CHAPTER 1

Introduction

1.1 General

The maintenance, rehabilitation and upgrading of structural members is one of the most crucial problems in civil engineering. Infrastructure decay caused by premature deterioration of buildings and structures has lead to the investigation of several processes for repairing or strengthening purposes. The restoration of concrete structures is often made necessary because of the corrosion of reinforcement and the spalling of concrete. Less frequently, damage resulting from fire, explosion or earthquake can account for this need. Strengthening is required when existing structures have to carry higher ultimate loads or sustain severer serviceability conditions. About three decades is the typical design life taken into account as far as civil engineering structures are concerned. During such significant period, changes in the initial conditions such as increased traffic loads or damage of bearing elements in the case of bridges or buildings may take place and require strengthening of structural members. Poor design may be another reason for the need of strengthening. There may be several reasons for the need to strengthen and upgrade structures, such as expired design life, changes in functionality, potential damage caused by mechanical actions and environmental effects, more stringent design requirements, original design and construction errors. From previous research and design practice, several methods for strengthening concrete structures have been developed, and one such method is externally plate bonding which is useful for flexural structural elements.

The earliest applications of the external steel plate bonding method were reported in South Africa and France, and this method has been used quite extensively around the world since 1967. Although steel plate bonding and steel patching and stitching techniques provided a solution for externally reinforcing concrete structure members, these methods have several disadvantages like transportation problem, corrosion of the plates, limited delivery lengths of plates, handling and installation difficulties and expensive formwork to hold the plates in position during adhesive cure. Therefore, an alternative material was required for this structural strengthening method. In recent years, more and more researchers and designers have opted for the alternative of fiber reinforced polymer (FRP) materials. Mainly three types of fibers, namely glass, aramid and carbon, are used. Aramid FRP and glass FRP composites are much more ductile than carbon FRP. CFRP is a very high performance material which can have an extensional modulus comparable to that of steel (200 GPa) and tensile strength 1-2 GPa or higher (Daniel and Ishai 1994). Most importantly, CFRP is an extremely light weight material with specific gravity of 1.6 (Mallick 1993). CFRP laminates are found in continuous lamina of thickness 0.13 to 0.25 mm (Mallick 1993) which is wounded to a drum.

FRP materials have been used primarily in aerospace and defense, rather than in the civil infrastructure area, mainly due to the high cost of the raw material and manufacturing processes. FRP materials possess a high strength-to-weight ratio and corrosion resistance; the latter results in low maintenance costs. They have mechanical and physical properties superior to those of steel, particularly with respect to tensile and fatigue strengths, and these qualities are maintained under a wide range of temperatures. The easy handling of FRP plates or sheets reduces labor costs considerably. The problem involved in joining limited lengths of steel plate is overcome by the fact that FRP sheets may be delivered to site in rolls. With these remarkable features, FRP materials present significant potential in the field of civil engineering. External FRP plate bonding is becoming a promising technique for structural strengthening. The pioneering work of Swiss researchers on the use of FRP as a replacement for steel plate bonding, followed by continuous investigations of concrete elements with bonded FRP reinforcements, has demonstrated the satisfactory strengthening capacities of FRP materials.

The adhesive matrix is a very important factor in the application of FRP strengthened RC beam. It is the adhesive system used to bed FRP sheets and bond them to the concrete surface. Organic epoxy matrices are commercially available and have been widely applied to the FRP reinforcing technique. Nevertheless, the major disadvantages of organic matrix are their lack of fire resistance, susceptibility to UV radiation, and difficulty in handling and cleaning. An inorganic matrix is a new material for bonding of FRP composites. The inorganic matrix is a low-viscosity resin, prepared by blending aluminosilicate powder with a water-based activator. It is more suitable for penetrating carbon fiber sheets. In comparison to the organic epoxy, the inorganic matrix has the advantages of high resistance to fire and high temperatures, resistance to UV radiation, good workability, and no emission of odors and toxins during construction or curing.

ACI 440 has proposed a debonding strength model for RC beams strengthened with FRP, which is based on the use of organic matrix. Information about the strengthening effect of inorganic matrix system is only available in limited literature. It is worth noting that test beams with the inorganic matrix system failed by the rupturing of the fiber instead of by a premature bond failure which would often be associated with the use of organic resin. This research work focus on the use of inorganic matrix so that bond failure could be assumed as non-existent in the beam. Therefore, the major assumption in this thesis lies in the fact that debonding of FRP will not occur.

1.2 Importance of CFRP retrofit for reinforced concrete Structures

A large number of reinforced concrete Structures (Bridge and Buildings) in Nepal are structurally deficient by today's standards (buildings of Kathmandu valley, bridges of east-west highway). The main contributing factors are changes in their use, an increase in load requirements, corrosion deterioration due to exposure to an aggressive environment and explosion during war. In order to preserve those Structures, rehabilitation is often considered essential to maintain their capability and to increase public safety.

In the last decade, fiber reinforced polymer (FRP) composites have been used for strengthening structural members of reinforced concrete Structures. Many researchers have found that FRP strengthening is an efficient, reliable, and cost-effective means of rehabilitation (Marshall and Busel 1996; Steiner 1996; Tedesco, et al. 1996; Kachlakev 1998). Currently in the U.S., the American Concrete Institute Committee 440 is working to establish design recommendations for FRP application to reinforced concrete (ACI 440; 2000).

1.3 Problems and Issues

Various structures like building, bridges etc. are constructed at past in Nepal. These structures were safe for the load considered at that time. But at present, these structures are loaded with heavier load. These structures are unable to withstand the present loading. So, it is very important to increase the capacity of member of these structures to withstand the load coming on the structure at present condition. One of the method to enhance capacity of the structural element is by re-constructing of the structure to withstand the present loading coming on the structure. However, this method of strengthening is not economical and also time consuming. It demands huge amount of money.

Another method and possibly the most promising method of strengthening the reinforced concrete structures are external bonding of Carbon Fiber Reinforced Polymer (CFRP), Glass Fiber Reinforced polymer (GFRP), Aramid Fiber Reinforced Polymer (AFRP) or Steel Plate by means of epoxy adhesives. Out of them, CFRP is more suitable for flexural strengthening because of the tensile strength of CFRP (965Mpa) is more than GFRP (600Mpa) and AFRP. During flexural strengthening of beam, the use of higher percent of CFRP causes failure of beam in compression in stead of tension. When the beam fails in compression, CFRP is not utilization fully bonded on the tension face.

The cost of CFRP sheet is high. The CFRP sheet may be effective within certain range of the span of beam. So, there is no need to provide CFRP thought the span. Over that span, the use of CFRP increases the cost but not load carrying capacity of the beam. So, our main concern is to effectively utilize the CFRP sheet for strengthening of reinforced concrete beam.

1.4 Objectives

1.4.1 Overall Objective

The overall objective of this research work is to enhance the load carrying capacity of CFRP strengthened reinforced concrete beam. This research work evaluates the performance of reinforced concrete beam strengthened by CFRP sheet by performing non linear finite element analysis.

1.4.2 Specific Objectives

The specific objectives of this research work are:

i. To study the ductility behavior in terms of load-displacement curve of the reinforced concrete beam.

ii. To study the failure modes of the reinforced concrete beam.

iii. To study the effect of various parameters like percentage of beam reinforcement, shear span to effective depth ratio, percentage of CFRP sheet and offset of CFRP from the support on the performance of the beam.

iv. Observe the failure modes and evaluate the behavior and performance of CFRP strengthened beam.

v. Utilize the analytical tool to study the behavior of reinforced concrete beam under different influencing factors particularly rebar percentage, CFRP percentage and load range.

vi. Employ the FEM model to predict the critical percentage of CFRP and critical offset of CFRP from support.

1.5 Scope of the Study

Finite element method (FEM) models are developed to simulate the behavior of reinforced concrete beams strengthened with CFRP from linear through nonlinear response and up to failure, using commercial FEM software MARC (MARC 2003). Comparisons are made for load-deflection plots at mid-span; first cracking loads; yielding loads; loads at failure; and crack patterns at failure. Simplifications in modeling and assumptions developed during this research are presented. The behavior of reinforced concrete beam strengthened with CFRP is studied. Conclusions from the current research efforts and recommendations for future studies are included.

1.6 Limitation of the research work

Following are the limitations of this study:

i. Only simply supported reinforced concrete beams are studied.

ii. A perfect bond between CFRP and RC beam is assumed. Therefore, the bond failure between CFRP lamina and Concrete does not take place.

iii. There is sufficient shear reinforcement. So, shear failure of the beams does not exist.

iv. The results obtained from the finite element analysis are not verified with the experimental results.

1.7 Research Approach

This study follows the approach of finite element analysis to clarify the flexural behavior of RC beams externally bonded with CFRP sheets. The finite element model was developed in Marc2003 FEM program to simulate the behavior of reinforced concrete beam subjected to monotonic loading. In order to achieve the above mentioned objectives the following tasks were performed.

i. Gathering and review of relevant available researches on the behavior of reinforced concrete beam strengthened with CFRP.
ii. Selection of proper finite element technique for simulating the behavior of reinforced concrete beam and CFRP sheet.
iii. Determination of the appropriate boundary condition and loading condition to simulate the behavior under loading.
iv. Comparison of the load deflection characteristic and failure modes of the reinforced concrete beam.
v. Study of the effect of different parameters such as percentage of beam reinforcement, percentage of CFRP, size of beam and span of beam.
vi. Model preparation with varying different parameters like geometry of beam, rebar percent, CFRP percent and offset of CFRP from Support and evaluated.
vii. Pure bending test were conducted on RC beam under four point bending.
viii. An elaborate fiber section model was developed to trace the response of RC beam using model of concrete, rebar and CFRP.

1.8 Organization of the Thesis

The thesis has been organized in eight chapters as follows.

i. In chapter 1, introduction on the subject matter of the study, importance of the strengthening of RC beam, discussions on the problems and issues, objectives of the study, and research approach adopted for the study have been presented.
ii. In Chapter 2, literatures related to the proposed study have been reviewed.
iii. In Chapter 3, discussions on the Finite Element Modeling of reinforced concrete and CFRP have been carried out.
iv. In Chapter 4, discussions on Analytical Study have been carried out.
v. In Chapter 5, discussions on the results obtained from analytical study have been done.

vi. In chapter 6, conclusions have been drawn from the results of the analytical study.
vii. In Chapter 7, recommendations regarding the stated problems have been stated.
viii. In Chapter 8, recommendations for future study have been proposed.

CHAPTER 2

Literature Review

2.1 General

Conventional reinforced concrete members consist of Portland cement concrete and steel reinforcement. In such beams, concrete resists compressive forces and steel reinforcing bars embedded in concrete, typically on the tension side, resist tensile and dowel forces. Such an arrangement is structurally efficient because of concrete's inherent resistance to compression, where as that of steel's in tension and partially in shear. In some reinforced concrete members, steel reinforcement is also used to partially resist compression and enhance the flexural strength and stiffness of members with limited depth and even to limit crack width. Recent advances in fiber reinforced polymer (FRP) composite technologies have resulted in alternative reinforcing materials that can be used efficiently as supplemental, externally bonded reinforcement. A class of such materials comprises fiber reinforced polymeric materials, commercially available in the form of fabrics or sheets, that can be bonded to the outer surface of concrete members as external reinforcement to accomplish a number of desired objectives.

The repair of deteriorated, damaged and substandard civil infrastructures has become one of the most important issues for the civil engineer worldwide. The use of externally-bonded fiber-reinforced polymer (FRP) composites to strengthen, rehabilitate and retrofit civil engineering structures becomes very important. The benefits of fibre-reinforced polymer (FRP) composites in different types of structures such as masonry and metallic strengthening can be seen. The use of FRP covers practical considerations including material behavior, structural design and quality assurance.

2.2 Past Research on Strengthening of Beam

Deborah Bohot (1999) studied the Carbon Fiber Reinforced Polymers (CFRP) for their usefulness in structural design. A new way of using them in this capacity is to adhere them in sheets to the underside of concrete beams, around concrete connections for shear support, and as reinforcement along walls susceptible to tension. Some companies have taken the lead to prove the viability of this method and their doing so have spurned other companies, such as Zoltek, a carbon fiber manufacturer, to compete in this market as well. In order for Zoltek to take on the construction industry a protocol for their system needs to be developed. This paper discusses the elementary steps in analyzing a system using Zoltek's fibers for structural use. Tension Tests showed that the Zoltek fibers are inferior in strength to another already marketed system, Mbrace. The Zoltek fibers did show promise in some respect as they were more predictable in strength and modulus than the Mbrace fibers. They were also merited in good adhesion to the resins. Delamination tests were done as well to test the full resin system (primer, putty, saturant).

