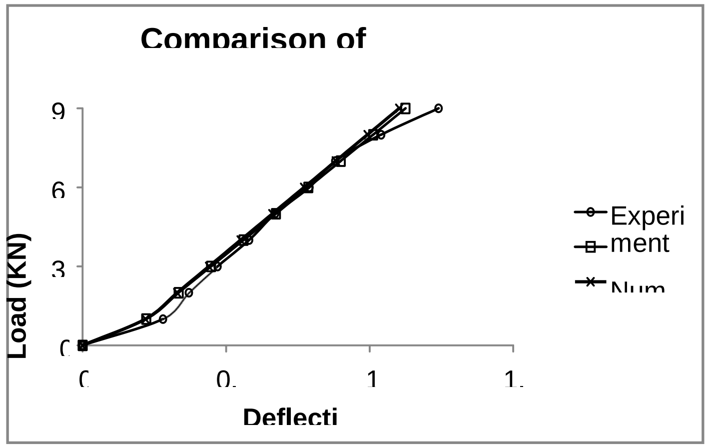


Figure 5.4 Comparison of Experimental value with Numerical and Exact analysis for CB1

The numerical and experimental outcomes for the beam are proven in Figure 5.4. The trend of the loads varying with the deflection provides that the linear elastic state exits in the structure, when the hundreds are equal to about 90 KN.

6. CONCLUSIONS

6.1 CONCLUSION

The gift experimental look at is carried out at the flexural behavior of bolstered concrete rectangular beams strengthened via GFRP sheets. Fourteen bolstered concrete (RC) beams susceptible in flexure having specific set of reinforcement detailing are casted and examined. The beams had been grouped into two collection classified S1 and S2. Each collection had one of a kind longitudinal and transverse steel reinforcement ratios. From the take a look at outcomes and calculated electricity values, the following conclusions are drawn:

1. The remaining load wearing potential of all of the toughen beams is higher when in comparison to the control beam.

2. The preliminary cracks within the bolstered beams are shaped at better load compared to govern beam.

3. From series S1, beam SB9 which became reinforced with the aid of U-wrap and changed into anchored with the aid

of the use of steel plate and bolt system, confirmed the very best final load fee of 415 KN. The percentage growth of the load ability of SB9 changed into sixty one.92 %.

4. The load wearing capability of beam SB6, which become strengthened by layers of U- wrap of length 88 cm in high quality second area and layers of U-wrap of duration forty four cm over first layers, was 415 KN which became closer to the load capability of beam SB9. The percentage growth of load sporting ability became fifty nine.61 % , from which it can be concluded that making use of FRP in the flexure area is pretty powerful method to enhance the weight carrying capability.

5. TB3 beam from Series S2, which became bolstered by using two layers of U-wrap in high quality moment zone and layers of U-wrap in flexure sector above first layers, became having most last load fee of 326 KN, than the other reinforced beams of identical class. The percent boom of this beam was 63 % which become highest among all bolstered beams.

6. Using of metallic bolt and plate machine is an effective method of anchoring the FRP sheet to prevent the debonding failure.

7. Strengthening of continuous beam with the aid of providing U-wrap of FRP sheet is a brand new and powerful way of enhancing the ability of load wearing.

8. Flexural failure at the intermediate aid phase can be avoided by using application of GFRP sheets.

9. In lower variety of load values the deflection obtained using Finite Element models are in appropriate settlement with the experimental effects. For better load values there may be a deviation with the experimental results because linear FEM has been followed.

6.2 SCOPE OF THE FUTURE WORK

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

a) Experimental study of continuous beams with opening

b) Non linear analysis of RC continuous beam

c) FEM modeling of unanchored U-wrap

d) FEM modeling of anchored U-wrap

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