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**BEHAVIOR OF REINFORCED CONCRETE BEAMS
STRENGTHENED WITH CFRP LAMINATES**

A Thesis

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Philosophy in Civil Engineering
(Structures)

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بِسْمِ اللَّهِ الرَّحْمَنِ الرَّحِيمِ

وَيَسْأَلُونَكَ عَنِ الرُّوحِ قُلِ الرُّوحُ مِنْ أَمْرِ
رَبِّي وَمَا أُوتِيتُمْ مِنَ الْعِلْمِ إِلَّا قَلِيلًا

صدق الله العظيم

سورة الإسراء / الآية ٨٥

**TO MY FAMILY
WITH ALL OF MY LOVE AND RESPECT**

**Khalid K. Shadhan
January 2008**

CERTIFICATION

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ABSTRACT

This research is devoted to investigate the behavior and performance of reinforced concrete beams strengthened with externally bonded Carbon Fiber Reinforced Polymer (CFRP) laminates.

The experimental work consists of fabrication and testing of six reinforced concrete beams. Four of these beams have been strengthened with carbon fiber reinforced polymer (CFRP) plates. The experimental variables considered in the test program include, the amount of internal reinforcement, the amount of external CFRP reinforcement, and the loading type, monotonic or repeated. All beams were tested under simply supported conditions.

The experimental results showed a significant increase in ultimate load capacity ranging from 35 to 73 percent which were registered in all strengthened beams, as compared to the control beams. Also, these strengthened beams showed a lower deflection at corresponding loads than the unstrengthened reinforced concrete beam. It was observed that the presence of CFRP plates on the tension face restrained cracking propagation which caused increase in load carrying capacities prior to and beyond the first cracking. The cracking loads are increased by 25 to 67 percent. The structural ductility of the CFRP strengthened RC beams is very low when compared with the corresponding control specimen. The reduction in ductility ratio is about 30 to 67 percent based on deflection.

The design specification of ACI Committee 440-02 recommended using a limitation on the strain of FRP plates using a bond dependent coefficient in order to prevent debonding of FRP plates. This design equation is evaluated based on the comparison made along with the present experimental results.

The analytical work presents the governing differential equations of the problem (strong form) and a finite element formulation (weak form), which is derived from the Principle of Total Potential Energy (PTPE). The finite element consists of a two-node beam element, plus two-node bar element, which allowed slipping with respect to the beam. The nodal degrees of freedom of the beam and the axial member are different to permit axial member slip. Separate displacement fields are assumed in the reinforced concrete beam and the axial member, and bond-slip is automatically derived from compatibility. Cubic polynomials are used for the vertical deflection and linear functions are used for the axial displacements in the concrete beam and the axial member. These assumptions lead to a quadratic bond-slip distribution.

A computer program designated NBSRC (Nonlinear Behavior of Strengthened Reinforced Concrete Beams) is constructed to trace the overall load-displacement relationship, including a brittle debonding failure of CFRP. The program is coded in FORTRAN language and consists of a main program in addition to twenty one subroutines. The validity of the developed model is demonstrated by comparing its predictions with tests results. The results obtained from the finite element analysis show good agreement with the results obtained from the experimental results.

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NOMENCLATURE

The major symbols used in this thesis are listed below; others are indicated with their equations where they first appear.

ABBREVIATIONS

ACI	American Concrete Institute
AFRP	Aramid Fiber Reinforced Polymer
ASTM	American Society for Testing and Materials
CCS	Concrete Cover Separation
CDC	Critical Diagonal Crack
CFRP	Carbon Fiber Reinforced Polymer
COV	Coefficient of variation
EBR	Externally Bonded Reinforcement
FE	Finite Element
FEM	Finite Element Method
FRP	Fiber Reinforced Polymer
GFRP	Glass Fiber Reinforced Polymer
IFC	Intermediate Flexural Crack
ISC	Intermediate Shear Crack
PEI	Plate End Interfacial
RC	Reinforced Concrete
TPE	Total Potential Energy

SYMBOLS

B_B	Beam deformation-displacement matrices
B_b	Bond deformation-displacement matrices
D_b	Bond force at the interface
d_B	Beam deformation array

d_b	Bond deformation array
K	Element stiffness matrix
K_B	Beam contributions to element stiffness matrix
K_b	Bond contributions to element stiffness matrix
k_B	Section stiffness matrix
k_b	Bond stiffness matrix
L_B	Beam differential operator
L_b	Bond differential operator
M_B	Beam section bending moment
N_u	Displacement shape functions
p	Element external nodal force array
p	Element force array
p_y	Transverse distributed load
Q	Element internal nodal force array
Q_B	Beam contribution to element internal nodal force array
Q_b	Bond contribution to element internal nodal force array
U	Element displacement array
u	Element displacement array
u_B	Beam axial displacement
v_B	Beam transverse displacement
$V_B(x)$	Beam section shear force
y_{FRP}	Distance of FRP plate from section reference axis
ε_B	Beam axial strain
κ_B	Beam curvature
Π_{TPE}	Total potential energy functional

CHAPTER ONE

INTRODUCTION

1.1 Background

It is well known that concrete is a building material with high compressive strength and poor tensile strength. A concrete beam without any form of reinforcement will crack and fail when subjected to a relatively small load. The failure occurs suddenly in most cases, and in a brittle manner. The most common way to reinforce a concrete structure is to use steel reinforcing bars that are placed in the structure before the concrete is cast. Since a concrete structure usually has a very long life, it is not unusual for the demands on the structure to change with time. The structure may have to carry larger loads at a later date, or fulfil new standards. In extreme cases a structure will have to be repaired due to an accident. A further reason can be that errors have been made during the design or construction phase resulting in need for strengthening the structure before usage .If any of these situations should arise, it needs to be determined whether it is more economic to strengthen the existing structure or to replace it. In comparison to building a new structure, strengthening an existing one is often more complicated, since the conditions are already set (Nordin, 2003).

Strengthening or repairing deteriorating infrastructure, such as bridges and buildings, has become a major challenge to construction activity all over the world (Mukhopadhyaya and Swamy,2001). There are many different ways to repair or upgrading a concrete structure. There is often a possibility to use a cast on technique to change the physical appearance of the structure and in that way giving it somewhat different properties in strength and stiffness. However, this also means that the structure needs more space which is not always possible (Nordin, 2003).

In the last decades, the development of strong epoxy adhesives has led to the plate bonding strengthening technique. This upgrading technique may be defined as one in which plates of relatively small thickness is bonded with an epoxy adhesive to, in most cases, a concrete structure to improve its structural behavior and strength. One advantage with this technique is that there are no large physical changes of the structure, another is that very high strengthening effects can be achieved.

1.2 Fiber Reinforced Polymer FRP

The term composite often refers to a material composed of two or more distinct parts working together. Often one of the parts is harder and stronger, while the other is more of a force transferring material. FRP is an abbreviation of fiber reinforced polymers and is a composite of fibers and adhesive. The materials FRP holds many advantages over other materials in civil engineering. It has very high stiffness-to-weight ratio and high strength-to-weight ratio. The material exhibit excellent fatigue properties, non-magnetic properties, corrosion resistance, and is generally resistant to chemicals. The thermal are controllable and both material and geometrical can tailored for the application. Fiber Reinforced Polymer, FRP, is a composite material consisting of fibers and a polymer matrix.

1.2.1 Fiber Materials

Several materials are available for the fibers, e.g. glass, aramid, carbon. Almost 95 percent of all applications for strengthening purposes in civil engineering are by carbon fibers (Nordin, 2003). Figure 1.1 demonstrate some typical response of uniaxially loaded fiber materials and steel. HM and HS are abbreviations of high modulus of elasticity and high strength, respectively. Fibers have a linear elastic behavior until

failure which is brittle. Table 1.1 compares the material properties of carbon fiber, concrete and steel.

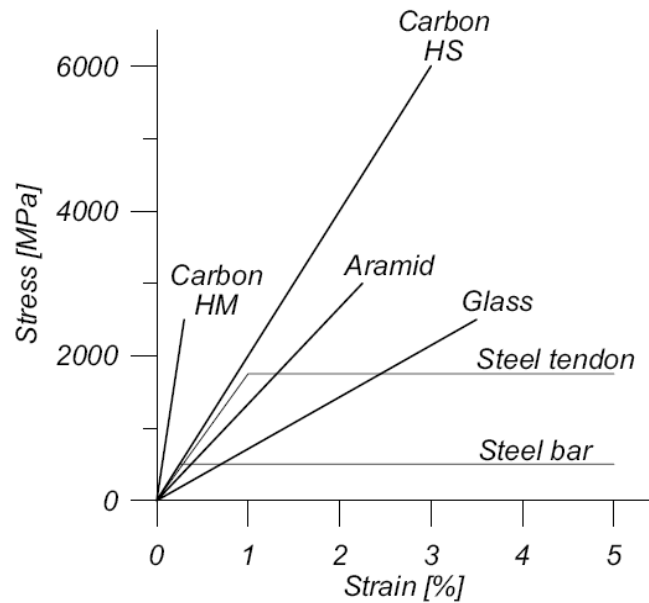


Figure 1.1: Stress-strain relationship of fibers and steel (Carolin, 2003)

Table 1.1: Properties of concrete, steel and CFRP (Carolin, 2003)

Material	Modulus of elasticity (GPa)	Compressive strength (MPa)	Tensile strength (MPa)	Density (kg/m ³)
Concrete	20-40	5-60	1-3	2400
Steel	200-210	240-690	240-690	7800
CFRP	200-800	N/A	2100-6000	1750-1950

The fibers are what make the FRP strong and there are three things that control the mechanical properties of the FRP (Nordin, 2003):

- *Constituent materials*

As mentioned earlier there is a wide array of different materials to use. What is important to remember is that the choice of fiber materials determines, together with choice of polymer, what kind of quality, properties and behavior the FRP finally will obtain.

- *Fiber amount*

Regarding the amount of fiber used in the FRP it is easy to say that the more fiber used the better properties will be achieved. This is somewhat true but with too high fiber content there will be manufacturing problems. If the fibers are tightly packed the matrix will have problems enclosing the fibers which might deteriorate the FRP.

- *Fiber orientation*

The FRP will be stiffer and stronger in the fiber direction. For example, a rod with all the fibers as very strong in its fiber direction but in perpendicular direction the FRP has not as good properties. A typical FRP product for the construction industry has therefore a anisotropic behavior compared to steel which is isotropic.

1.2.2 Matrix

The matrix, i.e. the polymer in the composite, is used to bind the fibers together, transfer the forces between the fibers and to protect the fibers from external mechanical and environmental damage. The shear forces created between the fibers are limited to the properties of the matrix. The matrix is also the limited factor when applying forces perpendicular to the fibers.

It is important that the matrix has the capability to take a higher strains than the fibers, if not there will be cracks in the matrix before the

fibers fail and fibers will be unprotected. Common matrix materials in civil engineering application are polyester, vinyl ester, and epoxy, through epoxy is generally favored (Lundqvist, 2007). Properties are of matrix materials shown in Table 1.2.

Table 1.2: Material properties of matrix materials (Lundqvist, 2007)

Material	Modulus of elasticity (GPa)	Tensile strength (MPa)	Ultimate tensile strain (%)	Density (kg/m³)
Polyester	2.1-4.1	20-100	1.0-6.5	1000-1450
Vinyl ester	3.2	80-90	4.0-5.0	----
Epoxy	2.5-4.1	55-130	1.5-9.0	1100-1300

1.3 FRP Strengthening in RC Members

Fiber Reinforced Polymer (FRP) materials are composite materials consisting of high strength fibers in the shape of polymer matrix. Fibers are selected based on the strength, stiffness, and durability required for specific applications; likewise the resins are selected based on environmental to which the FRP will be exposed to and the method by which the FRP is manufactured.

The use of FRP bonded to deteriorated, deficient, and damaged reinforced concrete structures has gained popularity in Europe, Japan, and North America. High-strength FRPs offer great potential for lightweight, cost-effective retrofitting of concrete infrastructure through external bonding to concrete members to increase their stiffness and load carrying capacity (Buyukozturk and Yu, 2006). However, these materials are still almost unknown to engineers in the civil engineering industry (Nordin, 2003), although the knowledge seems to be increasing.

Strengthening with FRP composite can be applied to various types of structural members including beams, columns, slabs and walls. Depending on the member type, the objective of strengthening may be one or a combination of several of the following (Buyukozturk et al., 2004) : (1) to increase axial, flexural or shear load capacities; (2) to increase stiffness for reduced deflections under service and design loads; (3) to increase the remaining fatigue life; (4) and to increase durability against environmental effects.

1.4 Debonding Problems in FRP Strengthened RC Beams

The major concern in adhesively bonding plates to the external surface of reinforced concrete beams is the premature debonding of the plates, which are unique forms of failure of plated beams. Previous researchers (i.e. Teng et al. 2002; Oehlers and Seracino 2004; Khomwan et al. 2005) have identified different forms of debonding mechanisms and these can be classified into four main categories, see Figure 1.2: (1) Plate End Interfacial (PEI) debonding; (2) Concrete Cover Separation (CCS); (3) Intermediate Flexure Crack-induced (IFC) and/or Intermediate Shear Crack-induced (ISC) debonding; and (4) Critical Diagonal Crack (CDC) debonding.

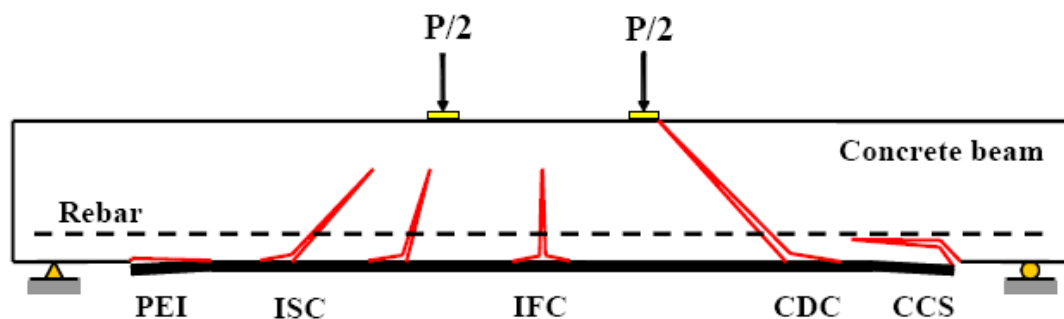


Figure 1.2: Debonding failure modes of FRP strengthened RC beam

It is worth noting that debonding almost always occurs at the concrete layer adjacent to the adhesive, due to the strength of the adhesive being much greater than the tensile strength of the adjacent concrete (Khomwan et al. 2005; Lundqvist, 2007).

Plate end debonding is caused by the existence of high interfacial shear and normal stresses at the plate end due to abrupt termination of the plate. When the beam is bent, the plate will tend to stay straight, therefore causing cracks to form at the plate ends and propagate inwards. These debonding cracks may occur at the level of the tension reinforcement (also known as Concrete Cover Separation CCS), or at the concrete/adhesive interface (also known as Plate End Interfacial PEI debonding), and they usually propagate gradually at first, then rapidly at plate end debonding failure (Teng et al. 2002; Oehlers and Seracino 2004; Khomwan et al. 2005).

Intermediate Crack-induced debonding is associated with the formation of flexural or flexural-shear cracks in the vicinity of the plate. Theoretically, the plate requires infinite strains across the intercepting cracks. As this is impossible, horizontal or intermediate crack interface cracks occur at the interface in order to accommodate the intercepting cracks. IC interface cracks usually first occur at position of maximum moment, where strains in the plate are largest (Oehlers and Seracino 2004), and propagate gradually from the roots of the intersecting cracks towards the plate ends. These IC interface cracks usually have little impact on the overall strength of the beam until they propagate further along the beam causing plate strains to reduce, that is the plate is debonding and this is referred to as IC debonding, and eventually cause a decrease in strength of the beam as IC debonding failure takes place. Because concrete has a much lower tensile strength than the adhesive, these debonding cracks generally occur in the concrete adjacent to the

adhesive to concrete interface. For IC debonding induced by flexural crack, crack widening causes debonding propagation. For intermediate crack-induced debonding induced by flexural-shear crack, the relative vertical displacement between the two faces of the crack also produces peeling stresses at the interface; however, this effect is less significant hence it is considered that the propagation of debonding is predominately caused by widening of crack (Teng et al. 2002).

Critical Diagonal Crack (CDC) debonding, also known as vertical shear debonding is associated with vertical shear deformations of the beam. When the critical diagonal crack forms, sliding and rotation action occurs across the crack causing the plate to separate from the beam by a debonding crack propagating from the root of the diagonal crack to the plate end (Mohamed Ali, 2000). As the critical diagonal crack intercepts the plate, the debonding crack propagation at the plate/concrete interface is dependent on the IC debonding strength. CDC debonding is a sudden failure with no visual warning and the presence of stirrups has little or no effect on CDC debonding (Lundqvist, 2007). Therefore, beams are prone to CDC debonding failure when the shear load exceeds the concrete shear capacity of the beam.

Debonding problems in FRP bonded RC systems are a priority mechanics and design issue due to their premature and brittle nature. If not well understood and properly considered in the design, debonding failures may not only render the strengthening ineffective, but may also detrimental to the structural performance and safety due to reduced ductility (Buyukozturk and Yu, 2006).