Y.J. Kim, A. Fam, A. Kong, and R. El-Hacha (2003) studied the application of a new generation of externally bonded composite material in flexural strengthening of reinforced concrete beams. The steel reinforced polymer (SRP) composite consists of high-carbon steel unidirectional Hardwire® fabrics embedded in epoxy resin, and offers high strength and stiffness characteristics at a reasonable cost. In this paper, the mechanical properties of SRP are evaluated and its application in flexural strengthening of RC beams is investigated. Six beams were tested in three-point bending to study the effect of SRP retrofitting on flexural behavior, failure modes, and crack patterns. Test parameters included variation of the width of SRP sheets and the use of SRP U-wraps to prevent premature failure caused by delamination of the longitudinal sheet. Significant increase in flexural capacity, up to 53%, and pseudo-ductile failure modes were observed in SRP- strengthened beams. Failure was governed primarily by concrete cover delamination at the ends of SRP sheets or concrete crushing. The U-wraps improved flexural stiffness by means of controlling diagonal cracking and providing anchorages to the longitudinal SRP sheets, which reduced their slip. Shear stress concentrations near cut-off points of SRP sheets have also been investigated. An analytical model was used to predict the nominal flexural strength of SRP-strengthened beams.

C.A. Zeris (2006) studied the experimental investigation for comparing the response of four full size reinforced concrete (RC) beams, mostly without current detailing provisions for ductility, typical of existing RC buildings designed with past seismic regulations. All specimens were initially strengthened with synthetic fiber reinforced polymers (FRP) and were tested as continuous two-span beams with different shear span to depth ratios, their response being compared to a control specimen encompassing currently enforced detailing provisions. Three alternative strengthening methods are considered. The tests demonstrate that specimens with longitudinal side strengthening exhibit a larger ductility capacity, compared to the specimen strengthened with conventional top and bottom FRP application. Furthermore, the reliability of current design analytical models for strength prediction is evaluated, considering their ability to predict experimental behavior.

M.A. Youssef (2005) studied the importance to rehabilitate ageing and deteriorated existing steel structures to develop simple and efficient rehabilitation techniques. One of the currently developed techniques involves bonding Fiber Reinforced Plastic (FRP) sheets to the flanges of steel beams. This paper presents an analytical model to predict the linear and nonlinear behavior of steel beams rehabilitated using this technique. The model is based on the solution of the differential equations governing the composite behavior of a rehabilitated steel beam and includes representation of the peel and shear behavior of the adhesive material. A bending test was conducted on a W-shaped steel beam, with glass FRP sheets bonded to its flanges, and the experimental results were used to validate the model. The model predictions for the failure load, failure mechanism, midspan deflection, steel strains, and FRP strains were found to be in excellent agreement with the experimental results. The model was also used to predict some parameters that were difficult to evaluate experimentally. This provided a better understanding of the behavior of the rehabilitated beam.

Amjad J. Aref, Yasuo Kitane, George C. Lee (2004) studied the finite element analysis (FEA) and simple methods of analysis of an innovative hybrid FRP-concrete bridge superstructure system. The hybrid superstructure was designed, and its one-fifth scale model was tested under quasi-static loading. Three trapezoidal glass fiber reinforced polymer (GFRP) box sections were bonded together to make up a one-lane super- structure, and a layer of concrete was placed in the compression zone of those sections. This new design was proposed in order to make the best of each constituent material, and to increase stiffness of GFRP composite structures. A detailed finite element model was created to predict the static flexural behavior of the bridge superstructure under live loads. Results from the FEA have shown that a linear FEA can accurately predict the static behavior of this new bridge superstructure under design live loads. Moreover, nonlinear constitutive models of concrete and GFRP were incorporated in the FEA code to examine the bridge behavior beyond its elastic range. And, simple analytical methods including the beam analysis and the orthotropic plate analysis were also presented in this paper, and their accuracy was validated.

Hsuan-Teh Hu , Fu-Ming Lin, Yih-Yuan Jan (2003) used numerical analyses using the ABAQUS finite element program to predict the ultimate loading capacity of rectangular reinforced concrete beams strengthened by fiber-reinforced plastics applied at the bottom or on both sides of these beams. Nonlinear material behavior, as it relates to steel reinforcing bars, plain concrete and fiber-reinforced plastics is simulated using appropriate constitutive models. The influences of fiber orientation, beam length and reinforcement ratios on the ultimate strength of the beams are investigated. It has been shown that the use of fiber-reinforced plastics can significantly increase the stiffness as well as the ultimate strengths of reinforced concrete beams. In addition, with the same fiber-reinforced plastics layer numbers, the ultimate strengths of beams strengthened by fiber-reinforced plastics at the bottom of the beams are much higher than those strengthened by fiber-reinforced plastics on both sides of the beams.

Zhishen Wu, Wenxiao Li, Naoki Sakuma (2004) used a novel hybrid FRP–concrete structural system. CFRP sheets with high strengths are axially bonded on the bottom surface of the concrete core to carry tensile load while GFRP sheets with high rupture strains are hoop-directionally wrapped to bear the shear load. The GFRP sheets also provide confinement to concrete core and prevent premature debonding of CFRP sheets. A minimum reinforcement ratio of rebar is used to control the localization and propagation of flexural cracks. Instead of conventional shear keys of hybrid systems, an epoxy resin bonding technique is adopted to assure composite effect between FRP and concrete and hence avoid buckling failure mode of FRP shell. A series of 4-point bending experiments were carried out to study the performances of the proposed hybrid FRP–concrete beams. Based on the principles of strain compatibility and equilibrium, an iterative analytical model is developed to evaluate the flexural behavior of the hybrid beam specimens.

P. Alagusundaramoorthy, I.E.HariK, C.C.Choo (2002) Study about the four point bending flexural tests on two concrete control beams and twelve concrete beams strengthened with externally bonded carbon fiber reinforced polymer (CFRP) sheets/fabric. An analytical procedure, based on compatibility of deformations and equilibrium of forces, is developed. The effectiveness of externally bonded CFRP sheets or fabric on the flexural strength of concrete beams is studied. The centerline deflections at service load and at failure load, maximum strain in CFRP sheets/fabric at failure, stress variation in CFRP sheets/fabric along the span length at failure load, and failure load are calculated, and compared with experimental results. The failure load of the strengthened beams is also calculated using Whitney's stress block and compared with the presented analytical procedure.

Bo Gao, Christopher K.Y. Leung, Jang-Kyo Kim (2005) Proposed that amongst various methods developed for strengthening and rehabilitation of reinforced concrete (RC) beams, external bonding of fiber reinforced plastic (FRP) strips to the beam has been widely accepted as an effective and convenient method. The experimental research on FRP strengthened RC beams has shown five most common modes, including (i) rupture of FRP strips; (ii) compression failure after yielding of steel; (iii) compression failure before yielding of steel; (iv) delamination of FRP strips due to crack; and (v) concrete cover separation. In this paper, a failure diagram is established to show the relationship and the transfer tendency among different failure modes for RC beams strengthened with FRP strips, and how failure modes change with FRP thickness and the distance from the end of FRP strips to the support. The idea behind the failure diagram is that the failure mode associated with the lowest strain in FRP or concrete by comparison is mostly likely to occur. The predictions based on the present failure diagram are compared to 33 experimental data from the literature and good agreement on failure mode and ultimate load has been obtained. Some discussion and recommendation for practical design are given.

J.F. Chen, H. Yuan, J.G. Teng (2006) Proposed that the concrete beam can be strengthened by bonding a fiber reinforced polymer (FRP) plate to the tension face, and a common failure mode for such beams involves the debonding of the FRP plate that initiates at a major flexural crack, which is widely referred to as intermediate crack (IC) debonding. To understand IC and other debonding failures, the bond behavior between FRP and concrete has been studied extensively using simple pull-off tests, in which a plate is bonded to a concrete prism and is subject to tension. However, the behavior of the FRP-to-concrete interface in a beam can be significantly different from that captured in a pull-off test as, in a beam, whether debonding along the FRP-to-concrete interface occurs at a major flexural crack or not depends on the conditions at this crack as well as at the adjacent crack on the path of the debonding propagation. This paper is therefore concerned with the debonding process of an FRP-to-concrete bonded joint where the FRP plate is subject to tension at both ends, which closely approximates the IC debonding process in a flexurally strengthened RC member. The same problem has been the topic of a previous study by the authors, where a bilinear local bond–slip model was employed for the FRP-to-concrete interface. However, that solution is rather complex and difficult to apply in practice. The aim of this study is to produce a simplified solution by employing the simple linearly softening local bond–slip law for the interface. Results from this simplified analytical solution are compared with those from the previous solution, showing little loss of accuracy in predicting the load–displacement response and the ultimate load. The most significant outcome of the new solution is a simple expression for the ultimate load of the bonded joint which offers the potential for direct practical application. While the emphasis of the paper is on FRP-to-concrete joints, the solution and methodology are applicable to similar joints between other materials such as FRP-to-steel or steel-to-concrete bonded joints.

M.S. Mohamed Ali, D.J.Oehlers, R. Seracinob (2005) Proposed a convenient and proven technique for increasing the vertical shear capacity of reinforced concrete beams is to externally bond fiber reinforced polymer (FRP) to the sides of the beam with the fibers orientated in the transverse or vertical direction. The FRP, which can be in the form of pultruded plates or applied in the wet lay-up procedure, acts as external FRP stirrups resisting vertical shear in the same way as the conventional internal steel tirrups. However, internal steel stirrups are ductile as they are both fully anchored and can yield, which is in contrast to external FRP stirrups that can debond in a brittle fashion and do not yield. Hence, there is no guarantee that the peak vertical shears forces that can be resisted by the steel stirrups and by the transverse FRP plates coincide. In this paper, a partial-interaction model has been developed that quantifies the vertical shear interaction between transverse FRP plates and steel stirrups.

Damian Kachlakev, Thomas Miller & Solomon Yim (2001) Linear and non-linear finite element method models were developed for a reinforced concrete bridge that had been strengthened with fiber reinforced polymer composites. ANSYS and SAP2000 modeling software were used; however, most of the development effort used ANSYS. The model results agreed well with measurements from full-size laboratory beams and the actual bridge. As expected, a comparison using model results showed that the structural behavior of the bridge before and after strengthening was nearly the same for legal loads. Guidelines for developing finite element models for reinforced concrete bridges were discussed.

2.3 Strengthened RC Beam Behavior

2.3.1 Structural Behavior

The strengthened beam and slab has capacity to restore their strength and stiffness, assuming that debonding the CFRP wrap would not cause member failure. It limits the crack width under increased loads or sustained loads. Retrofitted concrete members enhance the flexural strength and strain-to-failure of concrete elements necessitated by increased loading conditions such as earthquakes or traffic loads. FRP enables designing new concrete members having depth limitations or needing high demand on ductility. Strengthening of structural elements enhance the service life of concrete members.

2.3.2 Bonding of CFRP Sheets at the bottom of RC beam

The FRP sheet is laid at the bottom of the reinforced concrete beam with the help of epoxy adhesive. The tension face of concrete surface is made rough to a coarse sand paper texture by scarifying with a toothed grinder and cleaned with an air blower. The concrete surface is made free of all apparent moisture. The bonding surface of the CFRP sheet is made rough using fine sand papers until the matrix-rich on the surface is removed and fibers are exposed, and cleaned with acetone. A two-component epoxy primer is mixed thoroughly and applied to the concrete surface, and is allowed to dry for thirty minutes. A two-component structural epoxy paste is applied over the primer on the concrete surface and on the bonding surface of CFRP sheet using a 3 mm V-notched trowel. The CFRP sheet is installed over the concrete surface by starting at one end and by applying enough pressure to press out the excessive epoxy paste and trapped air pockets. The excessive epoxy paste is removed using an acetone rich wash cloth. The surfaces are clamped together until the epoxy paste is cured.

CHAPTER 3

Finite Element Modeling of Beam

3.1 General

There are seventy eight beam specimens required in this study. Due to limitation of available resource, experimental study of reinforced concrete beam is not possible in our laboratory. So, analytical studies were done by using a commercial FEM software Marc2003 in this research work.

3.2 Modeling of Reinforced Concrete Beams

3.2.1 Full-Size Beams

There are mainly three types of beam considered in this study. They are-

i. Laboratory scale Beam
ii. Building Specific Beam
iii. Bridge Specific Beam

3.2.1.1 Laboratory Scale Beam

This is the beam of size 150x300 mm with span 4m. In total, there are thirty two beams. There are two types of beams corresponding to the reinforcement ratio of 0.33% and 0.75%. Among them, sixteen beams were modeled with 0.33% rebar and another sixteen beams were modeled with 0.75% rebar. Each beam had a different strengthening scheme as described below:

- Two Control Beam with 0.33% and 0.75% rebar and no FRP strengthening.
- Six beams with 0.33% rebar and 0.17%, 0.33%, 0.50%, 0.67%, 0.83% &1.00% unidirectional CFRP laminates attached to the bottom of the beam. The fibers were oriented along the length of the beam.
- Six beams with 0.75% rebar and 0.17%, 0.33%, 0.50%, 0.67%, 0.83% &1.00% unidirectional CFRP laminates attached to the bottom of the beam. The fibers were oriented along the length of the beam.
- Nine beams with 0.33% rebar and 0%, 5%, 10%, 15%, 20%, 25%, 30%, 35%& 40% offset from the Supports of unidirectional CFRP laminates are attached to the bottom of the beam. The fibers were oriented along the length of the beam.
- Nine beams with 0.75% rebar and 0%, 5%, 10%, 15%, 20%, 25%, 30%, 35%&

40% offset from the Supports of unidirectional CFRP laminates are attached to the bottom of the beam. The fibers were oriented along the length of the beam.

3.2.1.1 Building Specific Beam

This is the beam of size 300x450 mm with span 5m. In total, there are twenty three beams. There are two types of beams corresponding to the reinforcement ratio of 0.33% and 0.75%. Among them, sixteen beams were modeled with 0.33% rebar and another seven beams were modeled with 0.75% rebar. Each beam had a different strengthening scheme as described below:

- Two Control Beam with 0.33% and 0.75% rebar and no FRP strengthening.
- Six beams with 0.33% rebar and 0.17%, 0.33%, 0.50%, 0.67%, 0.83% &1.00% unidirectional CFRP laminates attached to the bottom of the beam. The fibers were oriented along the length of the beam.
- Six beams with 0.75% rebar and 0.17%, 0.33%, 0.50%, 0.67%, 0.83% &1.00% unidirectional CFRP laminates attached to the bottom of the beam. The fibers were oriented along the length of the beam.
- Nine beams with 0.33% rebar and 0%, 6%, 12%, 18%, 24%, 30%, 36%&42% offset from the Supports of unidirectional CFRP laminates are attached to the bottom of the beam. The fibers were oriented along the length of the beam.

3.2.1.1 Bridge Specific Beam

This is the beam of size 500x1800 mm with span 20m. In total, there are twenty three beams. There are two types of beams corresponding to reinforcement ratio of 0.33% and 0.75%. Among these beams, sixteen beams were modeled with 0.33% rebar and another seven beams were modeled with 0.75% rebar. Each beam had a different strengthening scheme as described below:

- Two Control Beam with 0.33% and 0.75% rebar and no FRP strengthening.
- Six beams with 0.33% rebar and 0.17%, 0.33%, 0.50%, 0.67%, 0.83% &1.00% unidirectional CFRP laminates attached to the bottom of the beam. The fibers were oriented along the length of the beam.
- Six beams with 0.75% rebar and 0.17%, 0.33%, 0.50%, 0.67%, 0.83% &1.00% unidirectional CFRP laminates attached to the bottom of the beam. The fibers were oriented along the length of the beam.

- Nine beams with 0.33% rebar and 0%, 5%, 10%, 15%, 20%, 25%, 30%, 35%& 40% offset from the Supports of unidirectional CFRP laminates are attached to the bottom of the beam. The fibers were oriented along the length of the beam.

The current study presents results from the finite element analysis of the seventy-eight full-scale beams. The finite element model uses three-dimensional layered elements to model CFRP composites.

The MARC finite element program (Marc2003) was used in this study to simulate the behavior of the seventy eight beams. In general, the conclusions and methods would be very similar using other nonlinear FEA programs. Each program, however, has its own nomenclature and specialized elements and analysis procedures that need to be used properly. The designer/analyst must be thoroughly familiar with the finite element tools being used, and must progress from simpler to more complex problems to gain confidence in the use of new techniques.

3.3 Element Types

3.3.1 Concrete Element

In Marc2003, concrete can be modeled using element type 7 of this FEM software. Element type 7 is an eight-node, isoparametric, arbitrary hexahedral. As this element uses trilinear interpolation functions, the strains tend to be constant throughout the element. This element can be used for all constitutive relations and was used in this study for modeling concrete.

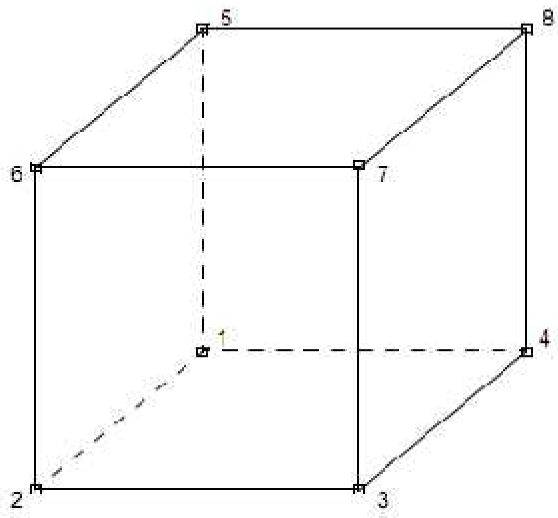

Figure 3-1: Typical 8-noded 3-D Concrete Element

3.3.2 Rebar Element

In Marc2003, Rebar can be modeled using element type 146 of this FEM software. This element is an isoparametric, three-dimensional, 8-node empty brick in which we can place single strain members such as reinforcing rods or cords (that is, rebars). The element is then used in conjunction with the 8-node brick continuum element (for example, element types 7 or 84) to represent cord reinforced composite materials. This technique allows the rebar and the filler to be represented accurately with respect to their stress distribution, so that separate constitutive relation can be used in each (for example, cracking concrete and yielding rebar). The position, size, and orientation of the rebars are input either via the REBAR option. It is assumed that several "layers" of rebars are presented. The number of such "layers" is input through the REBAR option. Each layer is assumed to be similar to a pair of opposite element faces (although the rebar direction is arbitrary), so that the "thickness" direction is from one of the element faces to its opposite one. For instance (see Figure 3-2), if the layer is similar to the 1, 2, 3, 4 and 5, 6, 7, 8 faces of the element, the "thickness" direction is from 1, 2, 3, 4 face to 5,6,7,8 face of the element. The element is integrated using a numerical scheme based on Gauss quadrature. Each layer contains four integration points (see Figure 3-2). At each such integration point on each layer, we must input (via either the REBAR option or the REBAR user subroutine) the position, equivalent thickness (or, alternatively, spacing and area of cross-section), and orientation of the rebar.

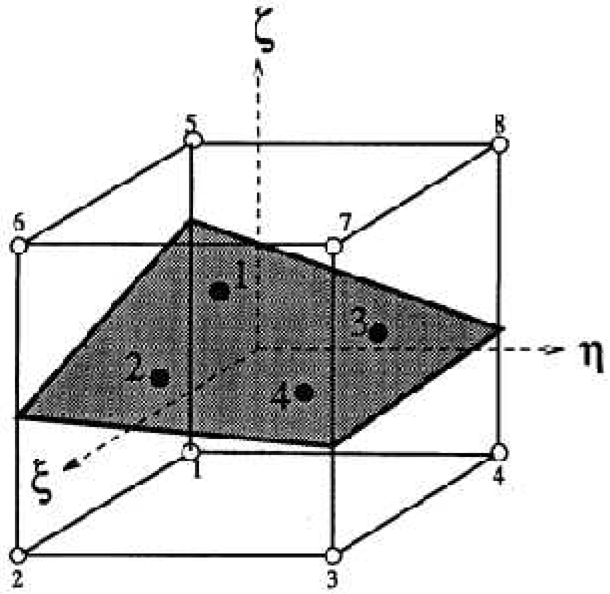

Figure 3-2: Typical Layer in 8-noded 3-D Rebar Element

3.3.3 CFRP Composites Element

In Marc2003, CFRP Composite can be modeled using element type 139 of this FEM software. This is a four-node, thin-shell element with global displacements and rotations as degrees of freedom. Bilinear interpolation is used for the coordinates, displacements and the rotations. The membrane strains are obtained from the displacement field; the curvatures from the rotation field. The element can be used in curved shell analysis as well as in the analysis of complicated plate structures. For the latter case, the element is easy to use since connections between intersecting plates can be modeled without tying. Due to its simple formulation when compared to the standard higher order shell elements, it is less expensive and, therefore, very attractive in nonlinear analysis. The element is not very sensitive to distortion. All constitutive relations can be used with this element.

The element is defined geometrically by the (x,y,z) coordinates of the four corner nodes. Due to the bilinear interpolation, the surface forms a hyperbolic paraboloid which is allowed to degenerate to a plate. The element thickness is specified in the GEOMETRY option.

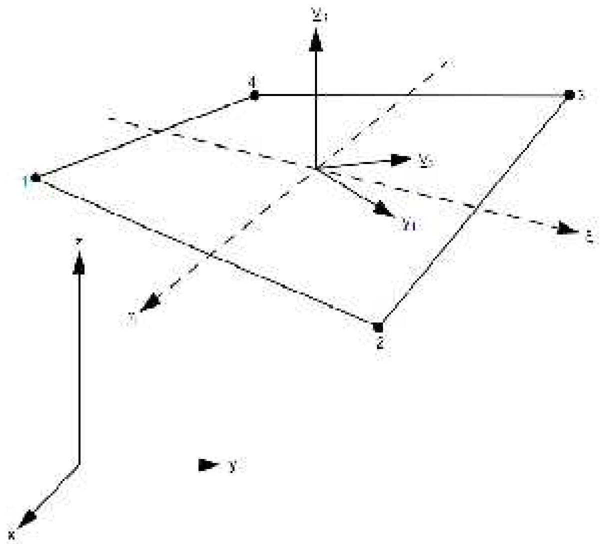

Figure 3-3: Typical 4-noded Thin-shell Element

3.4 MATERIAL PROPERTIES

3.4.1 Concrete

Development of a model for the behavior of concrete is a challenging task. Concrete is a quasi- brittle material and has different behavior in compression and tension. The tensile strength of concrete is typically 8-15% of the compressive strength (*Shah, et al. 1995*). Figure 3-4 shows a typical stress-strain curve for normal weight concrete (*Bangash 1989*).

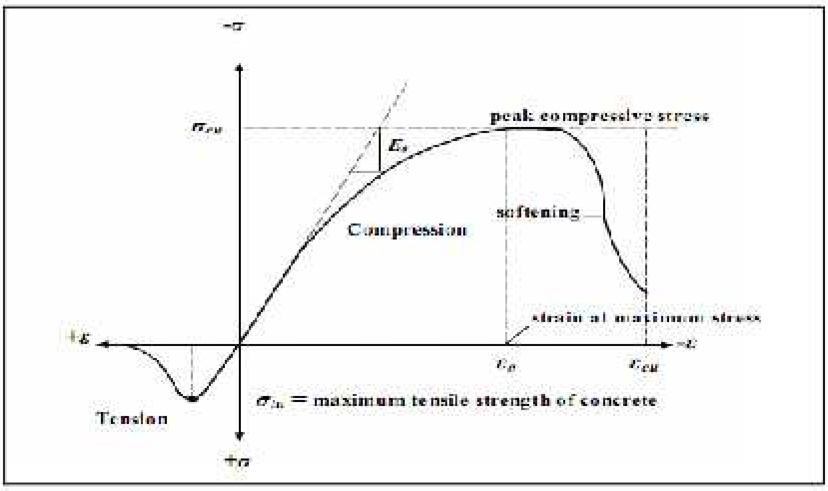

Figure 3-4: Typical uniaxial compressive and tensile stress-strain curve for concrete (*Bangash 1989*)

In compression, the stress-strain curve for concrete is linearly elastic up to about 30 percent of the maximum compressive strength. Above this point, the stress increases gradually up to the maximum compressive strength. After it reaches the maximum compressive strength $\exists_{cu,}$ the curve descends into a softening region, and eventually crushing failure occurs at an ultimate strain $\aleph_{cu.}$. In tension, the stress-strain curve for concrete is approximately linearly elastic up to the maximum tensile strength. After this point, the concrete cracks and the strength decreases gradually to zero (*Bangash 1989*).

3.4.1.1 FEM Input Data

For concrete, the following material properties were used as input parameter:

Elastic modulus (E_c) = 20000 Mpa

Ultimate uniaxial compressive strength (f'_c) = 20 Mpa

Ultimate uniaxial tensile strength (f_t) = 3 Mpa

Poisson's ratio (µ) = 0.2

Shear transfer coefficient ([illegible]) = 0.4

Compressive uniaxial stress-strain relationship for concrete.