The design specification of ACI Committee 440-02(2002) recommended using a limitation on the strain of FRP plates using a bond dependent coefficient in order to prevent debonding of FRP plates. This limitation was derived based on visual observation and experience , but

the theoretical justification is unclear (Achintha and Burgoyne, 2006). Continued research is needed in this area to understand and model debonding failures in FRP strengthened members under monotonic as well as cyclic loading conditions, and to develop related design and application guidelines.

1.5 Research Objectives

The use of externally FRP to strengthen and rehabilitate existing concrete structures is a relatively new technique. Until recently, there is a lack of design code and guidelines for their use (Al-Mahaidi, 2003). The objectives of the present work are to:

- investigate the structural responses of CFRP strengthened RC beams under monotonic loading.
- examine the effect of the material and bond degradations due to the cyclic loading on the behavior of CFRP strengthened RC beams.
- investigate the importance of the debonding failure modes on the overall flexural behavior of strengthened reinforced concrete beams.
- evaluate the ACI Committee 440-02 current design equation related to debonding failure.
- develop finite element model to be applicable for nonlinear flexural analysis of FRP strengthened RC beams taking into consideration the effect of FRP debonding.

1.6 Outline of Contents

This thesis is arranged into eight chapters. Chapter One introduces the problem within hand and its importance and applications in structural engineering.

Chapter Two reviews literature relevant to the topics of this research including analysis of composite sections, partial bond composite members, and the finite element formulation.

The next chapter, Chapter Three, deals with the description of the experimental program carried out at the Structural Laboratory of the Civil Engineering Department of Babylon University. The beams specimen details, materials properties, installation procedures and test setup for the experiment are presented.

In Chapter Four, after the definitions of the generalized element forces and corresponding deformations, the basic governing differential equations (strong form) for strengthened RC beam with bond interfaces are derived. Further, Displacement-based finite element formulation (weak form), which is derived from the principle of Total Potential Energy (TPE), is presented later.

The numerical algorithm used for the solution of the nonlinear strengthened reinforced concrete problem at hand together with the developed computer program and its flowchart are presented in Chapter Five.

Chapter Six presents a discussion regarding the use of CFRP strengthening and the results obtained from the tests presented in Chapter Three.

Analysis results obtained from finite element solution are discussed and compared with the experimental results in Chapter Seven.

Finally, Chapter Eight gives the conclusions of this work, together with suggestions for further work.

CHAPTER TWO

REVIEW OF LITERATURE

2.1 General

Fiber Reinforced Polymer (FRP) composites as a strengthening material in the construction industry has been in use for almost twenty years now, with the first applications in Japan and Switzerland. The first commercial use of the material in itself occurred in the mid fifty's. The number of strengthening projects utilizing FRP around the world is increasing as the method becomes more accepted and design guidelines are developed.

During the last two decades of the previous century, the use of FRP composites in reinforced concrete members has emerged as one of the most promising technologies to address the rehabilitation of infrastructures. There is a wide range of applications for FRP reinforcement that covers new construction as well as the rehabilitation of existing structures.

The aim of this chapter is to present a review for the available information concerning the behavior of reinforced concrete members with externally FRP laminates, development of the related theories and the use of finite element nonlinear analysis for the assessment of strengthening with FRP laminate.

2.2 FRP in Infrastructures Rehabilitation

Rehabilitation is a generic term for three concepts; repair, strengthening, and retrofit. Repair is defined as restoring a degraded or damaged structure member to its original capacity. Strengthening is used to increase the capacity of a structural member beyond its designed performance level. The term retrofit is reserved for the seismic upgrade of

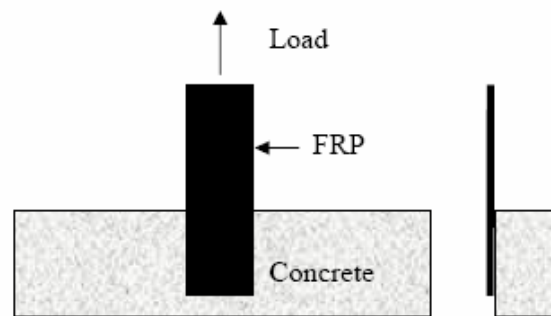
a structure. FRP material can be utilized for all three applications. The benefits of using FRP in strengthening operations includes simplicity of handling ,cutting and installation , small change in strengthened structural member's dimensions . Strengthening with FRP is a rehabilitation method not just reserved for concrete structures. Successful applications have been done for steel (Carolin, 2003), timber (Johansson et al., 2007), and masonry (Triantafillou, 1998). Another field of application for FRP is prestressing of bars and plates, e.g. (Nordin , 2003).

2.3 Bond of FRP Plate-Concrete Interface

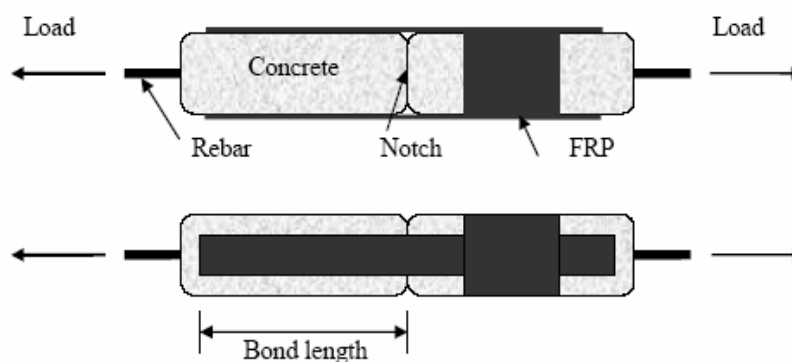
With the development of the technology of upgrading the existing concrete structures by using externally bonded FRP composites, a number of issues related to conventional structural behavior of the concrete structures after being upgraded have been studied in the past decades. Among them is a keen interest to clarify the mechanisms of the interface bond between FRP composites and concrete substrates, because the bonding interface is relatively weak in comparison with the neighboring materials in the whole upgraded system. In most of strengthening cases, the interface bond is critical in transferring stresses from the existing concrete structures to externally bonded FRP composites. When a structural element is encircled in FRP composites the mechanical role of interface bond becomes less important ,but it still has the function of keeping the integrity and durability of the composite FRP-concrete system (Dai ,2003). Therefore, a good understanding on the interface bond is a prerequisite for achieving more reliable but rational design for concrete structures externally bonded with FRP composites.

The most important role of the bond interface between FRP plates and concrete is to transfer shear stresses from existing concrete structure to externally bonded FRP plates for both shear strengthening and flexural

strengthening cases. Therefore , the shear bond properties of FRP plate-concrete interfaces have been extensively studied. Various test methods including single-lap-type and double-lap-type (see Figure 2.1) have been developed to characterized the local interfacial shear bond behaviors by studying the strain distribution in the FRP sheets or to evaluate the average interfacial bond strength. The evaluated interface characteristics parameters include average shear bond strength ,effective bond length, maximum shear bond stress ,interfacial fracture energy ,as well as the local bond stress-slip relationship.



(a) Single-lap shear bond test



(b) Double-lap shear bond test

Figure 2.1: Shear bond test methods (Dai ,2003)

- *Bond length of FRP plates*

Many researchers have studied the effects of sheet bond length. While bond strength increase as the sheet bond length is increase, when the sheet bond length is increased beyond a certain extent, the bond length does not increases any further. Therefore, the average bond length decreases with the increase of sheet bond length. In other words, there exists an effective bond length (Bizindavyi and Neale, 1999).

Besides the premature debonding phenomenon caused by shear stress concentration, the existence of effective bond length is another factor that causes the tensile strength of FRP materials not to be fully utilized. Nevertheless, owing the different definitions given by different researchers and the dissimilar materials used in their tests, the effective bond lengths can vary significantly, such as 75mm (Bizindavyi and Neale, 1999), 100mm (Ueda et al., 1999), 93mm (Lorenzis et al., 2001), 64-135mm (Nakaba et al., 2001).

- *Bond width of FRP sheets*

It has been shown that the sheet width does not influence the average bond strength of interfaces when the width of sheets ranges from 50mm to 200mm. However, when the sheet bond width is less than 50mm, the smaller the sheet width, the higher average shear bond strength that the interface can achieve (Brosens and Gemert, 1999).

- *Stiffness of FRP sheets*

There are many reports on the effects of the stiffness of FRP sheets. Clearly, the bond strength increases with the stiffness of FRP sheets. Meanwhile, a higher FRP stiffness will results in longer anchorage length (Dai, 2003).

- *Strength of concrete*

Concrete strength is thought to be a factor affecting the interface bond significantly. But it is, in fact, rather difficult to verify its effects in experimentally. Only when the concrete strength is greatly can its effects become visible. Lorenzis et al. (2001) indicated that the dependence of the interfacial fracture energy on the concrete compressive strength f'_c is negligible. However, Chajes et al. (1996) and Horiguchi (1997) reported that the bond strength is proportional to $(f'_c)^{1/2}$ and $(f'_c)^{2/3}$, respectively. Brosens and Gemert (1999) reported that the bond strength is proportional to the square root of concrete tensile strength. Nakaba et al. (2001) and Sato et al. (2000) reported that the maximum interfacial bond stresses are proportional to $(f'_c)^{0.19}$ and $(f'_c)^{0.20}$, respectively.

- *Surface treatment of concrete*

Tests results by Chajes et al. (1996) showed that surface preparation of the concrete influences the bond strength. They found that the use of mechanical abrading (similar to sandblasting) a higher average interfacial bond strength could be achieved. Mitsui et al. (2000) conducted a further quantitative study on the effects of surface conditions. They evaluated the surface roughness index of concrete by changing the treatment methods, which were sandpaper polishing, desk sander grinding, sandblasting and chipping. They observed the three dimensional profile of the concrete surface, using an optical displacement meter and obtained measurements of the maximum depth, superficial area and so on. They concluded that the methods of sandblasting and chipping could lead to higher bond strength. Regarding the effects of concrete surface cleanness, it has been confirmed that the bond strength will decrease markedly if the concrete surface is greasy. But whether or not the moisture of concrete

surface affects the bond strength or not depends on the type of bonding adhesives used. It should be noted that in practice it is difficult to control the roughness of concrete surface quantitatively. Experimental database indicates that there is a wide scatter of bond strength cost by concrete surface condition, even when different workers (laboratories) follow a same surface treatment method (Dai ,2003). But as a minimum, a clean concrete substrate with open pore structures is needed to achieve a good bond between the adhesive and concrete ,or, in other words, to insure that the bond failure occurs in the concrete.

- *Properties of bond layers*

It has been found that bond layers with lower elastic moduli, but good ductility, can lead to higher interface bond strengths. The mechanical properties of bond layers can be adjusted through changing the elastic modulus of either bonding resin or putty. However the effective bond length increases when the elastic modulus of bond layers decreases (Dai et al. , 2002).

- *Interface defects*

The lifting of bonded sheets can simulated the interface defects that can be induced during bonding construction. It has been reported that a 12×12mm lifting area of FRP sheets in specimens prepared for pull-out shear bond tests, which was equivalent to 6 to 13 percent of the whole sheet bonding area, has no significant effects on the overall pull-out bond strength (Dai ,2003).

2.4 External Strengthening with Steel Plates

The concept of bonding FRP to strengthen existing structures originates from steel plate bonding technique. Steel plate bonding is

widely used in many strengthening applications including bridge retrofitting because such bonding is relatively simple, effective and efficient with minimum site disruption compared with section enlargement, external post-tensioning, deck widening or increasing deck overlay. Changes in dimensions of the retrofitted structures are only of the order of a few millimeters.

The ultimate behavior of steel plated RC beams appears to be closely related to the geometry of the plated cross-section. For thin plates, failure usually occurs in flexure. However, if the plate aspect ratio falls below a certain value, separation of the plate from the beam can occur, initiating from the plate end and resulting in the concrete cover being ripped off. These observations are consistent with the fact that simple elastic longitudinal shear stresses are inversely proportional to the plate width. Consequently, as the steel plate width decreases, the longitudinal shear stresses increase. In addition, the bending stiffness of the plate increases; thereby increasing the peeling stresses normal to the beam (Hollaway and Leeming, 2001).

In corrosive environments, however, deterioration of the bond between the plate and concrete substrate caused by the corrosion of steel renders the force transfer mechanism between the two components less effective. The long-term performance of the system then becomes questionable particularly in outdoor applications. Measures such as priming the steel surface impede the corrosion process but do not completely stop it.

Macdonald and Calder (1982) reported that "slight signs" of corrosion were feasible on unprimed as well as primed bonded steel plates after two years of natural exposure. Other shortcomings include high self-weight resulting in the need to use lapping devices and lifting equipments during operation and a restriction on the maximum length of

the steel plate. For the latter, fabrication and handling issue arise when grater plate lengths are required which necessitates the use of joints, as otherwise the high wilding temperatures would destroy the adhesive bond. These drawbacks have led to the search for an alternative material and the result is fiber reinforced polymer.

2.5 Experimental Research on FRP Strengthened Beams

To overcome some of the shortcomings that are associated with steel plate bonding, it was proposed in the mid-1980s that fiber reinforced polymer (FRP) plates could prove advantageous over steel plates in strengthening applications (Meier and Kaiser, 1991). Unlike steel, FRPs are unaffected by electrochemical deterioration and can resist the corrosive effects of acids, alkalis, salts and similar aggressive materials under a wide range of temperatures (Al-Mahaidi, 2003). The high material cost of FRP might be a deterrent to its use, but upon a closer look, FRP can be quite competitive.

The FRP plates are much thinner than its steel counterpart, which allow lap joints to be made between different elements. The reduced eccentricity of the plate also reduces the tendency for peeling failure. Many published test data showed that flexural strengthening with FRP plates behaves very similarly to steel plates. Besides the classical failure modes, such as steel fracture ,concrete crushing or shear failure, bond failures could occur in the interface between the externally bonded FRP plates and the concrete substrate (Tan ,2002).

In recent years, many researchers have focused on the flexural performance of the RC strengthened with FRP composites through experimental investigations and it can be classified into two categories:

- Performance of strengthened RC beams under monotonic load.
- Performance of strengthened RC beams under cyclic load.

A brief review of some of the published literature under the two categories above is presented in the following sections.

2.5.1 Monotonic Performance of Strengthened RC Beams

Many experimental studies on reinforced concrete beams bonded with FRP soffit plates have been undertaken to establish the performance of flexurally strengthened beams. These studies have focused on the effectiveness of the technique and in providing an understanding of the overall behavior and debonding failure mechanisms.

Saadamanesh and Ehsani (1991) tested five rectangular concrete beams and one T-beam, strengthened with GFRP, under four-point loading. The results of the rectangular beam tests indicated that the ultimate flexural strength of reinforced concrete beams can be significantly increased by gluing GFRP plates to the tensile face. However, beams having no conventional steel reinforcement failed at a very low load due to debonding of the FRP plate. Thus, it was concluded that a minimum amount of steel reinforcement is necessary to limit the width of the flexural cracks to prevent debonding of GFRP plates. Results of the T-beam test indicated that bonding of the GFRP plate doubled the flexural capacity of the beam. It was also cited that beams strengthened with FRP plates reduced crack widths and delayed the formation of the flexural cracks. However, bonding of the FRP plates reduced the ductility of the beam compared to the exhibited by the control specimens.

Ritchie et al. (1991) tested a series of sixteen under-reinforced concrete beams to study the effects of external strengthening using three different types of FRP; glass, carbon, and aramid fibers were bonded to

the tension side of the beams using a two-part epoxy. The results of the tests showed an increase in stiffness (over the working load range) from 17 to 99 percent and an increase in strength from 40 to 97 percent based on the type, amount, and orientation of FRP that was applied to the beam. Since the beams in the initial tests failed at the ends of the FRP plate instead of at the constant moment region, a series of anchorage types were developed in hopes to shift the failure to the constant moment region (see Figure 2.2).

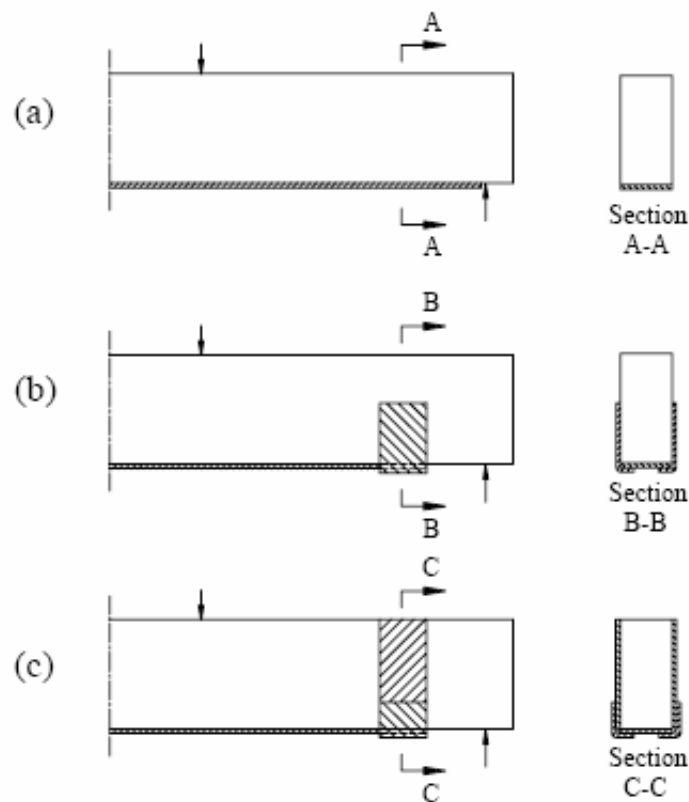


Figure 2.2: End anchorages: (a) plate extended up to support, (b) partial-height anchorage, (c) full-height anchorage (Ritchie et al., 1991)

The first anchorage type consisted of anchoring the ends of the plates with fiberglass angles (Figure 2.2b). This led to a larger load capacity, but the failure was still located at the ends of the plate. The

second method consisted of wrapping FRP plates around the beam at the ends (Figure 2.2c). This type also raised the load capacity, but was unsuccessful in transferring the failure to the constant moment region. The final method consisted of extending the plates up to the supports (Figure 2.2a). This method was successful in both increasing the load capacity and shifting the failure to the constant moment region, but brittle and debonding failure modes still occurred.