3.4.1.2 Compressive Stress-Strain Relationship for Concrete

The MARC program requires the uniaxial stress-strain relationship for concrete in compression. Numerical expressions (*Desayi and Krishnan 1964*), Equations 3-1 and 3-2, were used along with Equation 3-3 (*Gere and Timoshenko 1997*) to construct the uniaxial compressive stress-strain curve for concrete in this study.

$$f = \frac{E_c \varepsilon}{1 + \left(\frac{\varepsilon}{\varepsilon_o}\right)^2} \quad \text{.................................... (3-1)}$$

$$\varepsilon_o = \frac{2 f_c'}{E_c} \quad \text{.................................... (3-2)}$$

$$E_c = \frac{f}{\varepsilon} \quad \text{.................................... (3-3)}$$

Where:

f = stress at any strain ε, Mpa

ε = strain at stress f

ε_0 = strain at the ultimate compressive strength f_c'

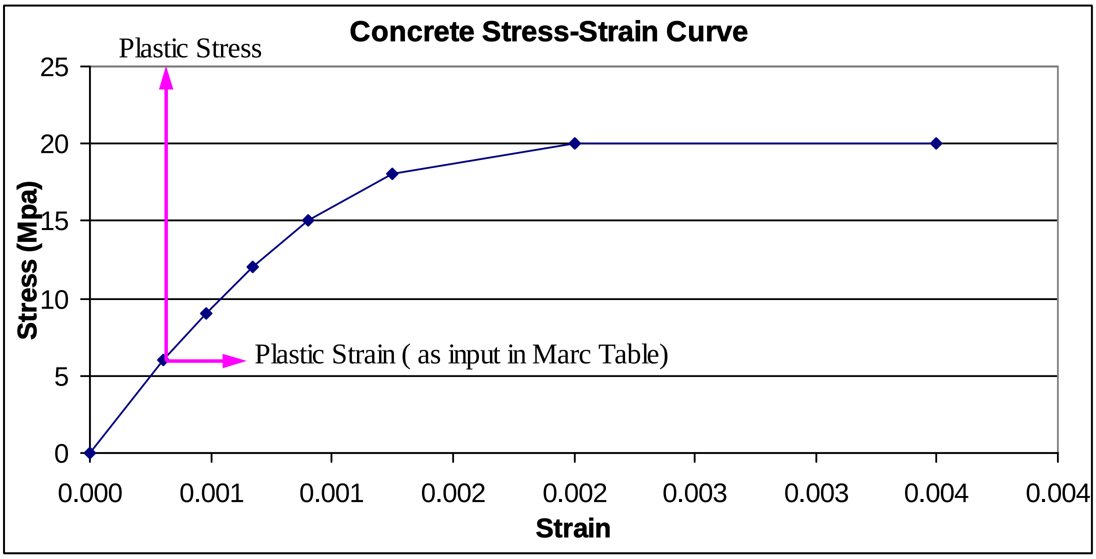

Figure 3-5: Simplified compressive uniaxial stress-strain relationship of concrete that was used in this study.

In MARC2003, the linear part of stress-strain relationship was inputted in the form of modulus of elasticity and non-linear part of stress-strain relationship was inputted in the form of table as shown in the Table 3-1.

Table 3-1 Stress-strain relation of Concrete in compression

General			MARC 2003		
Point	Stress(Mpa)	Strain	Point	Stress(Mpa)	Strain
0	0	0.00000			
1	6	0.00030	1	6	0.00000
2	9	0.00048	2	9	0.00018
3	12	0.00067	3	12	0.00037
4	15	0.00090	4	15	0.00060
5	18	0.00125	5	18	0.00095
6	20	0.00200	6	20	0.00170
7	20	0.00350	7	20	0.00320

3.4.1.3 Tensile Stress-Strain Relationship for Concrete

Tensile strength (f_t) was adopted based on ACI code 318-95(1999):

$$f_t \, X 0.53\sqrt{f_c^{'}} \quad \text{……………………………} \quad (3\text{-}4)$$

Concrete behavior in tension was assumed as linearly elastic as shown in figure below. After principal stress in tension reached f_t , concrete would crack and , soften linearly until a strain of 0.005 (Filippou 2007) where it would completely looses its tensile load carrying capacity. Tensile softening modulus was calculated with this assumption.

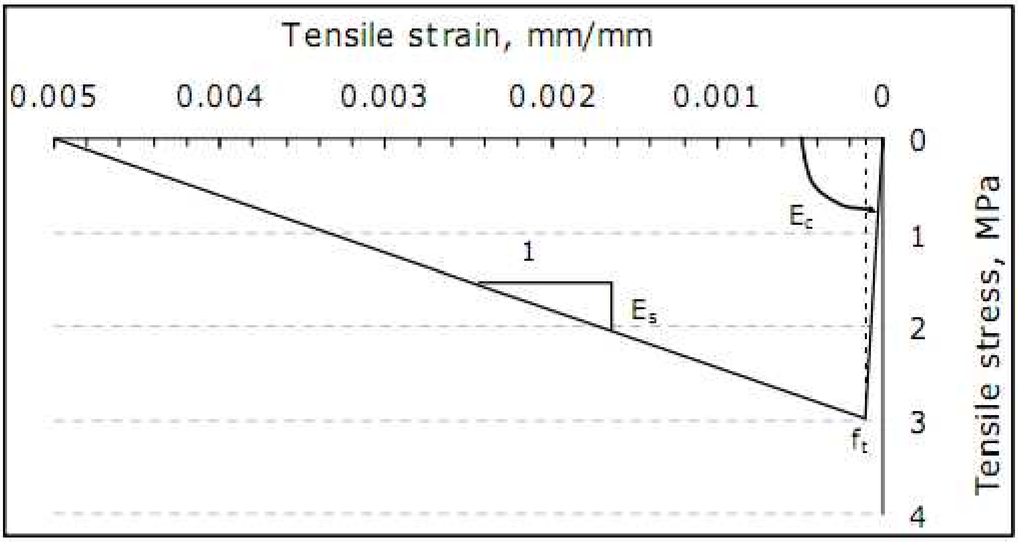

Figure 3-6: Constitutive model for normal strength concrete in tension

3.4.1.4 Failure Criteria for Concrete

Following failure criteria are assumed in MARC2003 for the failure of concrete:-

Cracking strain = 0.0002

Concrete failure range (strain) = 0.003 to 0.005

Shear strength of concrete = 3.0 Mpa

Shear transfer Coefficient (β_t) = 0.4 (WCOM-D)

Softening modulus = 615 Mpa

Crushing strain = 0.005

3.4.2 Steel Reinforcement (Rebar)

Steel reinforcement used in this research work was Fe415 steel reinforcing bars. Elastic modulus and yield stress, for the steel reinforcement used in this FEM study follow the design material properties proposed in the *Reinforced concrete Design* book (*Pillai and Menon,2nd edition*). The steel for the finite element models was assumed to be elastic- plastic material and identical in tension and compression. Poisson's ratio of 0.3 was used for the steel reinforcement in this study (*Gere and Timoshenko1997*). Figure 3-7 shows the stress-strain relationship used in this study. Material properties for the steel reinforcement for all models are as follows:

Elastic modulus, E_s = 200,000 MPa
Yield stress, f_y = 415 MPa
Poisson's ratio, ⇒ = 0.3

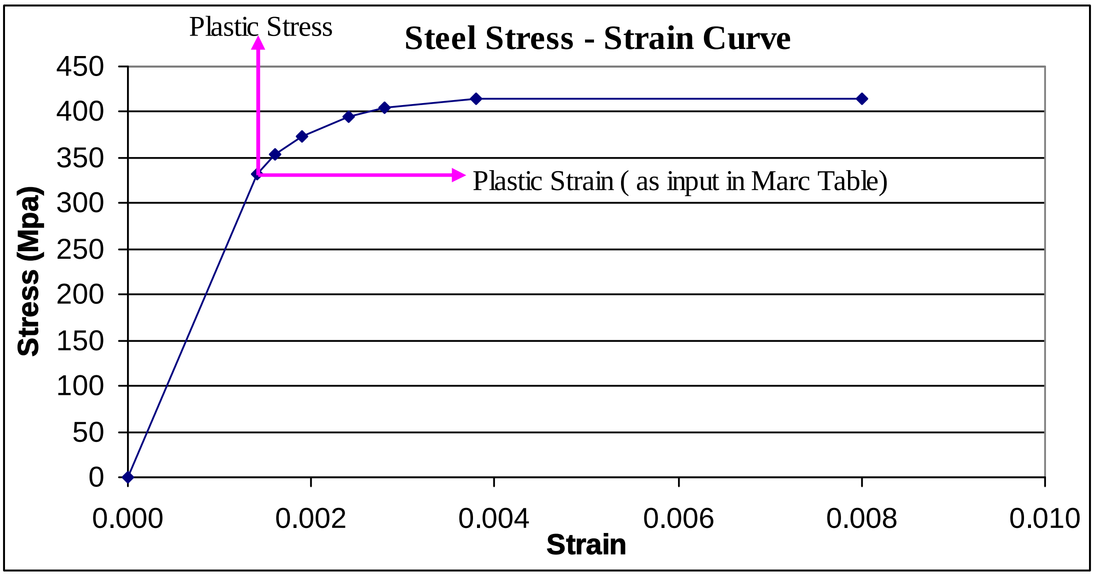

Figure 3-7: Tensile uniaxial stress-strain relationship for steel that was used in this study.

In MARC2003, the linear part of stress-strain relationship was inputted in the form of modulus of elasticity and non-linear part of stress-strain relationship was inputted in the form of table as shown in the Table 3-2.

Table 3-2 Stress-strain relation of steel in tension

General			MARC 2003		
Point	Stress(Mpa)	Strain	Point	Stress(Mpa)	Strain
0	0	0.0000			
1	332	0.0014	1	332	0.00000
2	353	0.0016	2	353	0.00020
3	374	0.0019	3	374	0.00050
4	394	0.0024	4	394	0.00100
5	405	0.0028	5	405	0.00140
6	415	0.0038	6	415	0.00240
7	415	0.0080	7	415	0.00660

3.4.3 CFRP Composites

FRP composites are materials that consist of two constituents. The constituents are combined at a macroscopic level and are not soluble in each other. One constituent is the reinforcement, which is embedded in the second constituent, a continuous polymer called the matrix (*Kaw1997*). The reinforcing material is in the form of fibers, i.e., carbon and glass, which are typically stiffer and stronger than the matrix. The FRP composites are anisotropic materials; that is, their properties are not the same in all directions. Figure 3-8 shows a schematic of FRP composites.

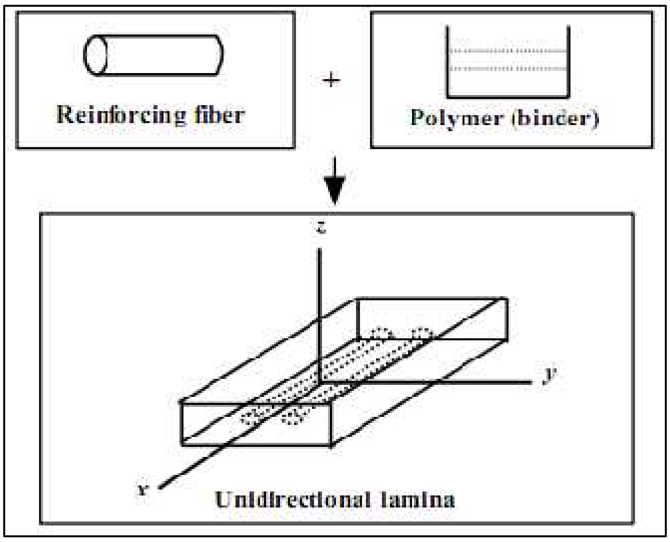

Figure 3-8: Schematic of FRP composites (*Gibson 1994, Kaw 1997*)

As shown in Figure 3-8, the unidirectional lamina has three mutually orthogonal planes of material properties (i.e., *xy*, *xz*, and *yz* planes). The *xyz* coordinate axes are referred to as the principal material coordinates where the *x* direction is the same as the fiber direction, and the *y* and *z* directions are perpendicular to the *x* direction. It is a so-called specially orthotropic material (*Gibson 1994, Kaw 1997*). In this study, the specially orthotropic material is also transversely isotropic, where the properties of the FRP composites are nearly the same in any direction perpendicular to the fibers. Thus, the properties in the *y* direction are the same as those in the z direction.

Glass fiber reinforced polymer was used for shear reinforcement on the Horsetail Falls Bridge because of its superior strain at failure. Carbon fiber reinforced polymer was used for flexural reinforcement because of its high tensile strength. Linear elastic properties of the FRP composites were assumed throughout this study. Figure 3-9 shows the stress-strain curves used in this study for the FRP composites in the direction of the fiber.

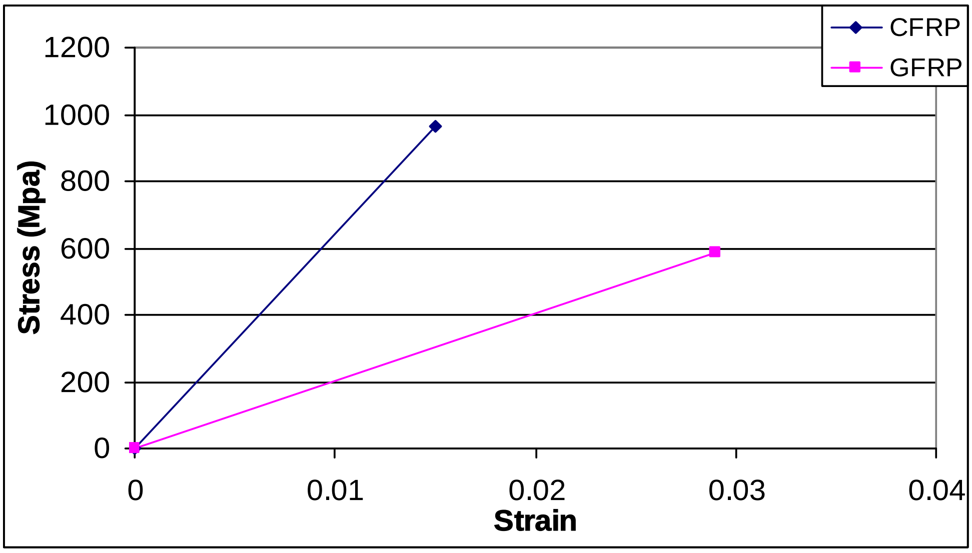

Figure 3-9: Stress-strain curves for the FRP composites in the direction of the fibers.

Input data needed for the CFRP composites in the finite element models are as follows:

- Number of layers.
- Thickness of each layer.
- Orientation of the fiber direction for each layer.

- Elastic modulus of the FRP composite in three directions (E_x, E_y and E_z).
- Shear modulus of the FRP composite for three planes (G_{xy}, G_{yz} and G_{xz}).
- Major Poisson's ratio for three planes (v_{xy}, v_{yz} and v_{xz}).

Note that a local coordinate system for the FRP layered solid elements is defined where the *x* direction is the same as the fiber direction, while the *y* and *z* directions are perpendicular to the *x* direction.

The properties of isotropic materials, such as elastic modulus and Poisson's ratio, are identical in all directions; therefore no subscripts are required. This is not the case with specially orthotropic materials. Subscripts are needed to define properties in the various directions. For example, $E_x \neq E_y$ and $\nu_{xy} \neq \nu_{yx}$. E_x is the elastic modulus in the fiber direction, and E_y is the elastic modulus in the y direction perpendicular to the fiber direction. The use of Poisson's ratios for the orthotropic materials causes confusion; therefore, the orthotropic material data are supplied in the v_{xy} or major Poisson's ratio format for the MARC program. The major Poisson's ratio is, the ratio of strain in the *y* direction to strain in the perpendicular *x* direction when the applied stress is in the *x* direction. The quantity v_{yx} is called a minor Poisson's ratio and is smaller than v_{xy}, whereas E_x is larger than E_y. Equation 3-5 shows the relationship between v_{xy} and v_{yx} (*Kaw 1997*).

$$\nu_{yx} = \frac{E_y}{E_x} \nu_{xy} \qquad \text{……….. 3-5}$$

where:

ν_{yx} = Minor Poisson's ratio

E_x = Elastic modulus in the *x* direction (fiber direction)

E_y = Elastic modulus in the *y* direction

ν_{xy} = Major Poisson's ratio

A summary of material properties used for the modeling of all beams is shown in Table 3-3.

Table 3-3: Material properties for FRP composites (*Kachlakev and McCurry 2000*)

FRP Composite	Elastic Modulus (Mpa)	Major Poisson's Ratio	Tensile Strength (Mpa)	Shear Modulus (Mpa)
CFRP	E_x = 63000	|||||⇒$_{xy}$ = 0.22|	965	|||||G$_{xy}$ = 3270 |
	E_y = 4800	⇒$_{xz}$ = 0.22|		|||||G$_{xz}$ = 3270 |
	E_z = 4800	|⇒$_{yz}$ = 0.30 |		|||||G$_{yz}$ = 1860 |

3.5 Geometrical Configuration of the Beam

Rebar is expressed in the ratio of cross-section of beam. Offset of CFRP from support means the distance between support and end of CFRP. It is expressed in the ratio of span of beam.

3.5.1 Laboratory Scale Beam

The dimension of this beam is 4000 mm x 300 mm x 150 mm (LxDxB). The span between the two supports was 4000 mm. Rebar provided was 0.33% & 0.75%. Offset of CFRP from support is kept 0%, 5%, 10%, 15%, 20%, 25%, 30%, 35% and 40% corresponding to 0.33% and 0.75% of rebar. The dimensions for all beams are shown below:-

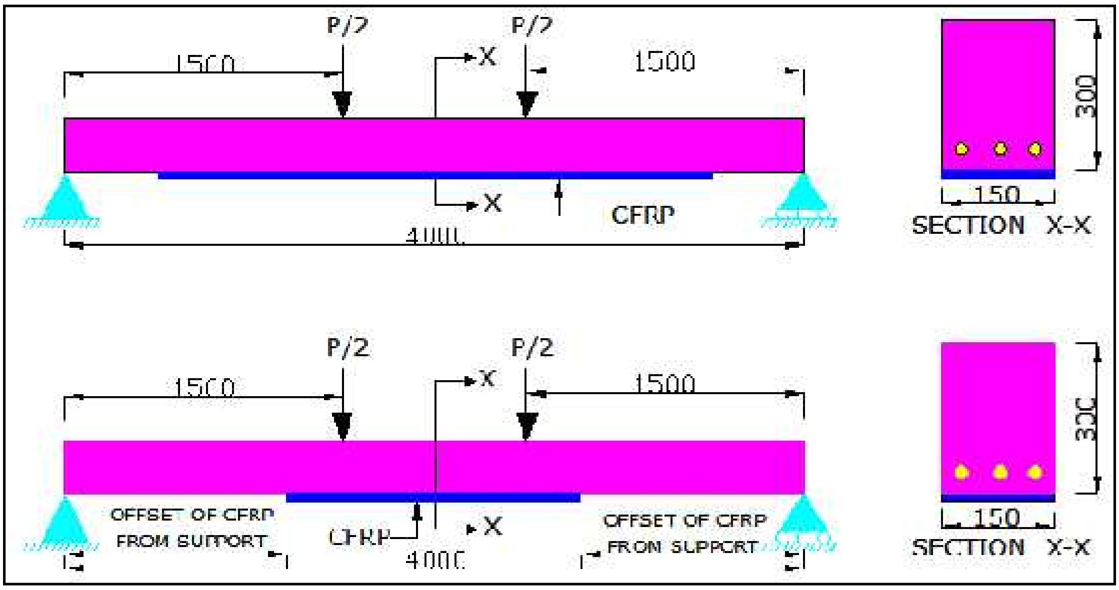

Figure 3-10: Laboratory Scale Beam

3.5.2 Building Specific Beam

The dimension of this beam is 5000 mm x 450 mm x 300 mm (LxDxB). The span between the two supports was 5000 mm. Rebar provided was 0.33% & 0.75%.

Offset of CFRP from support is kept 0%, 6%, 12%, 18%, 24%, 30%, 36% and 42% corresponding to 0.33% rebar. The dimensions for all beams are shown below:-

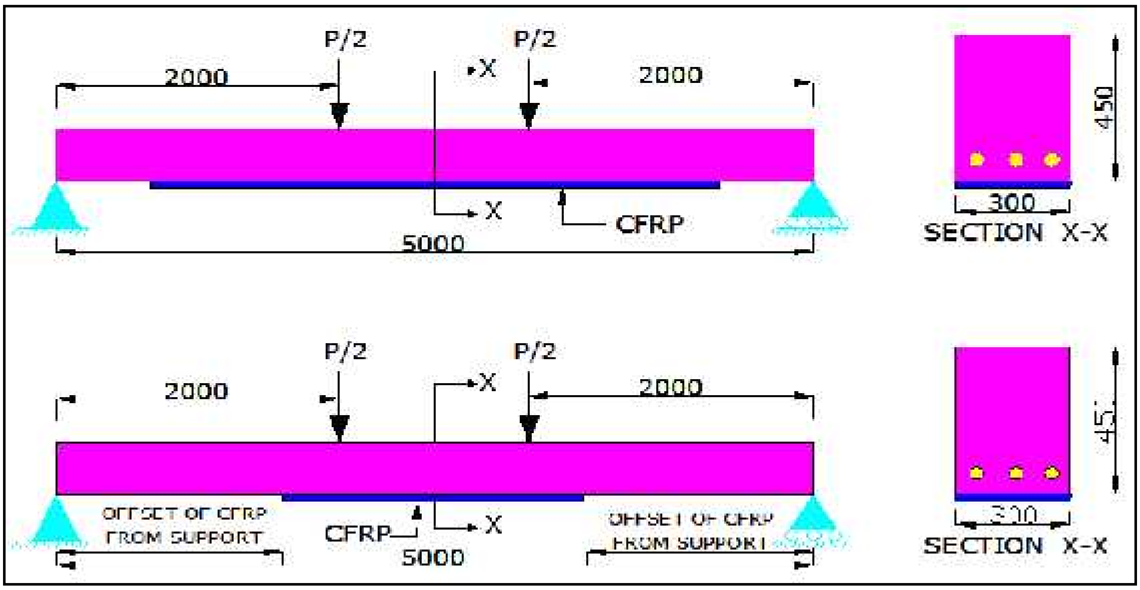

Figure 3-11: Building Specific Beam

3.5.3 Bridge Specific Beam

The dimension of the full-size beams was 20000 mm x 1800 mm x 500 mm (LxDxB). The span between the two supports was 20000 mm. Rebar provided was 0.33% & 0.75%. Offset of CFRP from support is kept 0%, 5%, 10%, 15%, 20%, 25%, 30%, 35% and 40% corresponding to 0.33% rebar. The dimensions for all beams are shown below:-

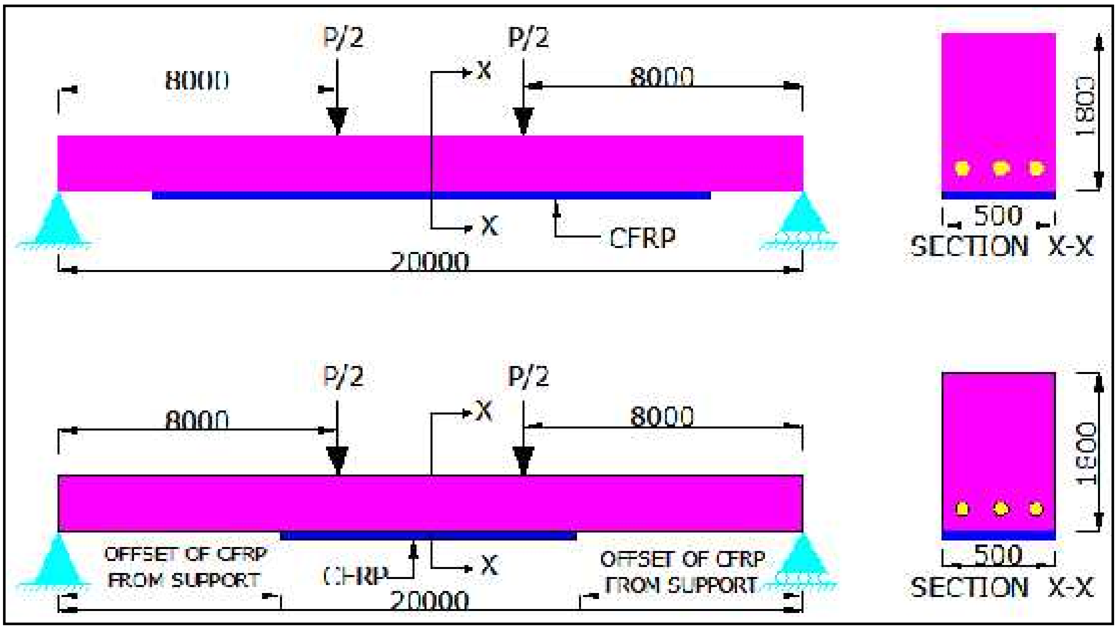

Figure 3-12: Bridge Specific Beam

3.6 Non-Linear Solution

In nonlinear analysis, the total load applied to a finite element model is divided into a series of load increments called load steps. At the completion of each incremental solution, the stiffness matrix of the model is adjusted to reflect nonlinear changes in structural stiffness before proceeding to the next load increment. Newton- Raphson equilibrium iterations for updating the model stiffness can be used in the MARC program (Marc2003).

Newton-Raphson equilibrium iterations provide convergence at the end of each load increment within tolerance limits. Figure shows the use of the Newton-Raphson approach in a single degree of freedom nonlinear analysis.