Chajes et al. (1994) performed a series of laboratory tests on reinforced concrete beams with bonded composite fabric to evaluate the improvement to the flexural capacity. The fabrics used were made from aramid, E-glass and graphite fibers. Originally, all beams were strengthened for flexure without any shear strengthening. End taps (Figure 2.2b) were later employed to prevent fabric debonding which occurred in the first series of tests. Beams strengthened with aramid failed due to rupture of the composite. These different modes of failure were attributed to the variation in the fabric ultimate strain. The ultimate strain for the aramid fabric was twice that of E-glass and three times that of the graphite. Increases in the flexural capacity ranged from 36 to 57 percent with corresponding increases in flexural stiffness of the 45 to 53 percent. This increase in strength was accompanied by a decrease in ductility. The reported ductility index for beams strengthened with fabric exhibited a ductility index in range of two or three, while beams without the composite fabric exhibited a ductility index in the range of four to five.

Sharif et al. (1994) also studied the repair of pre-loaded reinforced concrete beams with epoxy bonded fiberglass reinforced plastic plates. Ten beams were over designed in shear to avoid a brittle shear failure due to the anticipated increased shear load in the damaged beams after the repair. The RC beams were initially loaded to 85 percent of the ultimate flexural capacity and subsequently repaired with FRP plates, bonded to

the soffit of the beam. The plate thickness was varied to assess failure initiated in the curtailment zone due to the high concentration of shear and peeling stresses. Different repaired anchoring schemes were conducted in an effort to eliminate such failures and ensure ductile behavior. The results generally indicated that the flexural strength of the repaired beams was increased over that of the original, unrepaired specimen. The ductility of the repaired beams was inversely proportional to the plate thickness. The use of an end anchorage provided a proper anchorage system and improved the ductility of beams repaired with plates of large thickness.

Arduini and Nanni (1997) conducted experiments in which CFRP sheets were bonded to the soffit of precracked RC beams. They concluded that the CFRP significantly enhanced the ultimate load capacity of the beams, while the flexural stiffness was increased to a lesser extent. However, the ductile response of the control specimens was often changed to a brittle failure mode in the members bonded with FRP. They recommended further studies in the area of concrete-FRP adhesion in order to limit the extent of FRP peeling so that the strengthening technique can be more effective.

Garden and Hollaway (1998) tested a total of twelve reinforced concrete beams in four points bending. The beams were 100mm square in section and 1000mm long. The tests were to determine the influence of plate end anchorage (Figure 2.3) on the structural behavior and failure modes of the strengthened beams. All beams were under-reinforced in flexure with the steel ratio less than 1 percent. The shear span to depth ratio (a/d) were varied at 3.0, 3.4, and 4.0. The results show that the significant increase in ultimate load was found for the strengthened beams with plate end anchorage. The increase in ultimate capacity of strengthened beams without plate end anchorage was 88 to 177 percent

greater than the control specimens. For the strengthened beam with plate end anchorage, the increase in ultimate capacity was 192 to 209 percent greater than the control specimens. The control specimens failed in the conventional mode of steel yielding followed by concrete crushing. For the strengthened beams, Garden and Hollaway defined the failure modes as cover separation and partial cover separation. Cover separation involved the removal of full thickness of concrete cover starting from one of shear span (see Figure 1.2). Partial cover separation is the intermediate shear crack-induced debonding failure. The strengthened beams without end anchorage $a/d = 3.0$ failed by concrete cover separation. For the strengthened beams with end plate anchorage ($a/d = 3.4$ and 4.0), failure was by intermediate shear crack-induced debonding.

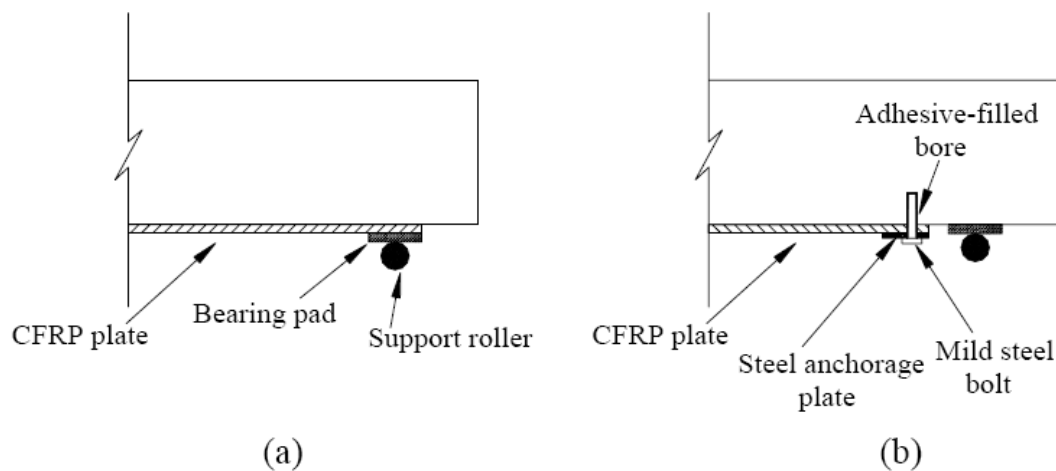


Figure 2.3: Plate end anchorage systems: (a) plate end under supports ,
(b) bolted plate ends (Garden and Hollaway 1998)

Rahimi and Hutchinson (2001) tested concrete beams with CFRP or GFRP laminates bonded to the soffit. The specimens were 200mm wide, 150mm deep and 2300mm long and were tested in four-point

bending. The test variables included the amount of conventional (internal) reinforcement and the type and amount of external reinforcement. It was found that the CFRP plates enhanced both the stiffness and the strength of the beams, with a maximum increase in ultimate load-carrying capacity of 230 percent. The authors concluded that the amount of tensile steel and internal shear reinforcement affected the magnitude of the performance enhancement. It was noted that the tensile force in the FRP increase at a greater rate after the flexural steel yields. For highly reinforced beams, the compressive strain of concrete is close to the ultimate value when the flexural steel yields. Thus, the enhancement of the beam strengthened by FRP reinforcement is limited by the quantity and elastic modulus of the CFRP laminate. The characteristics of the cover play a significant role in determining the ultimate load of plated beams. In the beams with thinner laminates, failure was in the concrete cover close to the loading point within the shear span. A layer of adhesive and cement paste was found on the FRP surface where plate detachment occurred. For beams with thicker plates, the failure location shifted towards the plate ends where the shear and normal peeling stresses increased. At peak load levels, the interface shear stress averaged along the beam was higher for beams with thicker FRP plates. However, peeling at the ends was not a primary failure mechanism for the FRP-bonded beams tested by Rahimi and Hutchinson, as is usually the case for steel-plated beams. For all of the FRP-strengthened beams tested, failure was by concrete cover separation and plate delimitation.

Lamanna (2002) proposed a relatively new method in applying FRP composites. He uses multiple mechanical fasteners unlike current practice, where a FRP strip is attached to the beam with an adhesive. Custom FRP strip were designed specifically for the use with mechanical fasteners. Small-scale beam tests were conducted to examine the

feasibility of the overall method and to further identify factors affecting the structural behavior of the strengthened beam. Lightly reinforced small-scale reinforced concrete beams measuring 1219mm long and having a cross section of 152×152 mm were used. Parameters such as fastener spacing, predrilling pilot holes and edge distance were examined in these tests. Test results showed increases of yield and ultimate moment capacities of 34 and 25 percent over the unstrengthened control beam. The concrete failure mode begins with a concrete pryout failure around the fastener, shown schematically in Figure 2.4. Once pryout failure began, the concrete behind the fastener was damaged, causing the fastener to rotate. As the fastener rotates, the FRP strip began to pull the fastener out of the concrete, which was damaged by the initial pryout failure.

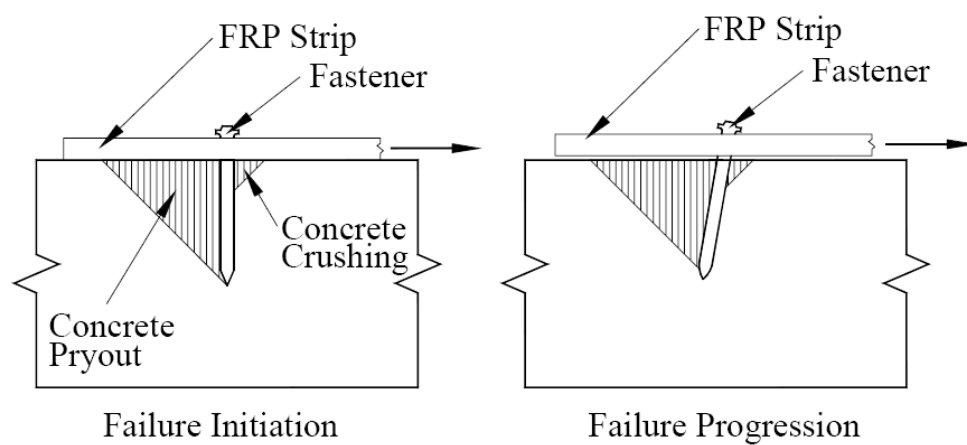


Figure 2.4: Schematic of the pryout failure (Lamanna, 2002)

In 2003, Brena et al. presented the results of four tests of full scale reinforced concrete specimens representative of common short span bridges in Texas. The specimens were strengthened to increase their flexural capacity using carbon fiber reinforced polymer laminates. The

laboratory specimens were strengthened using two different composite systems that were commercially available at the time of this investigation. The experimental results indicate that, similar to small scale specimens with rectangular cross sections, a considerable increase in flexural capacity can be achieved if proper measures are taken to prevent debonding of the composite materials from the surface of the concrete. Also, the results indicated that the strengthened specimens failed after the composites reached 50 to 65 percent of the rupture strength of the laminates.

A comprehensive experimental study was conducted by Zhang et al (2006). Eighteen RC beams, including sixteen beams strengthened with CFRP laminate at different levels of preload and two control beams, were tested to investigate the influence of preload level on flexural behavior of CFRP-strengthened RC beams. All beams were 2500mm long, and had 150mm wide, 250mm high cross section. The parameters considered are the preload level at the time of strengthening (0, 0.3, 0.6 and 0.8 of nominal flexural strength of the control beams), plies of CFRP laminates (1 and 2 plies), and rebar ratios (0.84 percent and 1.52 percent). The main failure mode was the intermediate flexure-shear crack induced interfacial debonding (simply as ICI debonding) with the entire depth of concrete cover separation in the constant moment region of the beam. Zhang et al concluded that although the level of preloaded at the time of strengthening has more influence on deflections of beam at yielding and at ultimate stages than on ultimate strength of the beam, their differences are marginal.

Mohammed (2007) studied the flexural behavior of reinforced concrete beams strengthened with externally bonded CFRP laminates with or without external anchorages. Nine beams were tested to failure. Eight of the beams were strengthened with CFRP laminates. Test

variables included the number of CFRP strip layers (one layer or two layers), the shape and configuration of external anchorage, and the effect of using CFRP strip layers throughout the total length of the beam with and without using external anchorages. Failure of all strengthened beams was initiated with CFRP debonding. The research leads to the conclusions that the external anchorages are very effective in increasing the interaction between the CFRP and the concrete section, and improving the structural behavior of the strengthened beams (increasing cracking load, reducing beam deflection, and increasing the ultimate load of the beam).

2.5.2 Cyclic Performance of Strengthened RC Beams

Seismic retrofitting of existing structures comprises a major portion of structural applications. Thus, performance of strengthened members under cyclic loading must be thoroughly investigated with emphases on brittle debonding failures to ensure the seismic safety of strengthened systems (Buyukozturk et al., 2004).

Barnes and Mays (1999) tested five reinforced concrete beams in fatigue. The beams were 2300mm long, 130mm wide and 230mm deep with bolted end anchorages. A 90mm wide, 1.2mm thick and 2020mm long CFRP plate was bonded to the soffit of the beams. The plate was anchored with steel plates 90mm wide, 40mm long and 6mm thick at both ends using two 90mm long and 10mm diameter mild steel bolts. The tested beam were of a similar size as those of Garden et al. (1998). Three loading options were used .First, identical loads were applied to both control and strengthened beams. Second, the control and strengthened beams were loaded to give the same stress range in steel reinforcement. Third, the beams were loaded to give the same percentage of the failure load. Each beam was subjected to a cyclic load in four-point bending at a

frequency of 1 Hz which is reasonably representative of the frequency of vehicle loading on elements of bridge decks. The results indicated that fatigue fracture of the internal reinforcing steel would appear to be dominant factor in the failure .Barnes and Mays suggested that it would appear reasonable to expect the same fatigue life for unstrengthened and strengthened beams with comparable values of stress range in the steel reinforcement.

Shahawy and Beitelman (1999) tested a total of sixteen T-beams under monotonic and cyclic fatigue loading. All beams were tested in third-point bending with a clear span of 5790mm and total depth of 445mm. The stirrups were tack-welded to the main reinforcing bars, which influences the fatigue life of the reinforcement. Some beams were then strengthened with one to four layers of biaxial CFRP fabric. Two types of CFRP configurations were used, strengthening only at the soffit and wrapped on three sides. One beam initially damaged in fatigue for 150,000 cycle and then rehabilitated with two layer of CFRP fabric. The objective of this test was to study the effect of strengthening on the extension of the fatigue life of severely deteriorated beams. The fatigue load was applied with a load ranging from 25 to 50 percent of the ultimate capacity of the statically tested specimens at a frequency of 1 Hz. The control beams failed at 295,000 cycle, whereas ,the strengthened beams failed at 1.8 to 3.2 million cycles. The specimens which was tested to 150,000 cycles and then repaired with two layers of CFRP fabric showed that the number of cycles at failure was similar to the strengthened beams without pre-damaged . The stiffness of the beams wrapped on three sides was equal to or greater than the control beam. The results indicated that the wrapping of beams with CFRP fabric is effective.

Papakonstantinou et al. (2001) conducted fatigue tests on seventeen concrete beams with and without GFRP composite sheets glued to the soffit of the beams. All beams were tested in third-point bending and were loaded at a frequency of 2 to 3 Hz. The failure of FRP strengthened cyclically loaded beams is dominated by fracture of the reinforcing steel. The failure was sudden with signs of severe damage appearing only a few cycles before failure. Debonding of the GFRP sheets was a secondary failure mechanism for the strengthened beams. The fatigue life of strengthened beams is increased comparing with the control beams at the same applied load. For the same applied stress range in the reinforcing steel, the fatigue life for the strengthened and control beams was the same.

Heffernan and Erki (2004) presented results of twenty 3.0m and six 5.0m beams loaded monotonically and cyclically to failure. The 3.0m beams were tested to provide a comparison between the fatigue behavior of both conventionally reinforced concrete beams and those strengthened with an identical cross-sectional area of CFRP sheets. The same steel reinforcement configurations were used for all the 3.0m beams and, thus, any changes to fatigue life were attributable to the presence of the CFRP sheets. The 5.0m beam tests were designed to validate the behavior observed in the smaller beam tests as well as to investigate the effect on fatigue life of varying the cross-sectional area of the CFRP sheets. The results of 3.0m and 5.0m beams showed a similar behavior in all instances with deviations in the observed strains from theoretical predictions being similar for both beam sizes. However, contrary to that observed in the 3.0m beam tests, no drop in the CFRP strains was observed in the 5.0m beams loaded cyclically to failure. All failures were initiated by fatigue fracture of one or more of the reinforcing steel bars. The fatigue life of the CFRP strengthened reinforced concrete beam

appeared to be at least as long as for equivalent strength conventionally reinforced concrete beam subjected to the same loads, where the fatigue life is largely dependent on the stress range applied to the steel reinforcement. The results indicated that concrete softening due to repeated loads leads to an increase in the stresses in the tensile steel reinforcement. These increases in steel stresses were not as severe for the CFRP strengthened beams as for the beams without CFRP sheets. No significant degradation in the CFRP sheets or the CFRP to concrete interface occurred due to cyclic loading and the basic assumptions for monotonic behavior remained valid for the cyclically loaded beams. Heffernan and Erki suggested that CFRP strengthening of reinforced concrete beams appears to be an appropriate procedure to re-establish the fatigue life of a reinforced concrete beam which has lost tensile steel due to corrosion or as a means to extend the fatigue life where necessary.

2.6 Modeling Research on FRP Strengthened RC Beams

Characterization and modeling of debonding in structural members strengthened with externally bonded reinforcement has long been a popular area of interdisciplinary research due to critical importance of debonding failures in bonded joints. In the last decades, there has been a concentration of research efforts in this area with respect to FRP strengthened flexural members, and considered progress has been achieved in understanding the causes and mechanisms of debonding failures. Research studies in this area can be classified in general terms into four categories:

- Section analysis models;
- Empirical models;
- Finite element models; and
- ACI Committee 440 .

2.6.1 Section Analysis Models

An et al. (1991) developed a model to predict the stresses and forces of a reinforced concrete beam with externally applied GFRP. Their study was based on five assumptions: (1) linear strain distribution throughout the beam, (2) small deformation; (3) tensile strength of concrete was ignored, (4) shear deformation was ignored, (5) perfect bond between concrete and GFRP. Using classical flexural theory and strain compatibility (Figure 2.5), effects of variables such as material strength, modulus of elasticity, and reinforcement ratios of the steel and GFRP were compared with the experimental results of a previous test conducted by Saadatmanesh and Ehsani (1991).

The same approach with little modifications was followed by many researchers (Triantafillou and Plevris 1991, Saadamanesh and Malek 1998, Duthinh and Starnes 2001, Alagusundaramoorthy et al. 2002, Brena et al. 2003). In all these studies full composite action was assumed to exist between the plate and concrete beam (no slip at the interface), therefore, these models failed to capture the debonding failure modes.

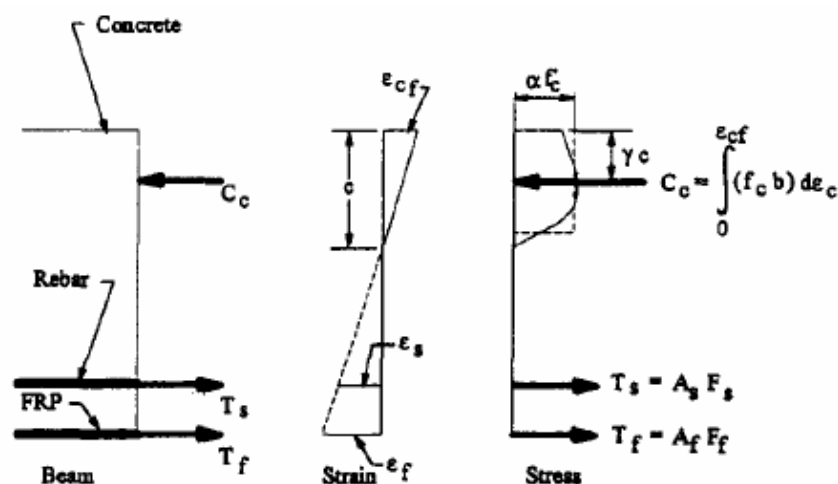


Figure 2.5: Stress and strain distribution (Saadatmanesh and Malek, 1998)

2.6.2 Empirical Models

Chen and Teng (Teng et al. 2002) recognized the similarities between intermediate crack-induced debonding and simple shear tests (Figure 2.1) , and hence proposed a model based on the bond strength model that they developed (Chen and Teng ,2001). Through finite element study (Chen et al. 2001), they realized that the stress distribution in debonded beams is different from those in bonded joints, particularly the peak values in the adhesive near an intermediate flexural crack. They made modifications to the bond strength model by recalibrated it with a collection of test data of beams and slabs and proposed the following equation for determining it with a collection of test data of beams and slabs and proposed the following equation for determining the IC debonding stress,

$$\sigma_{IC} = \alpha \beta_p \beta_L \sqrt{\frac{E_{FRP} \sqrt{f'_c}}{t_{FRP}}} \quad \dots (2.1a)$$

where,

$$\beta_p = \sqrt{\frac{2 - b_p/b_c}{1 + b_p/b_c}} \quad \dots (2.1b)$$

$$\beta_L = \begin{cases} 1 & \text{if } L \geq L_e \\ \sin\left[\frac{\pi L}{2L_e}\right] & \text{if } L < L_e \end{cases} \quad \dots(2.1c)$$

and,

$$L_e = \sqrt{\frac{E_p t_p}{\sqrt{f'_c}}} \quad \dots(2.1d)$$

The coefficient α was calibrated using a large number of pull tests with steel and FRP plates. It is suggested that for design a value of 0.4

should be used. Although the equation works well for pull tests, it can be seen from Table 2.1 that the average and characteristics (95%) values for α vary significantly for both beams and slabs, which indicates there is large scatter in results. There is also a variation in the values of α for beams and for slabs. This means that Chen and Teng's model does not sufficiently identify all the parameters that influence the IC debonding of beams (Lundqvist, 2007). The fact that the α value for beams (0.887) is greater than that for pull tests (0.427) indicates that the distributed flexural cracks in beams allows greater plate forces to be achieved ,and hence, the IC debonding of flexural members cannot be directly idealized into a simple pull test.

Table 2.1: IC debonding α coefficient (Teng et al., 2002)

α	Pull tests	Beams	Slabs	Beams & slabs
mean	0.427	1.100	0.720	0.887
95% characteristic	0.315	0.544	0.478	0.379

Recognizing that IC debonding is the most dominant debonding failure mode especially for beams strengthened with thin plates, Niu and Wu (2001a,b) performed studies to predict IC debonding initiation and failure based on fracture energy. Niu and Wu believed that the bond strength is responsible for initiation of debonding while the propagation is governed by the interfacial energy, and hence, quantified the interfacial fracture energy for debonding based on test results and proposed equation to predict the debonding failure load for plated members where a single flexural crack has formed . Niu and Wu (2001b) noted that the shear stress transferred near flexural cracks in plated beams with a single crack

is similar to that in simple shear tests. From mathematical solutions based on the flexure energy method, the maximum transferable load $P_{\max}$ in a simple shear test can be determined from Equation 2.2, provided that the bonding length is larger than the effective bond length L_e .

$$P_{\max} = b_2 \sqrt{2G_f E_2 t_2} \quad \dots (2.2)$$

where E_2 , t_2 , b_2 and G_f are elastic modulus, thickness, width of FRP and fracture energy, respectively. Niu and Wu also proposed an analytical method to predict the response of the plated beams (Niu and Wu, 2001b). This method, however, neglects the premature debonding failure of beams; with the assumption slip does not take place at both the bar/concrete and the plate/concrete interfaces, and the tensile strength of the adhesive is ignored.

Through their studies, Niu and Wu (2001a, b) found that for beams with distributed flexural cracks, the ultimate load at which debonding occurs can be significantly higher than that obtained from simple shear tests. Hence, they modified Equation 2.2 by assuming that the flexural cracks are smeared in the beam as illustrated in Figure 2.6 and the debonding failure is predicted when complete debonding occurs between two flexural cracks with a spacing of an effective transfer length given by Equation 2.3.

$$L_e = k \sqrt{E_2 t_2} \quad \dots (2.3)$$

where k is a constant determined from experiment.

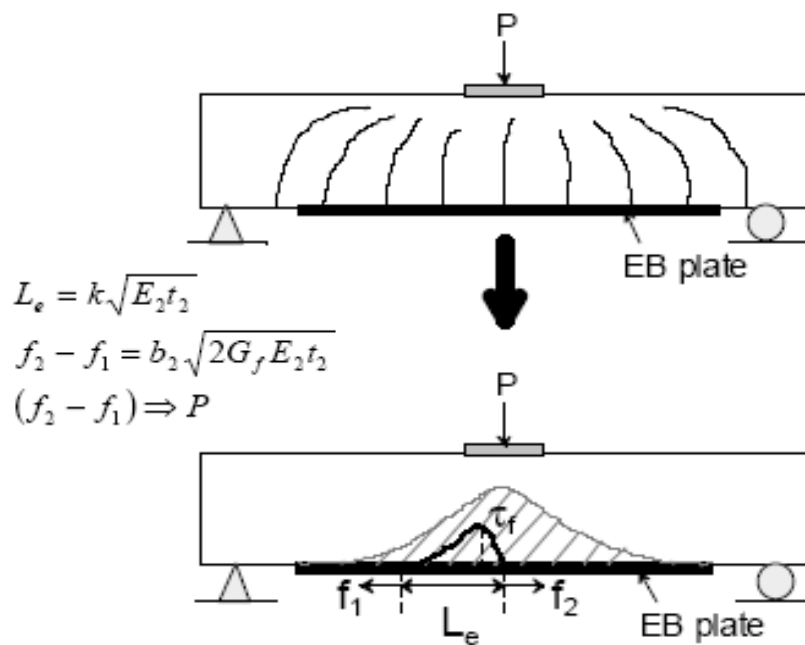


Figure 2.6: IC debonding mechanism (Niu and Wu, 2001b)

2.6.3 Finite Element Models

Finite Element (FE) modeling is a powerful tool for the analysis of concrete structures with almost continuous development since the 1960's.

Arduini et al. (1997) modeled RC beams strengthened with CFRP plates and sheets using a commercial package (ABAQUS) based on a smeared cracking approach. Beams strengthened with CFRP plates were analyzed using a two dimensional mesh, while those bonded with CFRP sheets were modeled in three dimensions. The FRP reinforcement was applied directly over the concrete elements assuming perfect bond. The authors reported that the results of the FE analysis showed a good correlation with the experimental data. However, the FE analysis results were stiffer than the test results. This was attributed to the perfect bond assumption and the limited number of nodes that could be used. The FE

analysis indicated that high stresses at the end of the FRP plate caused the delamination failure of the beam.

Ross et al. (1999) used ADINA to analyze their test beams. The beams were 200mm square and 3050mm long and had a clear span of 2740mm using four points bending. Two dimensional, eight-node plane stress elements were used to represent the concrete, while the reinforcing steel and FRP plates were modeled by three-node truss elements. A hypo-elastic constitutive relationship was used for the concrete based on uniaxial stress-strain relationship that accounts for biaxial and triaxial conditions. An elastic-plastic response was assumed for the steel and the CFRP was modeled as linear elastic until failure. The results of the FE analyses were compared to the experimental results and good agreement for the global behavior observed. However, the predicted failure mode was not achieved due to the delamination of the FRP plate. It was concluded that the single most important factor affecting the beam behavior was the bond strength between the concrete and FRP. The use of an anchorage system was suggested to prevent debonding and to utilize the full capacity of the plate.

Limam and Hamelin (1998) used a two-dimensional nonlinear FE analysis to calculate the response of reinforced concrete beams with CFRP sheets bonded to the tensile face. Eight-node membrane elements were used to represent the concrete and two-node truss elements were adopted for the steel and CFRP reinforcement. Perfect bond was assumed between the steel reinforcement and the concrete, bond slip was considered at the concrete-FRP interface through the use of two-node continuous contact elements (Figure 2.7). The concrete model was based on different yield surfaces in the tensile and compressive regions using a smeared crack approach. The steel reinforcement was modeled using an elasto-plastic model with strain hardening. The results from experimental

tests were used to define a constitutive law for the interface elements based on Mohr-Coulomb failure criteria. Comparisons of numerical results with the experimental data showed that the ultimate load and deflections were within 10 percent of the measured values but were dependent on the modeling of the interface layer. They concluded that a non-realistic model for the interface leads to large discrepancies between the analytical and the test results showing the importance of accurately modeling the concrete-CFRP interface.

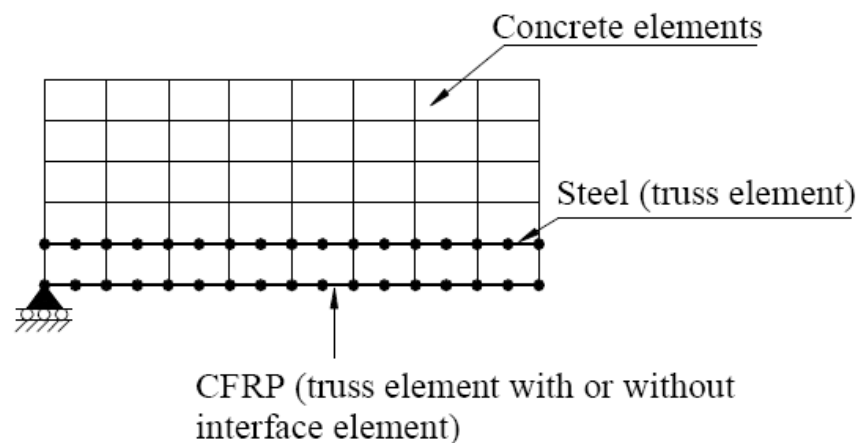


Figure 2.7 : Finite element mesh of Limam and Hamelin (1998)

Shin and Lee (2003) used a layered finite element method to analyze strengthened reinforced concrete beams. In this method, strain compatibility is assumed with each cross section of a member divided into layers (Figure 2.8). Some of the layers represent the reinforcing bars or externally reinforced bonded plates or CFRP laminates with other layers representing the concrete. Based on beam theory, the model was modified to take into account the effect of strengthening on the flexural behavior of reinforced concrete beams. The modification was made assuming perfect adhesion between the bottom fiber of the beam and the

CFRP laminates. The constitutive models adopted used a strain softening relationship for the concrete and an elastic-plastic model for the steel reinforcement. The CFRP was taken as elastic brittle.

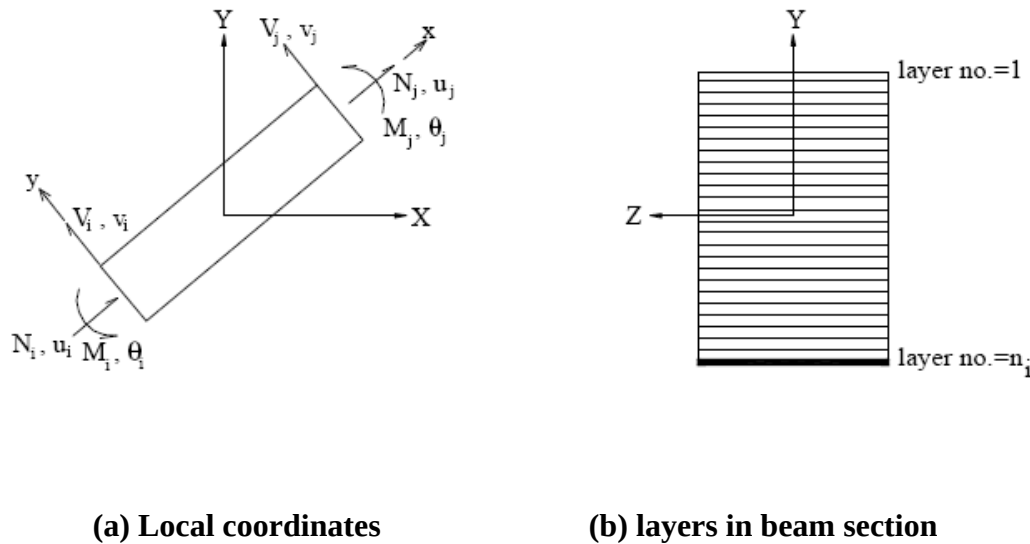


Figure 2.8: Beam element and layers in section (Shin and Lee, 2003)

Al-Shmmmary (2006) and Mohammed (2007) modeled RC beams strengthened with CFRP plates and sheets using ANSYS computer program (version 9.0, 2004) under Microsoft Windows XP (service pack 1, 2002) based on a smeared cracking approach. Perfect bond was assumed between CFRP sheets and concrete interface. This method, however, neglects the premature debonding failure of beams; with the assumption slip does not take place at the CFRP plate/concrete interface.

2.6.4 ACI Committee 440

The use of externally FRP to strengthen and rehabilitate exiting concrete structures is a relatively new technique. Until recently, there is a lack of design code and guidelines for their use (Al-Mahaidi, 2003).The

design specification of ACI Committee 440 recommended using a limitation on the strain of FRP plates using a bond dependent coefficient in order to prevent debonding of FRP plates. Section 9.5 of ACI 440.2R-02 discusses fatigue stress limits in the CFRP. To mitigate debonding under static loading conditions, ACI 440.2R-02 limits the strain in the FRP to $k_m \varepsilon_{fu}$, where ε_{fu} is the rupture strain of the FRP and,

$$k_m = \frac{1}{60\varepsilon_{fu}} \left(1 - \frac{nE_f t_f}{360,000} \right) \leq 0.9 \quad \text{for } nE_f t_f \leq 180,000 \text{ N/mm} \dots (2.4a)$$

$$k_m = \frac{1}{60\varepsilon_{fu}} \left(\frac{90,000}{nE_f t_f} \right) \leq 0.9 \quad \text{for } nE_f t_f \geq 180,000 \text{ N/mm} \dots (2.4b)$$

where n is the number of layers of FRP and E_f and t_f are the modulus and thickness of a single layer of FRP. This limitation was derived based on visual observation and experience, but the theoretical justification is unclear (Al-Mahaidi, 2003). These strain limits depend only on the stiffness properties of the FRP plates and do not take into account the concrete-FRP interface properties, width of the FRP plate or the plate curtailment location, all of which should be expected to affect debonding (Achintha and Burgoyne, 2006).

CHAPTER THREE

EXPERIMENTAL WORK

3.1 General

The main objective of the present study is to investigate the behavior of reinforced concrete beams strengthened with externally bonded carbon fiber reinforced polymers (CFRP) plates. Seven specimens were performed for the purpose of investigating the responses of strengthened RC under monotonic and repeated loading. One specimen was aborted due to problems with casting operations, resulting in a total of six successful tests.

First, the test program, which describes the experimental variables considered in the present work, details of the test specimens and strengthening schemes is described. Next, properties of the constituent materials used in manufacturing the specimens are tabulated. Finally, the experimental setup and instrumentation part of the test program will be explained.