Prior to each solution, the Newton-Raphson approach assesses the out-of-balance load vector, which is the difference between the restoring forces (the loads corresponding to the element stresses) and the applied loads. Subsequently, the program carries out a linear solution, using the out-of-balance loads, and checks for convergence. If convergence criteria are not satisfied, the out-of-balance load vector is re-evaluated, the stiffness matrix is updated, and a new solution is attained. This iterative procedure continues until the problem converges (Marc 2003).

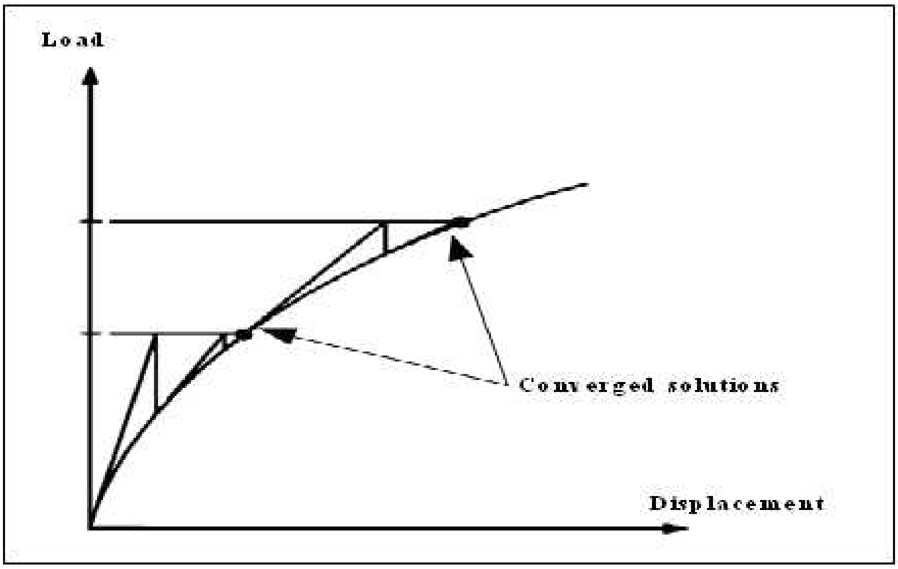

Figure 3-13: Newton-Raphson Solution

In this study, for the reinforced concrete solid elements, convergence criteria were based on force and displacement, and the convergence tolerance limits were initially selected by the MARC program. It was found that convergence of solutions for the models was difficult to achieve due to the nonlinear behavior of reinforced concrete. Therefore, the convergence tolerance limits were increased to a maximum of 1.5 times the default tolerance limits (8 % for force checking and 8% for displacement checking) in order to obtain convergence of the solutions.

3.7 Load Stepping and Failure Definition for FE Models

For the nonlinear analysis, automatic time stepping in the MARC program predicts and controls load step sizes. Based on the previous solution history and the physics of the models, if the convergence behavior is smooth, automatic time stepping will increase the load increment up to a selected maximum load step size. If the convergence behavior is abrupt, automatic time stepping will bisect the load increment until it is equal to a selected minimum load step size. The maximum and minimum load step sizes are required for the automatic time stepping. In this study, the convergence behavior of the models depended on behavior of the reinforced concrete.

CHAPTER 4

Theoretical Prediction and Analytical Study

4.1 General

Experimental study of strengthening of reinforced concrete beam under monotonic loading requires a lot of samples with different geometry like laboratory scale beam, building specific beam and bridge specific beam which is not possible in our laboratory due to limitation of resources. So analytical studies using MARC program (Marc2003) has been taken in this thesis work. In the FEM program, an 8-nodded brick element is used for concrete and rebar was modeled as 8-node empty brick in which we can place single strain members such as reinforcing rods or cords (that is, rebars). The element is then used in conjunction with the 8-node brick continuum element (for example, element types 7) to represent cord reinforced composite materials. This technique allows the rebar and the filler to be represented accurately with respect to their stress distribution, so that separate constitutive theories can be used in each (for example, cracking concrete and yield rebar). The position, size, and orientation of the rebars are input through the REBAR option. CFRP Composite is modeled by using four-node, thin-shell element with global displacements and rotations as degrees of freedom. The constitutive law adopted for the cracked concrete is composed of the tension-stiffening model, the compression-stiffening model, and the shear transfer model.

4.2 Damage Classification

Damage can be classified as Failure Damage, Considerable Damage and Light Damage. These indices are used for judging the degree of the damage.

i. Failure Damage

A failure condition represents the situation when the structure cannot be repaired and must be demolished.

ii. Considerable/Light Damage

The Considerable damage and the light damage conditions represent the situation when the structure can be repaired/retrofitted after external forces are applied. An approximate trial calculation for a repair/retrofitting cost may be performed based on the damage condition and the residual displacement/deformation of the whole structure. Designers would design weighing the repair/retrofitting cost and the return period of an earthquake against the initial cost.

iii. No Damage

No damage condition represents the situation that the repair/retrofitting need not be conducted. This is the most desirable situation for the designers.

4.3 Element Size

The element size used in the analysis was different for different beams. The element size used was described below:

i) Laboratory scale Beam

The element size used in laboratory scale beam of size 150x300 mm was 100x50x50 mm (lxbxh). The maximum aspect ratio is two.

ii) Building specific Beam

The element size used in building specific beam of size 300x450 mm was 100x50x50 mm (lxbxh). The maximum aspect ratio is two.

iii) Bridge specific Beam

The element size used in bridge specific beam of size 500x1800 mm was 500x100x200 mm (lxbxh). The maximum aspect ratio is five.

4.4 Boundary Condition

4.4.1 Displacement

All the analyses were done for simply supported Beam. For this, the left end bottom nodes of all beams are locked for x-displacement, y-displacement & z-displacement to idealize as hinge support. The right end bottom nodes of all beams are locked for y-displacement & z-displacement to idealize as roller support. The displacement boundary condition is shown in the figure 4-1.

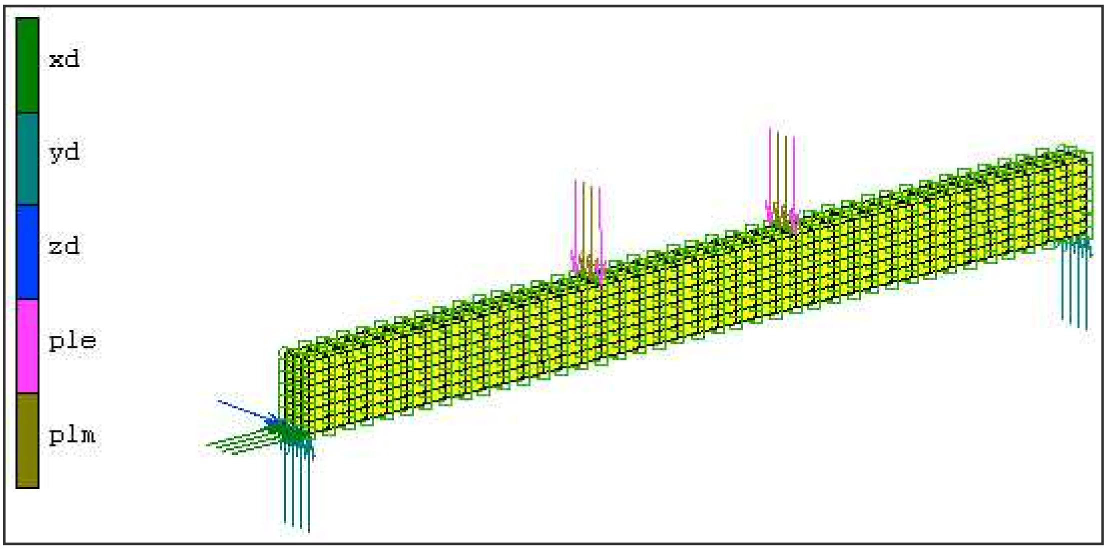

Figure 4-1: Boundary Condition

4.4.2 Load

The load applied on the beam in terms of monotonic point load. The point load is applied at a distance of 40% of span from the both support which is shown in the figure 4-1. The half point load is applied at edge node and full point load is applied at interior nodes as shown in figure 4-1. The total load applied on the beam was divided into number of steps of load to attain full load.

4.5 Parametric Study

4.5.1 Beam Size

Three types of beam were studied with different span. They are laboratory scale beam, building specific beam and bridge specific beam. The size of laboratory scale beam is 4000x150x300 mm (lxbxh), the size of building specific beam is 5000x300x450 mm (lxbxh) and the size of bridge specific beam is 20000x500x1800 mm (lxbxh).

4.5.2 Rebar Percent

There are two different percentages of rebar used for all of the beams. They are 0.33% and 0.75%.

4.5.3 CFRP Percent

There are different percentages of CFRP used for all of the beams. They are 0.17%, 0.33%, 0.50%, 0.67%, 0.83% and 1.00%.

4.5.3 Offset of CFRP from support

There are different percentages of offsets of CFRP from support used for all of the beams. They are 0%, 5%, 10%, 15%, 20%, 25%, 30%, 35% and 40%.

CHAPTER 5

Results and Discussions

5.1 Results

From the analysis, the results were obtained in the form of load displacement curves, cracking strain, stress on rebar and strain at extreme tension and compression fiber of the beam. These results are shown in **Fig.B-01** to **Fig.H-03**. **Fig.B-01** to **Fig.B-06** shows the load displacement curves of load at 40% of span and mid-span displacement. **Fig.B-01** to **Fig.B-03** shows the load displacement curve of 0.33% beam reinforcement and **Fig.B-03** to **Fig.B-06** shows the load displacement curve with 0.75% beam reinforcement. The point of cracking of beam, yielding of rebar and failure of beam were shown in the curve. **Fig.C-01** to **Fig.C-06** shows the moment capacity enhancement of the beam with the use of different percentage of rebar. **Fig.C-01** to **Fig.C-03** shows the moment Vs CFRP percentage curve with 0.33% rebar and **Fig.C-03** to **Fig.C-06** shows the moment Vs CFRP percentage curve with 0.75%rebar. These curves show that with the increase of CFRP percent, the moment capacity of the beam was increased. The increment of moment capacity is large up to certain limit (0.5% of CFRP corresponding to 0.33% rabar and 0.17% of CFRP corresponding to 0.75% rebar). After this limit the moment Vs CFRP percent curve goes to be flat. **Fig.D-01** t**o Fig.D-12** shows the stress on CFRP at yielding of rebar and at failure of beam. This curve shows that with the increase of CFRP percent, the stress on CFRP decreases. **Fig.E-01** to **Fig.E-06** shows the displacement control of CFRP at fixed Load. This curve shows that with the increase of CFRP percent, the displacement of beam goes on decreases at fixed load. **Fig.F-01** to **Fig.F-12** shows the Load Vs offset of CFRP from Support Curve. This curve shows that there is not any change in load carrying capacity of beam up to certain offset of CFRP from support. After that limiting offset, the load carrying capacity of the beam is decreases with the increase of offset of CFRP from support. **Fig.G-01** to **Fig.G-03** shows the effect of offset of CFRP on the deflection of the Beam. This curve shows that there is not any change in deflection of beam up to certain offset of CFRP from support. After that limiting offset, the deflection of the beam is increases with the increase of offset of CFRP from support. **Fig.H-01** to **Fig.H-03** shows the curve between optimum percent of CFRP corresponding to the rebar percent of beam. **Fig.I-01** to **Fig.I-03** shows the load corresponding to the final deflection of beam allowed in the design (span/250;

IS456:2000). This curve shows that with the increase of CFRP percent, the load carrying capacity of the beam is increases at fixed displacement. **Fig.J-01** to **Fig.J-06** shows the crack pattern, boundary condition and material, failure of CFRP.

Table-A01 to **Table-A03** shows the different percent of CFRP and corresponding cracking load, yielding load and ultimate load in laboratory scale beam with 0.33% rebar, **Table-A06** to **Table-A08** shows the different percent of CFRP and corresponding cracking load, yielding load and ultimate load in laboratory scale beam with 0.75% rebar, **Table-A11** to **Table-A13** shows the different percent of CFRP and corresponding cracking load, yielding load and ultimate load in building specific beam with 0.33% rebar, **Table-A16** to **Table-A18** shows the different percent of CFRP and corresponding cracking load, yielding load and ultimate load in building specific beam with 0.33% rebar, **Table-A21** to **Table-A23** shows the different percent of CFRP and corresponding cracking load, yielding load and ultimate load in bridge specific beam with 0.33% rebar & **Table-A26** to **Table-A28** shows the different percent of CFRP and corresponding cracking load, yielding load and ultimate load in bridge specific beam with 0.75% rebar. **Table-A04** shows the stresses on CFRP at yielding of rebar and ultimate load in laboratory scale beam with 0.33% rebar, **Table-A09** shows the stresses on CFRP at yielding of rebar and ultimate load in laboratory scale beam with 0.75% rebar**, Table-A14** shows the stresses on CFRP at yielding of rebar and ultimate load in building specific beam with 0.33% rebar, **Table-A19** shows the stresses on CFRP at yielding of rebar and ultimate load in building specific beam with 0.75% rebar, **Table-A24** shows the stresses on CFRP at yielding of rebar and ultimate load in bridge specific beam with 0.33% rebar **& Table-A29** shows the stresses on CFRP at yielding of rebar and ultimate load in bridge specific beam with 0.75% rebar. **Table-A05** shows the displacement control of CFRP in laboratory scale beam with 0.33% rebar at fixed load 40KN, **Table-A10** shows the displacement control of CFRP in laboratory scale beam with 0.75% rebar at fixed load 70KN**, Table-A15** shows the displacement control of CFRP in building specific beam with 0.33% rebar at fixed load 140KN, **Table-A20** shows the displacement control of CFRP in building specific beam with 0.75% rebar at fixed load 220KN**, Table-A25** shows the displacement control of CFRP in bridge specific beam with 0.33% rebar at fixed load 1400KN **& Table-A30** shows the displacement control of CFRP in bridge specific beam with 0.75% rebar at fixed load 2100KN. **Table-A31** to **Table-A33** shows the different percentage of offset of CFRP from support and corresponding cracking load, yielding load and ultimate load in laboratory scale beam with 0.33% rebar, **Table-A36** to **Table-A38** shows the different percentage of offset of CFRP from support and corresponding cracking load, yielding load and ultimate load in building specific beam with 0.33% rebar **& Table-A41** to **Table-A43** shows the different percentage of offset of CFRP from support and corresponding cracking load, yielding load and ultimate load in bridge specific beam with 0.33% rebar. **Table-A34** shows the effect of offset of CFRP from support on the stresses developed in CFRP at Yielding of rebar and failure of beam in laboratory scale beam with 0.33% rebar, **Table-A39** shows the effect of offset of CFRP from support on the stresses developed in CFRP at Yielding of rebar and failure of beam in building specific beam with 0.33% rebar **& Table-A44** shows the effect of offset of CFRP from support on the stresses developed in CFRP at Yielding of rebar and failure of beam in bridge specific beam with 0.33% rebar. **Table-A35** shows the effect of offset of CFRP from support on the displacement of laboratory scale beam with 0.33% rebar at fixed load 30KN, **Table-A40** shows the effect of offset of CFRP from support on the displacement of building specific beam with 0.33% rebar at fixed load 100KN **& Table-A45** shows the effect of offset of CFRP from support on the displacement of bridge specific beam with 0.33% rebar at fixed load 1150KN. **Table-A46** Shows the optimum percent of CFRP corresponding to the rebar percent of beam. **Table-A47** shows the load corresponding to the final deflection of laboratory scale beam (with 0.33% rebar) allowed in the design (span/250; IS456:2000), **Table-A48** shows the load corresponding to the final deflection of building specific beam (with 0.33% rebar) allowed in the design (span/250; IS456:2000) & **Table-A49** shows the load corresponding to the final deflection of bridge specific beam (with 0.33% rebar) allowed in the design (span/250; IS456:2000).

5.2 Discussions from the Study

The results obtained from the analytical study carried out in this thesis work are discussed below

5.2.1 Laboratory scale Beam (0.33% rebar)

5.2.1.1 Effect of CFRP percent on the load carrying capacity of beam

a) The load carrying capacity of strengthened beam is increased with the increase of CFRP percent (**Fig.B-1**). The maximum percent of CFRP used was 1.00%. CFRP up to 0.5% was effectively utilized and the use of CFRP more than 0.5% the beam becomes over-reinforced & failure of beam takes place in compression in stead of tension.

b) The enhancement of cracking moment is gradual due to the use of CFRP from 0.17% to 1.0% (**Fig.-C01**). The total increase in cracking moment capacity is 30.86%.

c) The enhancement of yielding moment was gradually up to 0.5% of CFRP but after 0.5% of CFRP the yielding moment capacity is not gradual and going to be flat (**Fig.-C01**). The total increase in yielding moment capacity is 80.72%. Among this 61.3% capacity enhancement at 0.5% CFRP and remaining 38.7% capacity enhancement at 1.0% of CFRP.

d) The enhancement of ultimate moment was gradually up to 0.5% of CFRP but after 0.5% of CFRP the ultimate moment capacity is not gradual and going to be flat (**Fig.-C01**). The total increase in ultimate moment capacity is 200%. Among this 69.06% capacity enhancement at 0.5% CFRP and remaining 30.94% capacity enhancement at 1.0% of CFRP.

5.2.1.2 Effect of CFRP percent on the stresses developed in CFRP

a) The stress on CFRP goes on decreasing at yielding of rebar with the increase of CFRP percent (**Fig.-D01**). The stress on CFRP decreases because with the increase of CFRP the stiffness of the beam is also increases which decrease the strain on CFRP.

b) The stress on CFRP was constant at ultimate load up to 0.5% of CFRP (**Fig.-D02**). The stress on CFRP was decreases after 0.5% of CFRP because with the increase of CFRP percent the beam becomes over-reinforced and fails in compression.

5.2.1.3 Effect of CFRP percent on the displacement of beam

a) The displacement of the beam decreases with the increase of CFRP percent (**Fig.-E01**) at fixed load (40 KN). The decrease in displacement is gradual up to 0.5% CFRP but after that the curve going to be flat. The total decrease in displacement is 72.91%.

b) The load carrying capacity enhancement corresponding to the final deflection of beam allowed in design (span/250; IS 456-2000) is 46.32%. The curve between Load Vs CFRP (%) is linear because the load corresponding to the final deflection allowed in the design is less than the yielding load.

5.2.1.4 Effect of offset of CFRP from support on the load carrying capacity of beam

a) There is not any change in cracking load up to the offset of 30% from the support (**Fig.-F01**). After 30% of offset, the cracking load is decreased because the stress concentration takes place on the concrete element without CFRP which is located just after the element with CFRP.

b) There is not any change in yielding load up to the offset of 30% from the support (**Fig.-F02**). After 30% of offset, the yielding load is decreased because the stress concentration takes place on the concrete element without CFRP which is located just after the element with CFRP.

c) There is not any change in ultimate load up to the offset of 15% from the support (**Fig.-F03**). After 15% of offset, the ultimate load is decreased because the stress concentration takes place on the concrete element without CFRP which is located just after the element with CFRP.

5.2.2 Building Specific Beam (0.33% Rebar)

5.2.2.1 Effect of CFRP percent on the load carrying capacity of beam

a) The load carrying capacity of strengthened beam was increased with the increase of CFRP percent (**Fig.B-2**). The maximum percent of CFRP used was 1.0%. CFRP up to 0.5% was effectively utilized and the use of CFRP more than 0.5% the beam becomes over-reinforced & failure of beam takes place in compression in stead of tension.

b) The enhancement of cracking moment is gradual due to the use of CFRP from 0.17% to 1.0% (**Fig.-C02**). The total increase in cracking moment capacity is 53.91%.

c) The enhancement of yielding moment was gradually up to 0.5% of CFRP but after 0.5% of CFRP the yielding moment capacity is not gradual and going to be flat (**Fig.-C02**). The total increase in yielding moment capacity is 95.64%. Among this 56.75% capacity enhancement at 0.5% CFRP and remaining 43.25% capacity enhancement at 1.0% of CFRP.

d) The enhancement of ultimate moment was gradually up to 0.5% of CFRP but after 0.5% of CFRP the ultimate moment capacity is not gradual and going to be flat (**Fig.-C02**). The total increase in ultimate moment capacity is 208.75%. Among this 60.33% capacity enhancement at 0.5% CFRP and remaining 39.67% capacity enhancement at 1.0% of CFRP.

5.2.2.2 Effect of CFRP percent on the stresses developed in CFRP

a) The stresses on CFRP were goes on decreasing at yielding of rebar with the increase of CFRP percent (**Fig.-D03**). The stress on CFRP was decreases because with the increase of CFRP the stiffness of the beam is also increases which decrease the strain on CFRP.

b) The stresses on CFRP were constant at ultimate load up to 0.5% of CFRP (**Fig.-D04**). The stress on CFRP was decreases after 0.5% of CFRP because with the increase of CFRP percent the beam becomes over-reinforced and fails in compression.

5.2.2.3 Effect of CFRP percent on the displacement of beam

a) The displacement of the beam decreases with the increase of CFRP percent (**Fig.-E02**) at fixed load (140 KN). The decrease in displacement is gradual up to 0.5% CFRP but after that the curve going to be flat. The total decrease in displacement is 75.41%.

b) The load carrying capacity enhancement corresponding to the final deflection of beam allowed in design (span/250; IS 456:2000) is 80.29%. The curve between Load Vs CFRP (%) is linear because the load corresponding to the final deflection allowed in the design is less than the yielding load.

5.2.2.4 Effect of offset of CFRP from support on the load carrying capacity of beam

a) There is not any change in cracking load up to the offset of 25% from the support (**Fig.-F04**). After 25% of offset, the cracking load is decreased because the stress concentration takes place on the concrete element without CFRP which is located just after the element with CFRP.

b) There is not any change in yielding load up to the offset of 25% from the support (**Fig.-F05**). After 25% of offset, the yielding load is decreased because the stress concentration takes place on the concrete element without CFRP which is located just after the element with CFRP.

c) There is not any change in ultimate load up to the offset of 20% from the support (**Fig.-F06**). After 20% of offset, the ultimate load is decreased because the stress concentration takes place on the concrete element without CFRP which is located just after the element with CFRP.

5.2.3 Bridge Specific Beam(0.33% Rebar)

5.2.3.1 Effect of CFRP percent on the load carrying capacity of beam

a) The load carrying capacity of strengthened beam was increased with the increase of CFRP percent (**Fig.B-3**). The maximum percent of CFRP used was 1.0%. CFRP up to 0.5% was effectively utilized and the use of CFRP more than 0.5% the beam becomes over-reinforced & failure of beam takes place in compression in stead of tension.

b) The enhancement of cracking moment is gradual due to the use of CFRP from 0.17% to 1.0% (**Fig.-C03**). The total increase in cracking moment capacity is 43.56%.

c) The enhancement of yielding moment was gradually up to 0.5% of CFRP but after 0.5% of CFRP the yielding moment capacity is not gradual and going to be flat (**Fig.-C03**). The total increase in yielding moment capacity is 51.37%. Among this 63.55% capacity enhancement at 0.5% CFRP and remaining 36.45% capacity enhancement at 1.0% of CFRP.

d) The enhancement of ultimate moment was gradually up to 0.5% of CFRP but after 0.5% of CFRP the ultimate moment capacity is not gradual and going to be flat (**Fig.-C03**). The total increase in ultimate moment capacity is 87.85%. Among this 85.04% capacity enhancement at 0.5% CFRP and remaining 14.96% capacity enhancement at 1.0% of CFRP.

5.2.3.2 Effect of CFRP percent on the stresses developed in CFRP

a) The stresses on CFRP were goes on decreasing at yielding of rebar with the increase of CFRP percent (**Fig.-D05**). The stress on CFRP was decreases because with the increase of CFRP the stiffness of the beam is also increases which decrease the strain on CFRP.

b) The stresses on CFRP were constant at ultimate load up to 0.5% of CFRP (**Fig.-D06**). The stress on CFRP was decreases after 0.5% of CFRP because with the increase of CFRP percent the beam becomes over-reinforced and fails in compression.

5.2.3.3 Effect of CFRP percent on the displacement of beam

a) The displacement of the beam decreases with the increase of CFRP percent (**Fig.-E03**) at fixed load (1400 KN). The decrease in displacement is gradual up to 0.5% CFRP but after that the curve going to be flat. The total decrease in displacement is 66.18%.

b) The load carrying capacity enhancement corresponding to the final deflection of beam allowed in design (span/250) is 31.96%. The curve between Load Vs CFRP (%) is linear because the load corresponding to the final deflection allowed in the design is less than the yielding load.

5.2.3.4 Effect of offset of CFRP from support on the load carrying capacity of beam

a) There is not any change in cracking load up to the offset of 20% from the support (**Fig.-F07**). After 20% of offset, the cracking load is decreased because the stress concentration takes place on the concrete element without CFRP which is located just after the element with CFRP.

b) There is not any change in yielding load up to the offset of 25% from the support (**Fig.-F08**). After 25% of offset, the yielding load is decreased because the stress concentration takes place on the concrete element without CFRP which is located just after the element with CFRP.

c) There is not any change in ultimate load up to the offset of 25% from the support (**Fig.-F09**). After 25% of offset, the ultimate load is decreased because the stress concentration takes place on the concrete element without CFRP which is located just after the element with CFRP.