3.2 Test Program

The beams were designed to study the flexural response of strengthened RC beams. The first assumption was to use a typical fraction of the balanced steel reinforcement ratio, ρ_b (defined as the steel ratio that results in simultaneous yielding of the reinforcement and crushing of the concrete at the extreme compression fiber). Since reinforced concrete beams are usually designed so that steel reinforcement yields first, a ratio of $0.5\rho_b$ was deemed appropriate (Carlin, 1998). This ratio would not cause the highest increase in strength after applying the CFRP. It was shown by an earlier study that for beams with the minimum steel

reinforcement ratio (defined by ACI 318-2005 as $1.4/f_y$) strength increase can be larger than 100% (An et al., 1991). Since using the minimum steel reinforcement in a bridge or building is not typical, it seems suitable to study the effects of CFRP strengthening on concrete beams with higher steel reinforcement ratio.

The next step was to decide the actual geometry of the specimens. The capacity of the available arm limited the size of the beam that could be tested. Using this constraint and the analytical model presented by Al-Mahaidi (2003), a maximum load was predicted. With a spreadsheet, many different dimensions were investigated until the most appropriate was chosen.

The effect of external loading on structures exhibit seldom pure flexure alone. Shear is most commonly encountered in combination with flexure. The ACI design approach is based on ultimate strength requirements. Design shear strength at each section of a member is required. It is achieved by combined contributions of concrete and shear reinforcement. A critical part of the design was ensuring the beams failed due to flexure and not shear. To accomplish this, the specimens were heavily reinforced for shear. This design was based on ACI 318-2005, section 11.

3.2.1 Specimens Identification and Strengthening Schemes

In order to identify the test specimens with different reinforcement ratios and strengthening schemes, the following designation system is used:

- Amount of reinforcement: (10) for two 9.5mm diameter reinforcing bars and (12) for two 12.7mm diameter reinforcing bar.

- Type of beam: (U) for unstrengthened control beam specimen, and (S) is used to indicate if the beam is strengthened in the longitudinal direction.
- Number of CFRP plates of longitudinal direction: (0) for nil strengthening, (1) for one plate, and (2) for two plates.
- Type of loading: (M) for monotonic loading, and (C) for cyclic (repeated) loading.

3.2.2 Specimens Description

In this work, four CFRP-strengthened RC beams and two control beams without strengthening were tested. All beams were 2000mm long, and had 140mm wide, 240mm high cross section. Figure 3.1 shows the dimensions and the reinforcement arrangement of the specimens.

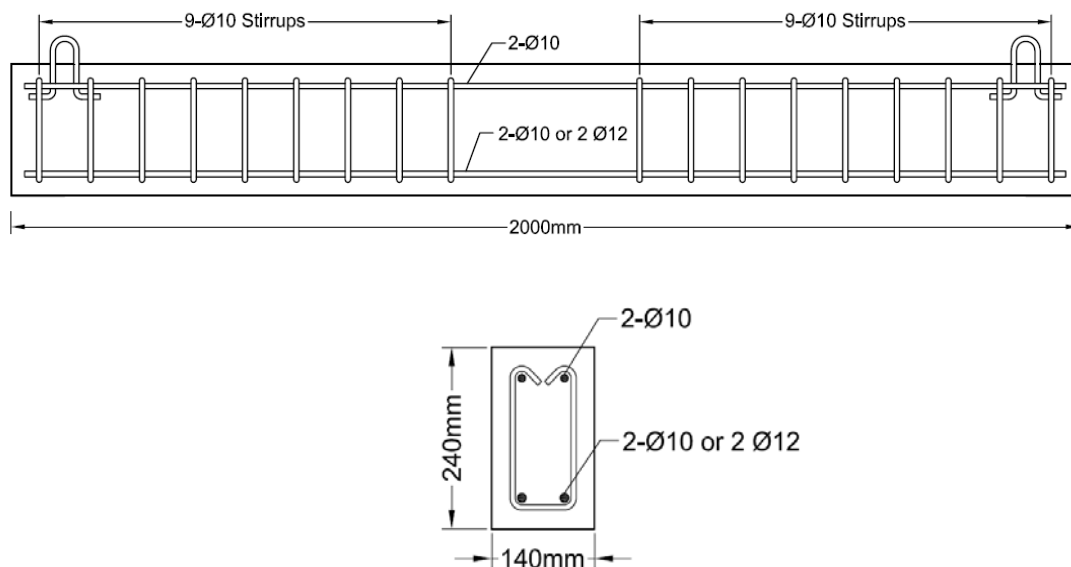


Figure 3.1: Reinforcement details of the tested beam specimens

These specimens were separated into series 10 with reinforcement ratio of 0.51 percent (two 9.5mm diameter rebar) and series 12 with

reinforcement ratio of 0.92 percent (two 12.7mm diameter rebar). The U-shaped stirrups of 10mm diameter reinforcing bar were spaced at 100mm along the beam length. All beams were designed for flexural failure in order to study the flexural behavior of the strengthened beams.

The parameters considered in this study are the amount of internal reinforcement, amount of external reinforcement, and type of loading monotonic or repeated loading. Table 3.1 in conjunction with Figure 3.2 illustrates the specimens identification system used that is based on the specimen identification pattern described above.

Table 3.1: Specimen identification and strengthening schemes

Specimens	Area of steel, mm²	Area of CFRP, mm²	End anchorages	Testing conditions
U10M0	142	Nil	Nil	Monotonic
S10M1	142	60	Ends only	Monotonic
S10M2	142	120	Ends only	Monotonic
U12M0	258	Nil	Nil	Monotonic
S12ME	258	120	Ends only	Monotonic
S12CE	258	120	Ends only	Repeated

The first concrete beam specimen in each series (U10M0 and U12M0) was kept without strengthening and was considered as a control beam for comparison. To study the effect of amount of external reinforcement, beam specimen S10M1 was strengthened with 50mm width CFRP plate, while the companion beam specimen S10M2 was strengthened with 100mm width CFRP plate.

The strengthened beam specimens in series 12 (S12ME and S12CE) were strengthened with two CFRP plates. The first beam

specimen tested under increasing monotonic load while the other beam specimen tested under repeated load.

It was believed that the CFRP Plate End Interfacial (PEI) debonding and/or Concrete Cover Separation (CCS) may be caused by the significant stress concentration at the CFRP plate end from geometrical and flexural stiffness discontinuities. To prevent plate end debonding or concrete cover separation, CFRP sheets were used as U-jacket at the CFRP plate ends in beam specimens S10M1, S10M2, S12ME and S12CE. The anchorages in these specimens had 225mm width, 0.13mm thickness and was extended from the bottom tension face to both side faces of the beam specimen.

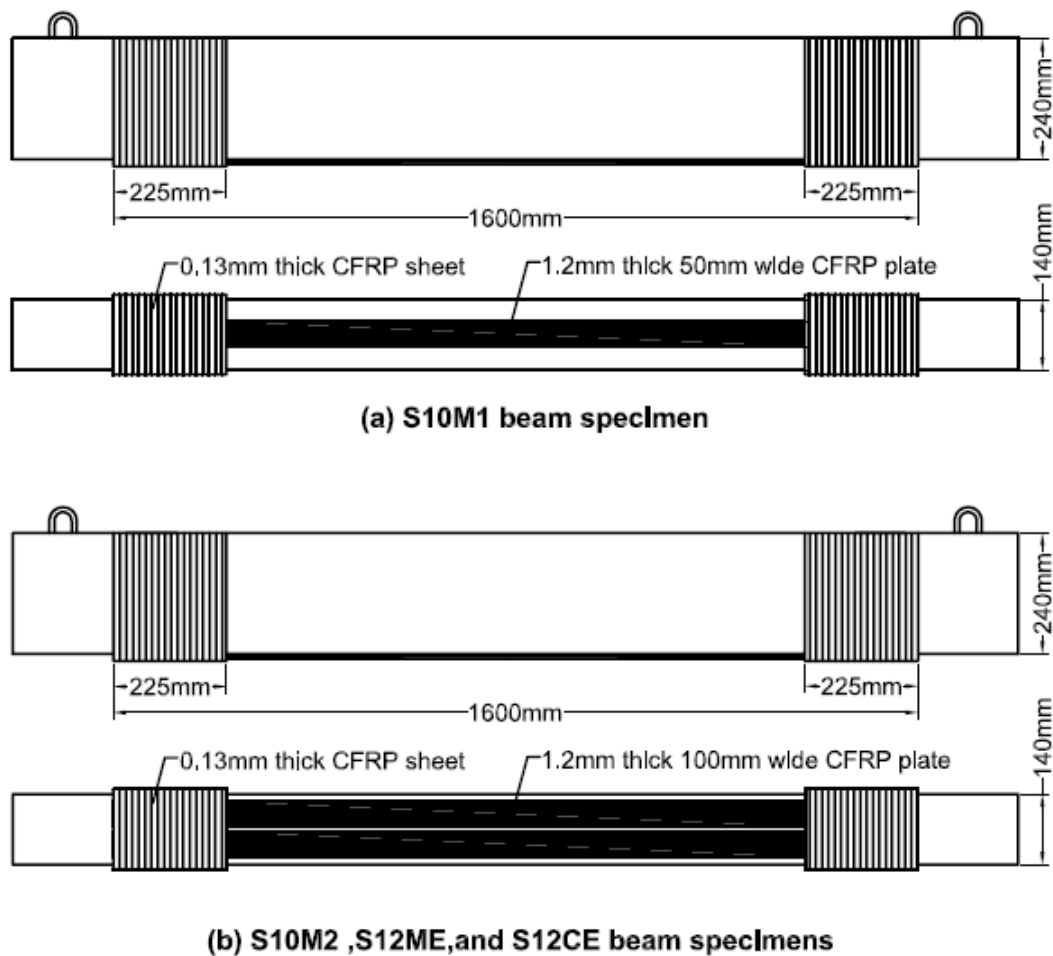


Figure 3.2: Schematic representation of CFRP strengthening schemes

3.3 Materials

3.3.1 Cement

Ordinary Portland cement (Type I) of Al-Kufa mark was used in casting all the specimens. The cement was kept in closed plastic containers throughout the experimental work to keep the cement in good conditions to minimize the effect of humidity. The cement properties are conformed to the IRQ 5/1989 specifications limits for Ordinary Portland Cement.

3.3.2 Fine Aggregate

Al-Akhaider natural sand was used as fine aggregate. The fine aggregate was sieved at sieve size (4.75mm) to separate the aggregate particle of diameter greater than 4.75mm. The sand was then washed and cleaned with water several times, later it was spread out and left to dry in air, after which it was ready for use. The grading of the sand was conformed to the requirements of ASTM C33-2003 as shown in Table 3.2.

Table 3.2: Grading of fine aggregate

Sieve Size, mm	Passing ,%	ASTM C33-03
9.50	100	100
4.75	100	95-100
2.36	82	80-100
1.18	67	50-85
0.60	49	25-60
0.30	21	5-30
0.15	3	0-10

The fineness modulus of the sand used in this research was 2.78. It was necessary to use sand with fineness modulus greater than 2.5 since otherwise concrete consistency would be sticky and concrete compaction would be difficult (Neville 1995).

3.3.3 Coarse Aggregate

Locally available gravel of (19mm) maximum size was used. The gravel was sieved at sieve size (19mm). The gravel was washed and cleaned by water several times, later it was spread out and left in air to dry before use. Gradation of coarse aggregate conformed to requirement of ASTM C33-2003 as shown in Table 3.3.

Table 3.3: Grading of coarse aggregate

Sieve size, mm	Passing, %	ASTM C33-03
25.0	100	100
19.0	100	90-100
12.5	44	20-55
9.5	12	0-15
4.75	0	0-5

3.3.4 Mixing Water

Ordinary clean tap water was used for casting and curing all the specimens.

3.3.5 Steel Reinforcing Bars

Longitudinal reinforcing steel bars, which were of Ukrainian origin and distributed by the Local General Company of Construction Materials, has crescent deformation pattern (C Pattern) (deformation pattern C

consists of diagonal ribs inclined at an angle 60 degree with respect to the axis of the bar).

Two tension coupons of each 9.5mm and 12.7mm diameter deformed steel bars were tested to acquire the yield stress and modulus of elasticity of the reinforcement. Each of specimens was machined to remove the ribs and create a smooth surface at the mid-length. The coupons were tested in uniaxial tension using a testing machine. The reinforcing bars have been tested in the Material Laboratory of the Material Engineering Department at Al-Kufa University and the experiments were conducted in accordance with ASTM A370-2005 specifications. Configuration of tested steel specimen is given in Figure 3.3.

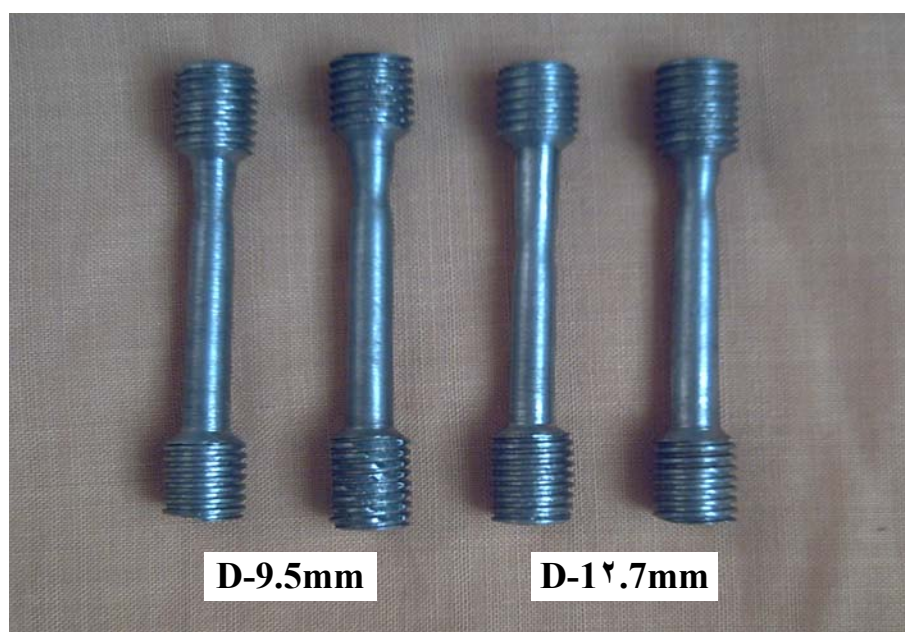


Figure 3.3: Tested steel reinforcing bar specimens

Details of the materials properties for tested reinforcing steel are given in Figure 3.4 in conjunction with Table 3.4. Properties of steel

reinforcement of conformed to requirements of ASTM A 615/A 615M-05a Specifications.

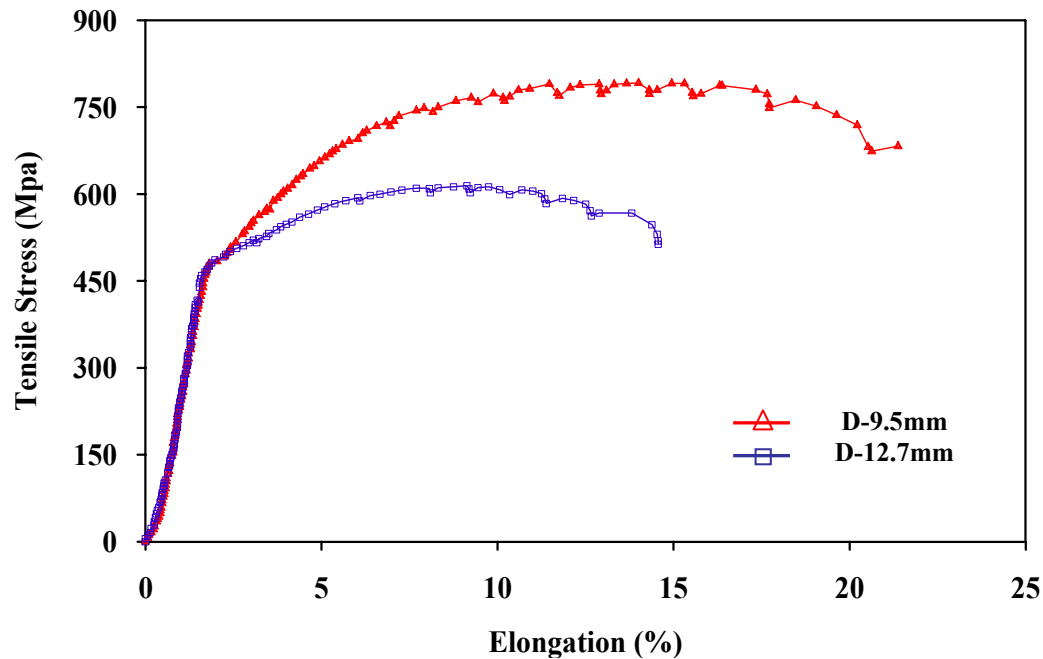


Figure 3.4: Stress-strain diagram for reinforcing steel

Table 3.4: Material properties for steel reinforcement

Properties	D-9.5mm	D-12.7mm
Cross-sectional area, mm ²	71	129
Yield stress , MPa	469.2	459.5
Ultimate strength, MPa	791.6	614.3
Elongation , %	21.4	14.6

3.3.6 Carbon Fiber Reinforced Polymer (CFRP)

When loaded in tension, CFRP fibers do not exhibited any plastic behavior (yielding) before rupture (Figure 3.5). The tensile behavior characterized by a linearly elastic stress-strain relationship up to failure, which is sudden and can be catastrophic. SikaCarboDur[®] type S512 was

used as flexural strengthening. While, SikaWarp[®] Hex-230C was used for U-jacket (see Figure 3.6).

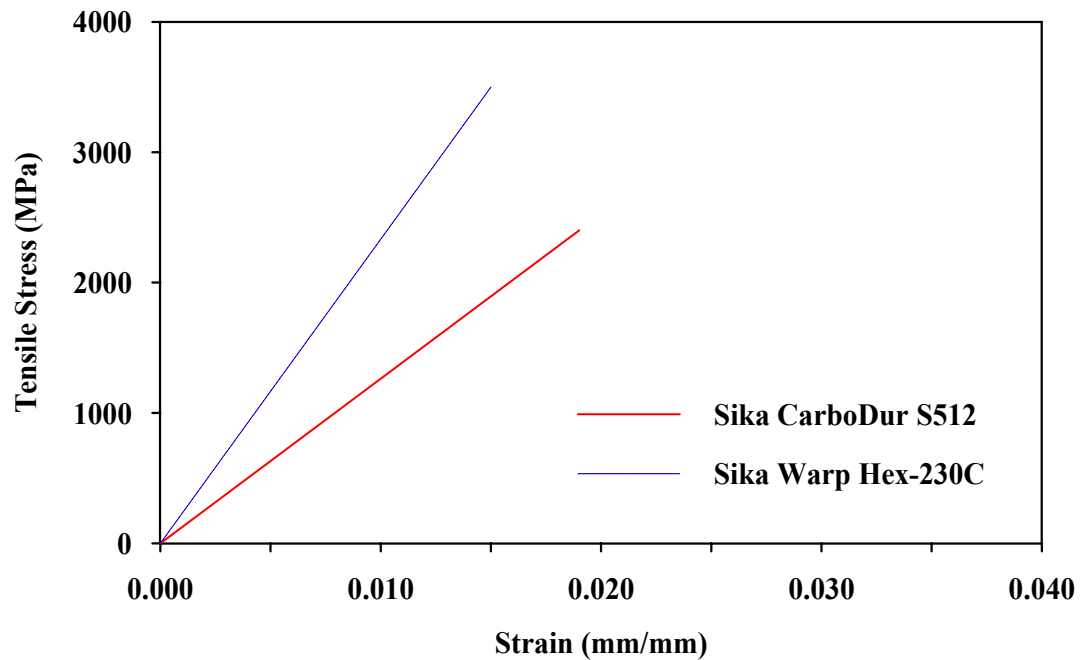


Figure 3.5: Stress-strain diagram for used CFRP laminates

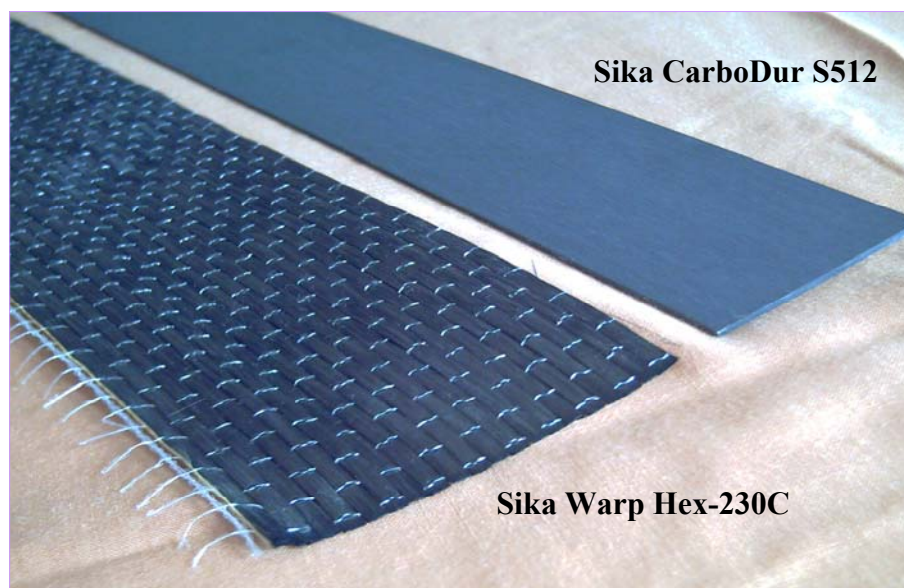


Figure 3.6: Photo of CFRP strengthening laminates

The mechanical properties of CFRP laminates used here are taken from manufacturing specifications (Sika, 2005) and are given in Table 3.5.

Table 3.5: Technical properties of CFRP laminate (Sika, 2005)

Properties	SikaCarboDur[®] S512	SikaWarp[®] Hex-230C
Tensile strength (MPa)	>2800	>3500
E-modulus (GPa)	165	230
Elongation at break (%)	>1.7	>1.5
Width (mm)	50	305
Thickness (mm)	1.2	0.13

3.3.7 Bonding Materials

The building and construction industries represent some of the largest users of adhesive materials, many applications are non-structural in the sense that the bonded assemblies are not used to transmit or sustain significant stresses (e.g. crack injection and sealing, skid-resistant layers, bonding new concrete to old).

Truly structural application implies that the adhesive is used to provide a shear connection between similar or dissimilar materials, enabling the components being bonded to act as a composite structural unit (Hollaway and Leeming, 2001).

Sikadur[®]-30 (epoxy adhesive) and Sikadur[®]-330(impregnation resin) were used in this work for the bonding of CFRP plates and sheet, respectively (Figure 3.7).



Figure 3.7: Two component epoxy adhesive

The technical properties of the bonding materials used in this study as given by the manufacturer are presented in Table 3.6.

Table 3.6: Technical properties of bonding materials (Sika, 2005)

Properties	Sikadur [®] -30	Sikadur [®] -330
Tensile strength , MPa	> 25	> 30
Bond strength , MPa	Concrete fracture	Concrete fracture
E-modulus , GPa	12.8	4.5
Elongation at break , %	---	0.9
Open time , min.	30 (at +35°C)	30 (at +35°C)
Full cure , days	7(at +35°C)	7(at +35°C)
Mixing ratio	1:3	1:4

3.4 Preparation of Test Specimens

3.4.1 Concrete Mix Proportions

During the design phase of the experimental program, (25MPa) concrete compressive strength was chosen to mimic an older structural member that would be subject to strengthening. It was decided to adopt the method of mix design given by Neville (1995). The mix was proportioned by weight at 1:1.5:3:0.50 for cement, sand, gravel and water respectively. Locally available materials were used throughout.

3.4.2 Fabrication

Seven sets of plywood formworks were fabricated. The formworks were prepared with internal dimensions of 140 mm width, 240 mm height and 2000 mm length. The moulds were made from wooden frame for the edge and plywood blocks were used for the base to obtain smooth surfaces. The reinforcement cage and formwork are shown in Figures 3.8.



Figure 3.8: Wooden formworks and reinforcing cage

3.4.3 Concrete Casting and Curing

Seven beam specimens were cast and cured under laboratory conditions at the Civil Engineering department of the University of Babylon. Also, five standard cylinders (305x152) mm were cast from the concrete of each beam specimens. The concrete casting and curing procedure is described below:

- Beam specimens and cylinders were treated with oil before putting the reinforcement cage or casting control specimens
- Before mixing, all the quantities were weighted and backed in a clean plastic container.
- Prior to starting rotation of the mixer add the coarse aggregate, some of the mixing water. Start the mixer, then add the fine aggregate, cement, and water with mixing running. Mix the concrete, after all ingredients are in the mixer, for 3 min. followed by a 3 min rest, followed by 2 min final mixing. To eliminate segregation, deposit machine-mixed concrete in the clean, damp mixing pan and remix by hand trowel until it appears to be uniform.
- Concrete was poured in the moulds in two layers, and each layer was compacted using a plunger mechanical vibrator (3000 vibration per minutes) having a metal rod with diameter of 50 mm for 5 second for each insertion (see Figure 3.9).The upper surface of concrete was smoothly finished after casting was completed using hand trowel.
- Beam and control specimens were removed from their forms and molds, respectively, within 24 hours and then burlap sacks were placed over the beams and wetted down. Then a plastic sheet was placed over the beams. The burlap sacks were monitored and kept wet until the fully twenty-eight days had past.

- Once the beams were cured, the beams were placed off to the side until they could be tested. The same procedures were performed on the concrete test cylinders.

One beam specimen was aborted due to problems with casting operations, resulting in a total of six successful beam specimens.



Figure 3.9: Casting beam specimens

3.4.4 Surface Preparation

The most crucial part of any strengthening application is the bond between the FRP and the surface to which the FRP is bonded. Proper bond ensures that the force carried by the structural member is transferred effectively to the FRP (Al-Mahaidi, 2003).

Before the CFRP was applied to the soffit of the beams, the surface of the concrete were grinded using an electrical hand grinder to expose the aggregate and to obtain a clean sound surface, free of all contaminants such as cement laitance, and dirt (see Figures 3.10 and 3.11).



Figure 3.10: Concrete surface preparation by electrical grinder



Figure 3.11: Exposed aggregate after surface preparation

3.4.5 Application of CFRP Composites

External strengthening followed the procedure recommended by the manufacturer which is described below.

- First of all, the CFRP was cut into the required lengths. Surface preparation of the CFRP followed with sanding of the surface and cleaning to remove any grease, oils, carbon dust or other contaminants prior to installation.
- A two-part adhesive (black and white) was mixed in required proportion, until the color was a uniform grey, and then applied with a special tool to the concrete surface to a thickness of 1.5 mm. The adhesive was also applied to the laminate to the same thickness.
- The laminate was then placed on the concrete ,epoxy to epoxy ,and a rubber roller was used to properly seat the laminate by exerting enough pressure so the epoxy was forced out on both sides of the laminate and the adhesive line did not exceed 3 mm in thickness.



Figure 3.12: Applying epoxy adhesive to CFRP laminates

- Timber packing and props were used to hold the CFRP laminates in place (Figure 3.13).
- After the CFRP installation was complete, the beam was cured at ambient temperature for at least 7 days before testing.



Figure 3.13: Timber packing and props to hold the CFRP plates

3.5 Concrete Cylinders

The actual mean compressive strength, f_c' results were obtained from the average of at least three 305mm by 152mm diameter cylinders tested in conjunction with test beams.

Three cylinders were tested in uniaxial compression with each beam specimens at the same time to obtain the compressive strength using a Universal Testing Machine in accordance with ASTM C39/C39M-05(2005). The remained two cylinders were tested for indirect tension by applying a line load along the 305mm long side to obtain the splitting tensile strength, which is the a measure of the tensile strength of concrete (ASTM C496/C496M-04).

The cylinder test results are summarized in Table 3.7 together modulus of elasticity and tensile strengths.

Table 3.7: Tests results for concrete cylinders

Specimens	Compressive strength, MPa	Tensile strength, MPa	*Modulus of elasticity, MPa
U10M0	24.08	2.24	23211
S10M1	24.73	2.57	23522
S10M2	25.81	2.75	24030
U12M0	24.47	2.59	23398
S12ME	27.69	2.67	24890
S12CE	25.38	2.73	23829
$\bar{x}$	25.36	2.59	23813
S_{n-1}	1.3	0.19	604
C.O.V(%)	5.1	7.3	2.5

$$*E_c = 4730\sqrt{f'_c} \text{ (ACI 318-2005)}$$

3.6 Test Setup

After curing, the beams specimens are transported to the Structural Laboratory of Civil Engineering Department in Al-Kufa University, this is to test them under monotonic and cyclic loading.

All beam specimens are tested in four-points loading. Beam specimens were tested as simply supported beams over 1800mm span in 2000 kN capacity hydraulic machine. Each beam specimen was supported and loaded by rollers. Forces were distributed through steel bearing plate 150mm in length to cover the entire Beam width. To observe crack development, beam specimens were painted white with emulsion paint

before testing. Cracks were traced by pencil. Figure 3.14 shows a schematic diagram for loading arrangement.

After the preparations were finished and the initial readings of the Demec discs were taken, the load was applied with a loading increment rate of about 100 N/sec. and deflection readings of dial gauge were taken at each 2.5 kN for the first 25 kN loading to specify the cracking loads, then the readings were taken at each 5 kN for the rest of loading stages.

Concrete strains were recorded at selected levels of loading. In addition to that, cracks were detected and drawn on the side face of the tested beam.

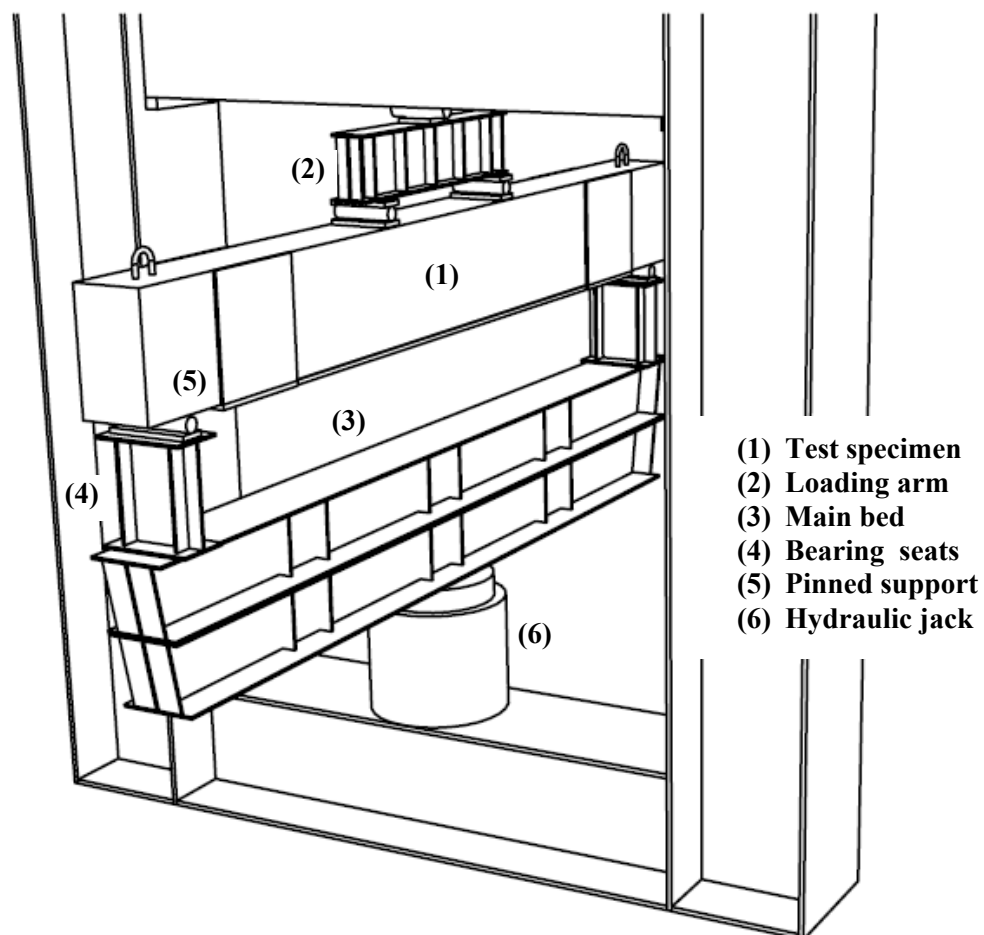


Figure 3.14: Schematic diagram showing the test rig

3.6.1 Instrumentation

During testing, the main characteristics of the structural behavior of the specimens were detected at every stage of loading. At each test, the first cracking load and ultimate load were recorded. The devices and the measurements recorded for each instrument are presented in Table 3.8 .In addition; the purpose of each measuring device used in the test setup is explained.

Table 3.8: Instrumentation details

Device	Measurement	Purpose
Demec points D1-D6	Strain	To monitor the strain in concrete surface.
Demec point D7	Strain	To monitor the strain in CFRP plate.
Dial gage	Deflection	To record the beam deflection.

The measurements, which were recorded during the tests, are the following:

- Strain in concrete.
- Strain in CFRP plates.
- Deflection.

Deflection of the beam specimens was measured at mid-span using a dial gage with travel distance of 50 mm and accuracy of 0.01 mm.

An ELE mechanical strain gage having an accuracy of 0.002 mm was used to measure the concrete and CFRP strains. Seven pair of Demec discs was used for each tested beam specimen. Demec discs were calibrated using an accompanying special ruler. Arrangement and distribution of these Demec discs are shown in Figure 3.15.