5.2.4 Laboratory scale Beam (0.75% rebar)

5.2.4.1 Effect of CFRP percent on the load carrying capacity of beam

a) The load carrying capacity of strengthened beam was increased with the increase of CFRP percent (**Fig.B-4**). The maximum percent of CFRP used was 1.00%. CFRP up to 0.17% was effectively utilized and the use of CFRP more than 0.17% the beam becomes over-reinforced & failure of beam takes place in compression in stead of tension.

b) The enhancement of cracking moment is gradual due to the use of CFRP from 0.17% to 1.0% (**Fig.-C04**). The total increase in cracking moment capacity is 39.53%.

c) The enhancement of yielding moment was gradually up to 0.17% of CFRP but after 0.17% of CFRP the yielding moment capacity is not gradual and going to be flat (**Fig.-C04**). The total increase in yielding moment capacity is 51.94%. Among this 27.27% capacity enhancement at 0.17% CFRP and remaining 72.73% capacity enhancement at 1.0% of CFRP.

d) The enhancement of ultimate moment was gradually up to 0.17% of CFRP but after 0.17% of CFRP the ultimate moment capacity is not gradual and going to be flat (**Fig.-C04**). The total increase in ultimate moment capacity is 83.75%. Among this 29.70% capacity enhancement at 0.17% CFRP and remaining 70.3% capacity enhancement at 1.0% of CFRP.

5.2.4.2 Effect of CFRP percent on the stresses developed in CFRP

a) The stress on CFRP goes on decreasing at yielding of rebar with the increase of CFRP percent (**Fig.-D07**). The stress on CFRP was decreases because with the increase of CFRP the stiffness of the beam is also increases which decrease the strain on CFRP.

b) The stresses on CFRP were constant at ultimate load up to 0.17% of CFRP (**Fig.-D08**). The stress on CFRP was decreases after 0.17% of CFRP because with the increase of CFRP percent the beam becomes over-reinforced and fails in compression.

5.2.4.3 Effect of CFRP percent on the displacement of beam

a) The displacement of the beam is decreases with the increase of CFRP percent (**Fig.-E04**) at fixed load (70 KN). The decrease in displacement is gradual up to 0.5% CFRP but after that the curve going to be flat. The total decrease in displacement is 70.92%.

b) The load carrying capacity enhancement corresponding to the final deflection of beam allowed in design (span/250) is 22.39%. The curve between Load Vs CFRP (%) is linear because the load corresponding to the final deflection allowed in the design is less than the yielding load.

5.2.5 Building Specific Beam(0.75% Rebar)

5.2.5.1 Effect of CFRP percent on the load carrying capacity of beam

a) The load carrying capacity of strengthened beam was increased with the increase of CFRP percent (**Fig.B-5**). The maximum percent of CFRP used was 1.0%. CFRP up to 0.17% was effectively utilized and the use of CFRP more than 0.17% the beam becomes over-reinforced & failure of beam takes place in compression in stead of tension.

b) The enhancement of cracking moment is gradual due to the use of CFRP from 0.17% to 1.0% (**Fig.-C04**). The total increase in cracking moment capacity is 39.76%.

c) The enhancement of yielding moment was gradually up to 0.33% of CFRP but after 0.33% of CFRP the yielding moment capacity is not gradual and going to be flat (**Fig.-C04**). The total increase in yielding moment capacity is 52.66%. Among this 60.65% capacity enhancement at 0.33% CFRP and remaining 39.35% capacity enhancement at 1.0% of CFRP.

d) The enhancement of ultimate moment was gradually up to 0.33% of CFRP but after 0.33% of CFRP the ultimate moment capacity is not gradual and going to be flat (**Fig.-C04**). The total increase in ultimate moment capacity is 81.46%. Among this 60.03% capacity enhancement at 0.33% CFRP and remaining 39.97% capacity enhancement at 1.0% of CFRP.

5.2.5.2 Effect of CFRP percent on the stresses developed in CFRP

a) The stress on CFRP goes on decreasing at yielding of rebar with the increase of CFRP percent (**Fig.-D09**). The stress on CFRP was decreases because with the increase of CFRP the stiffness of the beam is also increases which decrease the strain on CFRP.

b) The stresses on CFRP were constant at ultimate load up to 0.33% of CFRP (**Fig.-D10**). The stress on CFRP was decreases after 0.33% of CFRP because with the increase of CFRP percent the beam becomes over-reinforced and fails in compression.

5.2.5.3 Effect of CFRP percent on the displacement of beam

a) The displacement of the beam is decreases with the increase of CFRP percent (**Fig.-E05**) at fixed load (220 KN). The decrease in displacement is gradual up to 0.5% CFRP but after that the curve goes to be flat. The total decrease in displacement is 60.09%.

b) The load carrying capacity enhancement corresponding to the final deflection of beam allowed in design (span/250; IS 456-2000) is 27.51%. The curve between Load Vs CFRP (%) is linear because the load corresponding to the final deflection allowed in the design is less than the yielding load.

5.2.6 Bridge Specific Beam(0.75% Rebar)

5.2.6.1 Effect of CFRP percent on the load carrying capacity of beam

a) The load carrying capacity of strengthened beam was increased with the increase of CFRP percent (**Fig.B-06**). The maximum percent of CFRP used was 1.0%. CFRP up to 0.17% was effectively utilized and the use of CFRP more than 0.17% the beam becomes over-reinforced & failure of beam takes place in compression in stead of tension.

b) The enhancement of cracking moment is gradual due to the use of CFRP from 0.17% to 1.0% (**Fig.-C06**). The total increase in cracking moment capacity is 35.08%.

c) The enhancement of yielding moment was gradually up to 0.17% of CFRP but after 0.17% of CFRP the yielding moment capacity curve is not gradual and going to be flat (**Fig.-C06**). The total increase in yielding moment capacity is 22.58%. Among this 23.63% capacity enhancement at 0.17% CFRP and remaining 76.37% capacity enhancement at 1.0% of CFRP.

d) The enhancement of ultimate moment was gradually up to 0.17% of CFRP but after 0.17% of CFRP the ultimate moment capacity curve is not gradual and going to be flat (**Fig.-C06**). The total increase in ultimate moment capacity is 28.31%. Among this 26.02% capacity enhancement at 0.17% CFRP and remaining 73.98% capacity enhancement at 1.0% of CFRP.

5.2.6.2 Effect of CFRP percent on the stresses developed in CFRP

a) The stresses on CFRP were goes on decreasing at yielding of rebar with the increase of CFRP percent (**Fig.-D11**). The stress on CFRP was decreases because with the increase of CFRP the stiffness of the beam is also increases which decrease the strain on CFRP.

b) The stresses on CFRP were goes on constantly decreasing (**Fig.-D06**). The stress on CFRP was decreases because with the increase of CFRP percent the beam becomes over-reinforced and fails in compression.

5.2.6.3 Effect of CFRP percent on the displacement of beam

a) The displacement of the beam is decreases with the increase of CFRP percent (**Fig.-E06**) at fixed load (2100 KN). The decrease in displacement is gradual up to 0.5% CFRP but after that the curve going to be flat. The total decrease in displacement is 39.25%.

b) The load carrying capacity enhancement corresponding to the final deflection of beam allowed in design (span/250) is 12.12%. The curve between Load Vs CFRP (%) is linear because the load corresponding to the final deflection allowed in the design is less than the yielding load.

5.3 Effect of rebar percent on the offset of CFRP from the support

The effect of rebar percent is less on the offset of CFRP from the support (**Fig.-F10 to Fig.-F12**). This may happen because the rebar provided in the beam corresponding to the load required to be carried by the beam. For the same section of beam, more percent of rebar is demanded by the beam if the beam required to carry larger load but less percent of rebar is demanded by the beam if the beam is required to carry smaller load. This is verified in the analysis of laboratory scale beam. So, further analysis is not done on the Building specific beam and Bridge specific beam with 0.75% rebar.

CHAPTER 6

Conclusions

Based on the results obtained from the analytical study carried out in this thesis work, the following conclusions can be drawn.

1) For effective utilization of CFRP, maximum percent of CFRP is 0.5% (for laboratory scale beam, building specific beam and bridge specific beam) corresponding to 0.33% rebar and 0.17% (for laboratory scale beam & bridge specific beam), 0.34% (for building specific beam) corresponding to 0.75% rebar.
2) With the use of CFRP up to 1.0%, the total enhancement of cracking moment is 30.86% (for laboratory scale beam), 53.91% (for building specific beam) & 43.56% (for bridge specific beam) corresponding to 0.33% rebar and 39.53% (for laboratory scale beam), 39.76% (for building specific beam) & 35.08% (for bridge specific beam) corresponding to 0.75% rebar of the beam.
3) With the use of CFRP up to 1.0%, the total enhancement of yielding moment capacity is 80.72% (for laboratory scale beam), 95.64% (for building specific beam) & 51.37% (for bridge specific beam) corresponding to 0.33% rebar and 51.94% (for laboratory scale beam), 52.66% (for building specific beam) & 22.58% (for bridge specific beam) corresponding to 0.75% rebar of the beam.
4) With the use of CFRP up to 1.0%, the total enhancement of ultimate moment capacity is 200% (for laboratory scale beam), 208.75% (for building specific beam) & 87.85% (for bridge specific beam) corresponding to 0.33% rebar and 83.75% (for laboratory scale beam), 81.46% (for building specific beam) & 28.31% (for bridge specific beam) corresponding to 0.75% rebar of the beam.
5) The stress on CFRP at yielding of rebar is decreases with the increase of CFRP percent.
6) The stress on CFRP at failure of beam decreases after 0.5% CFRP (for laboratory scale beam, building specific beam & bridge specific beam) corresponding to 0.33% rebar and 0.17% CFRP (for laboratory scale beam & bridge specific beam), 0.33% CFRP (for building specific beam) corresponding to 0.75% rebar because of the beam fails in compression.
7) With the use of CFRP up to 1%, the displacement of beam is decreased up to 72.91% (for laboratory scale beam corresponding to load 40KN & 0.33% rebar),

75.41% (for building specific beam corresponding to load 140KN & 0.33% rebar) & 66.18% (for bridge specific beam corresponding to load 1400KN & 0.33% rebar) and 70.92% (for laboratory scale beam corresponding to load 70KN & 0.75% rebar), 60.09% (for building specific beam corresponding to load 220KN & 0.75% rebar) & 39.25% (for bridge specific beam corresponding to load 2100KN & 0.75% rebar).

8) The increase in load carrying capacity corresponding to the final displacement of beam allowed in design (span/250; IS 456:2000) is 46.32% (for laboratory scale beam), 80.29% (for building specific beam) & 31.96% (for bridge specific beam) corresponding to 0.33% rebar and 22.39% (for laboratory scale beam), 27.51% (for building specific beam) & 12.12% (for bridge specific beam) corresponding to 0.75% rebar.
9) In laboratory scale beam, there is not any change in cracking load and yielding load up to the offset of CFRP from support is 30% but the ultimate load goes on decreasing after the offset of CFRP from the support is 15%.
10) In building specific beam, there is not any change in cracking load and yielding load up to the offset of CFRP from support is 25% but the ultimate load goes on decreasing after the offset of CFRP from the support is 20%.
11) In bridge specific beam, there is not any change in cracking load up to the offset of CFRP from support is 20% but the yielding load and ultimate load goes on decreasing after the offset of CFRP from the support is 25%.

CHAPTER 7

Recommendations

On the basis of the conclusions drawn from the analytical study, the following recommendations can be made.

1. During strengthening of beam, the CFRP should be so provided that the beam should fail in tension for full utilization of CFRP.
2. The maximum percentages of CFRP for effective utilization of CFRP is about 0.5% corresponding to 0.33% rebar and about 0.17% corresponding to 0.75% rebar.
3. Load carrying capacity enhancement of beam depends on the geometry and percentage of rebar provided in the beam. So, analysis should be done carefully before providing the CFRP.
4. The displacement control of CFRP also depends on the geometry and percentage of rebar provided in the beam. So, analysis should be done carefully before providing the CFRP.
5. The offset of CFRP from the support should not be kept more than 25% of the span of the beam.

CHAPTER 8

Recommendations for Future Works

1. The further work can be carried out for the strengthening of beam for shear, strengthening of column.
2. The further work can be carried out for the strengthening of continuous beam for flexure and shear.
3. A similar study can be carried out with modeling of joint between CFRP and concrete.
4. The same study can be carried out with modeling of rebar with link element which makes easy to see the stress and strain variation in the rebar.
5. The effect of grade of concrete and steel on the strengthening of beam can be investigated.
6. The effect of the use of CFRP sheet and strip for flexural strengthening can be investigated.

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