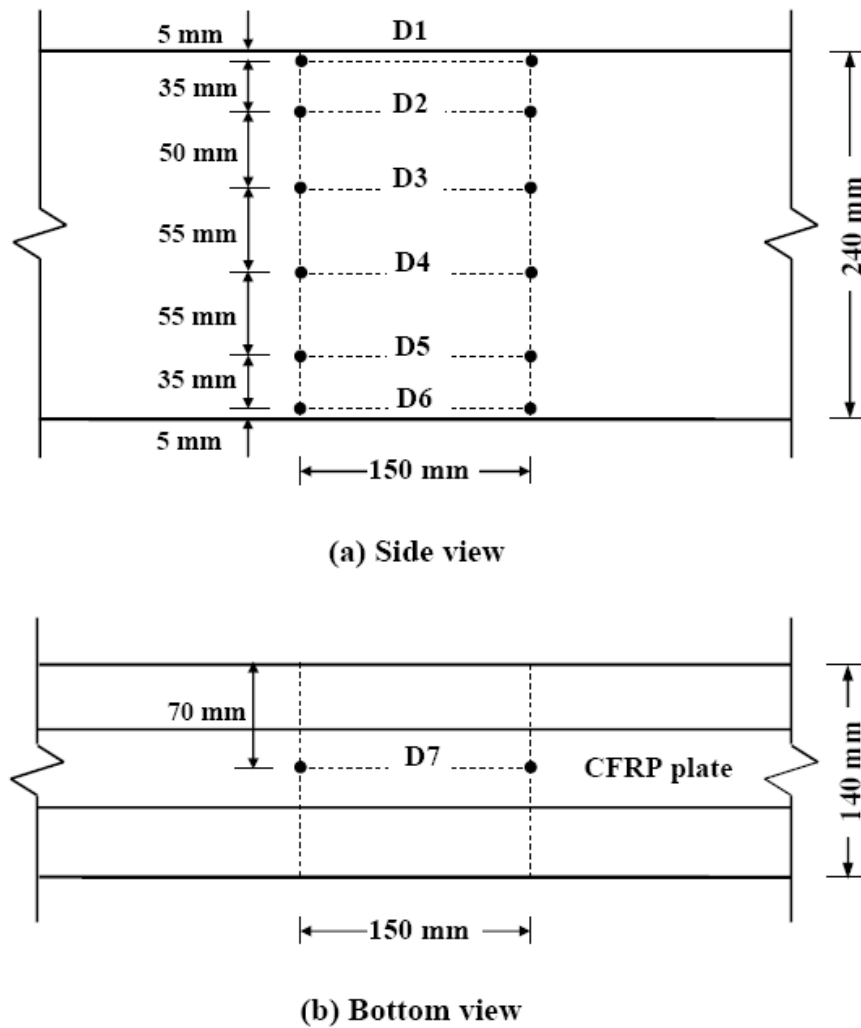


Figure 3.15: Configuration of Demec points

3.6.2 Testing Procedure

Test started with the application of 2.5 kN load to set and check dial gauge, then unloading to zero. At zero loading, initial reading of dial gauges and mechanical strain gauges were obtained. The load was applied in stages. At each load increment, observations of crack development on the concrete beams were marked with a heavy felt pen and the mechanically measured Demec strains reading were taken. The process of reading the gauges and crack observations took approximately five to ten minutes. Once this process was complete loading was resumed to the next

load step. In repeated load case, when the loading was reached to a pre-determined level, unloading was carried out gradually by loosening the valve of the hydraulic jack until a zero load reading was obtained. For another loading cycle, the same procedure was followed. The load was continued until ultimate load (defined as the highest capacity beyond which loading dropped).

CHAPTER FOUR

FINITE ELEMENT MODEL

AND MATERIALS IDIALIZATION

4.1 General

The finite element method is now firmly accepted as the most powerful general technique for the numerical solution for a variety of problems encountered in engineering.

This chapter presents a two-node displacement-based strengthened RC beam model with bond-interface. The element consists of a two-node beam element, plus two-node bar element, which allowed slipping with respect to the beam. The nodal degrees of freedom of the beam and the axial member are different to permit axial member slip. The section deformations and bond slips are determined through the beam compatibility and the bond-interface compatibility, respectively. Therefore, the compatibility conditions are satisfied pointwise. On the other hand, the equilibrium conditions are only satisfied in the integral sense through the Principle of Total Potential Energy (PTPE).

4.2 Basic Assumptions

Before deriving the basic governing equations, the following assumptions are introduced:

1. Plane sections are considered to remain plane and perpendicular to the reference axis after deformation.
2. The beam components are assumed straight and geometric nonlinearities are neglected.
3. Shear connection stiffness is uniform and continuous along the interface.

4. The strengthening plates are assumed thin: thus, they carry only axial tensile force, and have no shear or flexural stiffness.

4.3 Beam Deformation and Internal Forces

The conventional beam theory can be used to model the concrete beam. The effects of the bond slips on the response of the beam under consideration can be accounted for by using an interface model. For the finite element model of a strengthened reinforced concrete beam, one has to assume the displacement field that governs the response of each beam component (i.e. RC beam and CFRP plate) and the interface.

In the externally strengthened reinforced concrete beam presented here, it is assumed that the beam behavior is dominated by the axial and flexural deformations of RC beam and the shear deformation of the interface as well as the axial deformation of the FRP plate. With these assumptions, the following element displacements, forces, and section deformation are used in the formulation.

4.3.1 Element Displacements

In the present formulation, the deformable body consists of two basic components, namely the main body and the interface. The main body is the principle load-carrying component and is divided into different subcomponents by the interface. Only one dimensional component is considered in this formulation. It is reasonable to assume that the transverse deflections of the all beam components are the same. However, different axial displacements may occur in the beam component sections and the following displacement components are considered:

$$\mathbf{u}(\mathbf{x}) = \{\mathbf{v}_B(\mathbf{x}) \quad \mathbf{u}_B(\mathbf{x}) \quad \mathbf{u}_{FRP}(\mathbf{x})\}^T \quad \dots (4.1)$$

in which $v_B(x)$ is the transverse deflection of the RC strengthened beam along the y-axis; $u_B(x)$ and $u_{FRP}(x)$ are the axial displacement of the RC beam and FRP plate, respectively. Having the element deflection and the axial displacements of the RC beam and FRP plate, the relative displacement between the concrete substrate and FRP plate along the interface is:

$$u_r(x) = y_{FRP} \frac{dv_B(x)}{dx} + u_B(x) - u_{FRP}(x) \quad \dots (4.2)$$

in which y_{FRP} is the distance between the reference axis of concrete beam and FRP plate. Because the two beam components have the same transverse deflections, no relative displacement normal to the interface can be expected. The kinematics of strengthened RC beam with bond interface is shown in Figure 4.1.

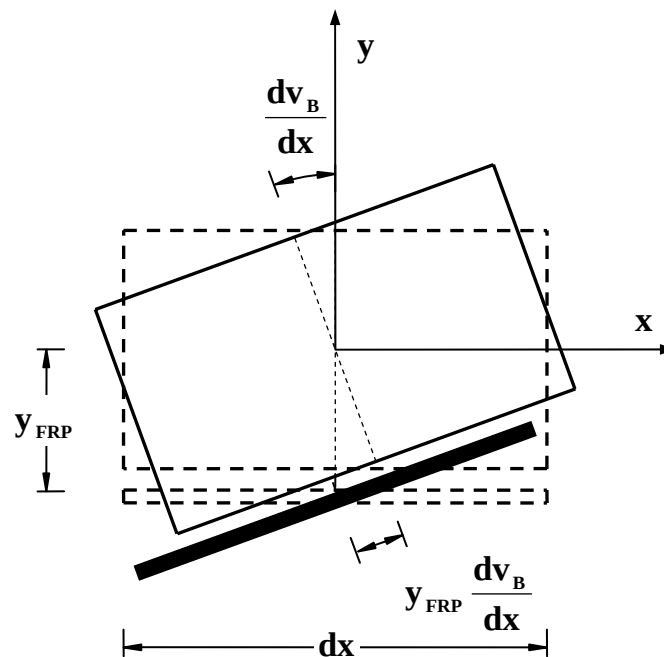


Figure 4.1: Kinimatics of RC beam strengthened with FRP plates

4.3.2 Element Forces

By ignoring the shear deformations of the beam components, only the following internal forces need to be considered for the beam section:

$$D_B(x) = \{M_B(x) \quad N_B(x) \quad N_{FRP}(x)\}^T \quad \dots (4.3)$$

in which $M_B(x)$ is the beam section bending moment about z-axis; $N_B(x)$ and $N_{FRP}(x)$ are the axial forces of the concrete beam and FRP plate, respectively, at their reference axes.

In addition, by ignoring the relative displacement normal to the interface, only the tangential component of the bond interface force $D_b(x)$ needs to be considered. The element forces are shown in Figure 4.2.

4.3.3 Section Deformations

The axial strain in the concrete beam and FRP plate are independent. The section deformations corresponding to the section forces in Equation 4.3 are as follows:

$$d_B(x) = \{\kappa_B(x) \quad \varepsilon_B(x) \quad \varepsilon_{FRP}(x)\}^T \quad \dots (4.4)$$

in which $\kappa_B(x)$ is the curvature of strengthened RC beam; and $\varepsilon_B(x)$, $\varepsilon_{FRP}(x)$ are the axial strains at the reference axes of the concrete beam and FRP plate, respectively. The other component of section deformation that must be considered is the bond slip $d_b(x)$ along the interface, which is associated with the bond force $D_b(x)$.

4.4 Basic Governing Equations (Strong Form)

The behavior of a deformable body must satisfy three basic conditions. These conditions are equilibrium, compatibility, and material constitutive laws (Salari et al 1998). For the strengthened RC beam element under consideration, these conditions are given in the following subsections.

4.4.1 Equilibrium Equations

To derive the differential equations of equilibrium for a strengthened reinforced concrete beam, the infinitesimal beam segment shown in Figure 4.2 is considered. Based on the small-deformation assumption, all of the equilibrium conditions are considered in the undeformed configuration.

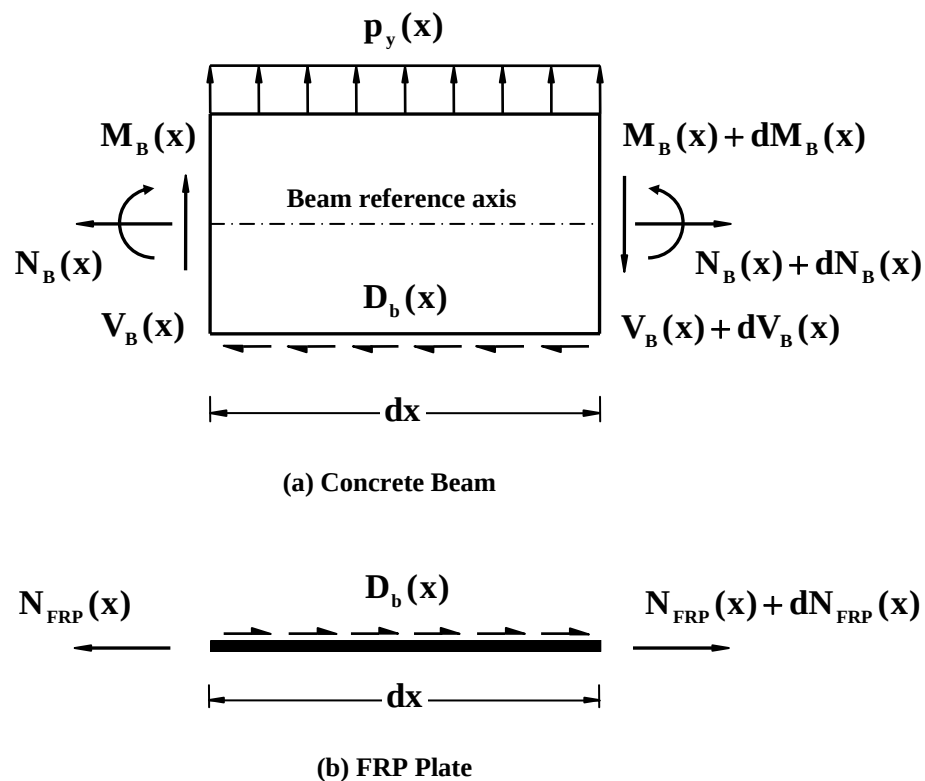


Figure 4.2: Infinitesimal segment of strengthened RC beam

Equilibrium of the segment in the y-direction, which includes the RC beam component, leads to the following expressions:

$$\frac{dV_B(x)}{dx} - p_y(x) = 0 \quad \dots (4.5)$$

in which $V_B(x)$ is the shear force in RC beam section along the y-axis; $p_y(x)$ denotes the distributed load on the beam segment. The moment equilibrium for RC infinitesimal segment about the z-axis results in the following relation:

$$\frac{dM_B(x)}{dx} - V_B(x) - y_{FRP} D_b(x) = 0 \quad \dots (4.6)$$

This work follows the Euler-Bernoulli beam theory, thus the shear deformations of the beam section are neglected. Therefore, the shear force $V_B(x)$ should be eliminated from the equilibrium equations. This can be done by taking derivative of Equation 4.6 with respect to x and combining with Equation 4.5. The following relations can be obtained:

$$\frac{d^2 M_B(x)}{dx^2} - p_y(x) - y_{FRP} \frac{dD_b(x)}{dx} = 0 \quad \dots (4.7)$$

Finally, axial equilibriums in the RC beam component and the FRP plate lead to the following equations:

$$\frac{dN_B(x)}{dx} - D_b(x) = 0 \quad \dots (4.8)$$

$$\frac{dN_{FRP}(x)}{dx} + D_b(x) = 0 \quad \dots (4.9)$$

Equations 4.7 to 4.9 which represent the governing equilibrium equations of strengthened RC beam with bond interface can be written in the following matrix form:

$$L_B^T D_B(x) - L_b^T D_b(x) - p(x) = 0 \quad \dots (4.10)$$

in which the element force vector $p(x) = \{p_y(x) \ 0 \ 0\}^T$, and L_B and L_b are beam section and bond differential operators, respectively, and may be given :

$$L_B = \begin{bmatrix} \frac{d^2}{dx^2} & 0 & 0 \\ 0 & \frac{d}{dx} & 0 \\ 0 & 0 & \frac{d}{dx} \end{bmatrix} \quad \dots (4.11a)$$

$$L_b = \begin{bmatrix} y_{FRP} \frac{d}{dx} & 1 & -1 \end{bmatrix} \quad \dots (4.11b)$$

It can be observed from the above equations that any section of the strengthened RC beam contains four unknown internal forces: M_B , N_B , N_{FRP} and D_b . On the other hand, only three independent equilibrium equations are available at each section. Therefore, the strengthened RC beam is internally indeterminate and its internal forces cannot be obtained through equilibrium alone.

4.4.2 Compatibility Equations

The flexural and axial deformations at any section of strengthened RC beam are related to the beam element displacements through compatibility conditions. Under small displacements, the compatibility conditions for the RC beam and FRP plate are as follows:

$$\kappa_B(x) = \frac{d^2 v_B(x)}{dx^2} \quad \dots (4.12a)$$

$$\varepsilon_B(x) = \frac{du_B(x)}{dx} \quad \dots (4.12b)$$

$$\varepsilon_{FRP}(x) = \frac{du_{FRP}(x)}{dx} \quad \dots (4.12c)$$

Furthermore, compatibility must hold between the interface deformations $d_b(x)$ and the relative displacements between the beam section components along the interface. Using Equations 4.2, this compatibility condition is:

$$d_b(x) = y_{FRP} \frac{dv_B(x)}{dx} + u_B(x) - u_{FRP}(x) \quad \dots (4.13)$$

Equations 4.12 and 4.13, which represent the compatibility conditions for the strengthened RC beam under consideration, can be written in the following matrix forms:

$$L_B u(x) - d_B(x) = 0 \quad \dots (4.14a)$$

$$L_b u(x) - d_b(x) = 0 \quad \dots (4.14b)$$

where $u(x)$ is the displacement vector defined in Equation 4.1 ; and L_B , and L_b are differential operators given in Equations 4.11a and 4.11b, respectively ; $d_B(x)$ is the section deformation vector , and $d_b(x)$ is the bond slip are those define in Section 4.3.3.

4.4.3 Force-Deformation Relations

The internal forces for the main body $D_B(x)$ are related to the deformation measures $d_b(x)$ by nonlinear constitutive relation, $\hat{g}_B$ as follows:

$$D_B(x) = \hat{g}_B \cdot (d_B(x)) \quad \dots (4.15)$$

In this study, the nonlinear relation in Equation 4.15 is derived from a layered discretization of the cross section of the strengthened RC beam with nonlinear, uniaxial stress-strain relations for the constituent materials. Such a model directly accounts for the interaction between axial force and bending moment. The relation between bond force and bond slip at the interface between beam components can be introduced through the nonlinear constitutive relation, $\hat{g}_b$ as follows:

$$D_b(x) = \hat{g}_b \cdot (d_b(x)) \quad \dots (4.16)$$

The material and bond constitutive laws depend on the particular element at hand and are discussed in details later in Section 4.6.

The equilibrium, compatibility, and constitutive equations for the strengthened RC beam presented above are conveniently represented in

the classical Tonti's diagram of Figure 4.3. This diagram is going to be modified for the element formulation presented in this study.

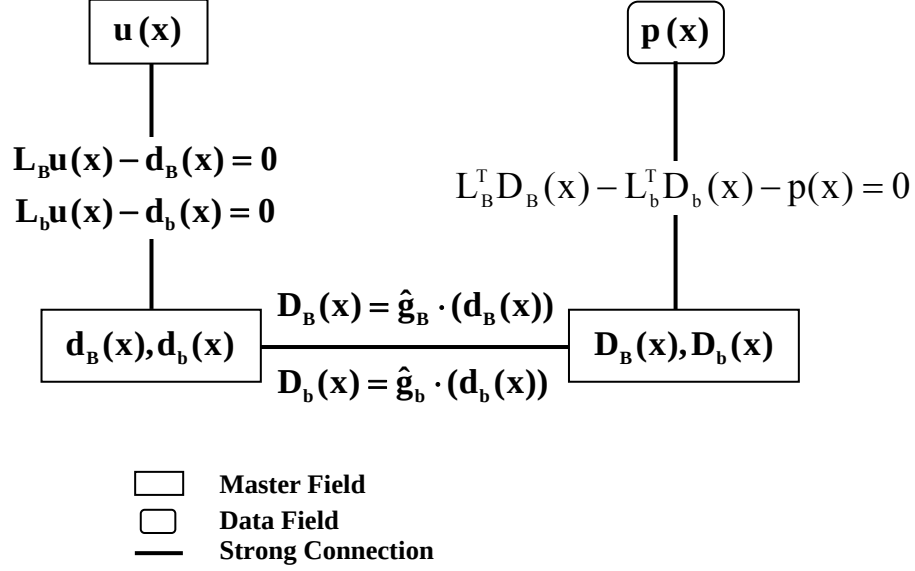


Figure 4.3: Tonti's diagram strong form formulation

4.5 Finite Element Formulation (Weak Form)

The basic governing equations (strong form) previously derived represent the equilibrium, compatibility conditions, and the material constitutive laws for a strengthened RC beam. These equations, along with appropriate static and kinematic boundary conditions, may govern the response of strengthened RC beam to applied loads. Exact solution of all field equations is impossible to obtain. Therefore, variational formulations (i.e. a weak form), along with discretization methods are used in order to solve some of the field equations and satisfy some of the boundary conditions only approximately.

However, different combinations of these equations may lead to different solution schemes. Starting from the equilibrium equations, with displacement fields as principle unknowns, one can obtain a

displacement-based finite element formulation (see Figure 4.4). The displacement-based formulation for RC beam element with bond slip under uniaxial bending was originally presented in Limkatanyu and Spacone (2002). This formulation is extending here to include more attractive problem of RC beam strengthened with FRP plates under monotonic loading.

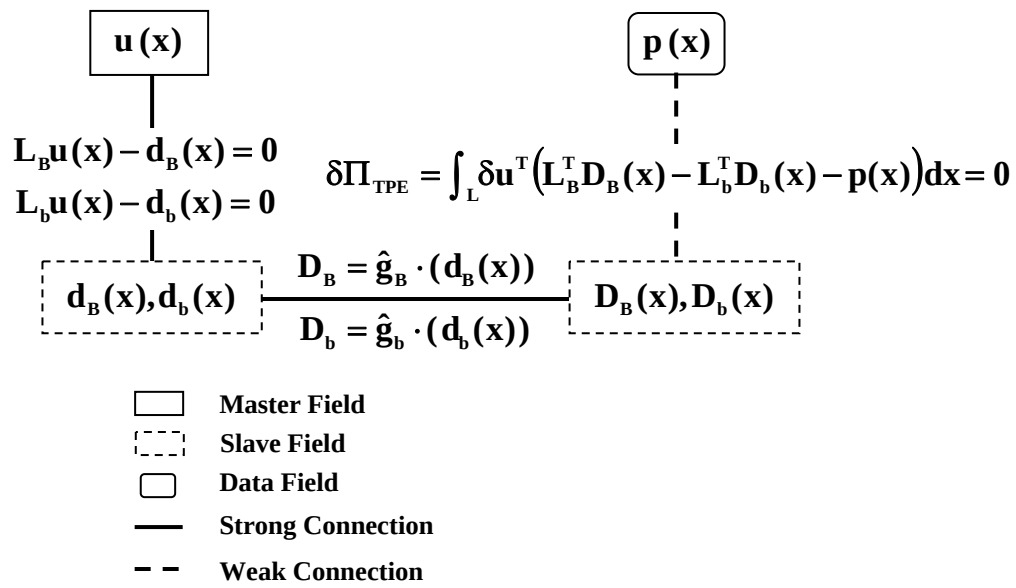


Figure 4.4: Tonti's diagram for weak formulation

4.5.1 Total Potential Energy (TPE) Functional

Following Limkatanyu and Spacone (2002), the equilibrium equation at the current state of the element is cast in weighted integral form by invoking the principle of Total Potential Energy Functional:

$$\Pi_{PE}[\mathbf{u}(\mathbf{x})] = \int_L \mathbf{u}^T(\mathbf{x}) [\mathbf{L}_B^T \mathbf{D}_B(\mathbf{x}) - \mathbf{L}_b^T \mathbf{D}_b(\mathbf{x}) - \mathbf{p}(\mathbf{x})] d\mathbf{x} \quad \dots (4.17)$$

Equilibrium is found by imposing that the first variation of the total potential energy functional Π_{PE} be equal to zero:

$$\delta \Pi_{PE} [\delta u(x)] = \int_L \delta u^T(x) [L_B^T D_B(x) - L_b^T D_b(x) - p(x)] dx = 0 \quad \dots (4.18)$$

where $\delta u(x)$ is a compatible virtual displacement field that serves as the weight function and the integration extends over the element length. For the sake of simplicity, the effect of element loads is not pursued further, and $p(x)$ is set equal to zero in Equation 4.18.

To satisfy the materials and bond constitutive laws, the section forces are written in the incremental forms, then the force-deformation relation are linearized as follows (Limkatanyu and Spacone 2002):

$$D_B(x) = D_B^0 + \Delta D_B(x) = D_B^0(x) + k_B^0(x) \Delta d_B(x) \quad \dots (4.19a)$$

$$D_b(x) = D_b^0 + \Delta D_b(x) = D_b^0(x) + k_b^0(x) \Delta d_b(x) \quad \dots (4.19b)$$

where $D_B^0(x)$ and $D_b^0(x)$ are the section force vectors at the initial point for the main body and bond interface, respectively; and $k_B^0(x)$ and $k_b^0(x)$ are the corresponding stiffness matrices. Upon substitution of the section linearized constitutive laws Equations 4.18, 4.19a, and 4.19b are written:

$$\int_L \delta u^T(x) \{ L_B^T [D_B^0(x) + k_B^0(x) \Delta d_B(x)] - L_b^T [D_b^0(x) + k_b^0(x) \Delta d_b(x)] \} dx = 0 \quad \dots (4.20)$$

The compatibility conditions are imposed by enforcing Equations 4.14a and 4.14b. Then, Equation 4.20 becomes:

$$\int_L \delta u^T(x) \left\{ L_B^T [D_B^0(x) + k_B^0(x) L_B \Delta u(x)] - L_b^T [D_b^0(x) + k_b^0(x) L_b \Delta u(x)] \right\} dx = 0 \quad \dots (4.21)$$

Rewriting the integral of Equation 4.21 in terms of the components of the internal forces and displacements and then performing integration by parts. The integration by parts is used to shift the differential operators L_B and L_b from the force fields $D_B(x)$ and $D_b(x)$ to the virtual displacement field $\delta u(x)$ (Limkatanyu and Spacone 2002). A single integration by parts is necessary for the axial force equations, and two successive integrations are required for the moment equilibrium equation in Equation 4.21. After changing sign for all terms of the last three equations, Equation 4.21 takes the form:

$$\int_L \left\{ \begin{matrix} L_B [\delta u(x)] \\ L_b [\delta u(x)] \end{matrix} \right\}^T \begin{bmatrix} k_B^0(x) & 0 \\ 0 & k_b^0(x) \end{bmatrix} \left\{ \begin{matrix} L_B [\Delta u(x)] \\ L_b [\Delta u(x)] \end{matrix} \right\} dx = \delta U^T P - \int_L L_B [\delta u^T(x)] D_B^0(x) - \int_L L_b [\delta u^T(x)] D_b^{i-1}(x) dx \quad \dots (4.22)$$

where $\delta U^T P$ derives from the boundary terms and represents the external virtual work done by the virtual nodal displacements δU on the externally applied nodal forces P .

Equation 4.22, which satisfies the equilibrium conditions in integral form and the compatibility constitutive laws pointwise, is the fundamental equation for the displacement-based finite element formulation of strengthened RC beam element with bond interface.

4.5.2 Finite Element Discretization

The two-node displacement-based strengthened reinforced concrete beam model with bond-interface is presented here. The element consists of a two-node beam element, plus two-node axial member, which allowed slipping with respect to the beam. The nodal degrees of freedom of the beam and the axial member are different to permit axial member slip. It was assume separate displacement fields in the reinforced concrete beam and the axial member, and bond-slip is automatically derived from compatibility as shown in Figure 4.5.

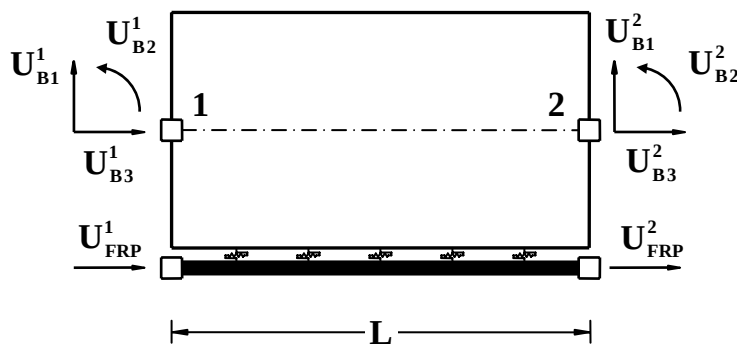


Figure 4.5: Displacement degrees of freedom of strengthened RC beam

To discretize Equation 4.22 in terms of the nodal quantities, the element displacements $u(x)$ are approximated in terms of the nodal displacements U using displacement shape functions $N_d(x)$ as follows:

$$u(x) = N_d(x) \cdot U \quad \dots (4.23)$$

in which

$$U = \{U_{B1}^1 \quad U_{B2}^1 \quad U_{B3}^1 \quad U_{FRP}^1 \quad U_{B1}^2 \quad U_{B2}^2 \quad U_{B3}^2 \quad U_{FRP}^2\}^T \quad \dots (4.24)$$

and $N_d(x)$ is a matrix that contains the cubic interpolation functions for the transverse displacement and the linear interpolation functions for the axial displacements:

$$N_d(x) = \begin{bmatrix} \phi_1(x) & \phi_2(x) & 0 & 0 & \phi_3(x) & \phi_4(x) & 0 & 0 \\ 0 & 0 & \varphi_1(x) & 0 & 0 & 0 & \varphi_2(x) & 0 \\ 0 & 0 & 0 & \varphi_1(x) & 0 & 0 & 0 & \varphi_2(x) \end{bmatrix} \dots (4.25)$$

with

$$\begin{aligned} \phi_1(x) &= \frac{2}{L^3}x^3 - \frac{3}{L^2}x^2 + 1 & \phi_2(x) &= -\frac{2}{L^3}x^3 + \frac{3}{L^2}x^2 \\ \phi_3(x) &= \frac{1}{L^2}x^3 - \frac{2}{L}x^2 + x & \phi_4(x) &= \frac{1}{L^2}x^3 - \frac{1}{L}x^2 \\ \varphi_1(x) &= 1 - \frac{x}{L} & \varphi_2(x) &= \frac{x}{L} \end{aligned}$$

Upon substitution of Equation 4.23 into Equation 4.22 and elimination of δU from the arbitrariness considerations

$$\begin{aligned} \int_L \left\{ \begin{Bmatrix} B_B(x) \\ B_b(x) \end{Bmatrix} \right\}^T \begin{bmatrix} k_B^0(x) & 0 \\ 0 & k_b^0(x) \end{bmatrix} \left\{ \begin{Bmatrix} B_B(x) \\ B_b(x) \end{Bmatrix} \right\} dx (\Delta U) &= P - \int_L B_B^T(x) D_b^0(x) dx \\ &- \int_L B_b^T(x) D_b^0(x) dx \end{aligned} \dots (4.26)$$

where $B_B(x) = L_B N_d(x)$ and $B_b(x) = L_b N_d(x)$. Equation 4.26, which represents the discretized form of the governing equations, can be written in compact form as:

$$K^0 \Delta U = P - Q^0 \dots (4.27)$$

where $K^0 = K_B^0 + K_b^0$ is the element stiffness matrix. K_B^0 is the main body contribution and K_b^0 is the bond contribution, defined as follows:

$$K_B^0 = \int_L B_B^T(x) k_B^0(x) B_B(x) dx \quad \dots (4.28a)$$

$$K_b^0 = \int_L B_b^T(x) k_b^0(x) B_b(x) dx \quad \dots (4.28.b)$$

and $Q^0 = Q_B^0 + Q_b^0$ are the element forces, where Q_B^0 is the main body contribution and Q_b^{i-1} is the bond contribution, defined as:

$$Q_B^0 = \int_L B_B^T(x) D_B^0(x) dx \quad \dots (4.29.a)$$

$$Q_b^0 = \int_L B_b^T(x) D_b^0(x) dx \quad \dots (4.29.b)$$

The right-hand side of Equation 4.27 is the force residual corresponding to the weak statement of the equilibrium Equation 4.10 and vanishes when the equilibrium configuration is reached.

4.6 Materials Idealization

The fiber beam element is shown in Figure 4.6 in the local reference x,y it is divided into a discrete number of cross sections . These are located at the control points of the numerical integration scheme used in the element formulation. In this study the Gauss-Lobatto integration scheme is used, since it allows for two integration points to coincide with end sections of the element, where significant inelastic deformations typically take place (Taucer et al., 1991). More details on the integration

scheme are provided in Chapter Five. Each section is subdivided into n layers. The section forces and corresponding deformations are defined in Section 3.2 and these grouped in the vectors given in Equations 4.3 and 4.4, respectively. Two more vectors are introduced to describe the state of the layers at each section. These contain the strain and stresses of the fiber and are written in the following form:

$$\varepsilon(x) = \{\varepsilon_1(x, y_1) \quad \cdots \quad \varepsilon_{n_{lyr}}(x, y_{n_{lyr}}) \quad \varepsilon_{FRP}(x, y_{FRP})\}^T \quad \dots (4.30)$$

$$\sigma(x) = \{\sigma_1(x, y_1) \quad \cdots \quad \sigma_{n_{lyr}}(x, y_{n_{lyr}}) \quad \sigma_{FRP}(x, y_{FRP})\}^T \quad \dots (4.31)$$

In the layer state vector x describe the position of the section along the longitudinal reference axis and y refer to the layer position in the cross section, as shown in Figure 4.6.

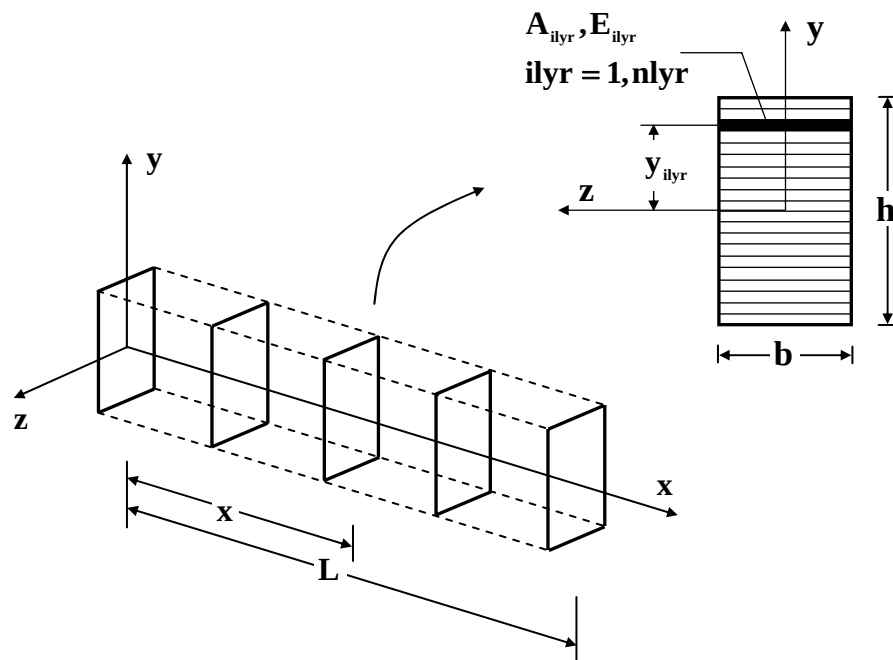


Figure 4.6: Subdivision of cross section into layers

Following the hypothesis that plane section remains plane and normal to the longitudinal axis, the layer strain vector and the section deformation vector are related by simple matrix relation:

$$\varepsilon(x) = I(x) \cdot d_B(x) \quad \dots (4.32)$$

where $I(x)$ is a linear geometric matrix as follow:

$$I(x) = \begin{bmatrix} -y_1 & 1 & 0 \\ \vdots & \vdots & \vdots \\ -y_{nlyr} & 1 & 0 \\ 0 & 0 & 1 \end{bmatrix} \quad \dots (4.33)$$

4.6.1 Fiber Constitutive Models

The nonlinear behavior of the strengthened RC beam element presented here derives entirely from the nonlinear behavior of the constituent materials and the bond-slip at the interface between the CFRP plate and concrete substrate. Thus, the validity of the analytical results depends on the accuracy of the constituent material and bond-slip models. Since the present study deals with the responses of RC beams strengthened with FRPs and the effect of the debonding of FRP is considered, three material model are required (for concrete, steel and FRP). Further, FRP-concrete bond-slip model is also required.

The element formulation simplifies the task of material model selection to uniaxial behavior, which thoroughly studied and well established to date. It is important to note that all stress-strain models must be explicit functions of strain. This is a significant feature of the material and bond models in connection with the adopted fiber model, where fiber strains are determined from section deformations.

4.6.1.1 Steel Stress-Strain Relation

A bilinear elasto-plastic model with a strain hardening part is used for the steel reinforcing bar as shown in Figure 4.7. The response in tension and compression is assumed to be identical. The behavior in elastic stage is:

$$f_s = E_s \varepsilon_s \quad \dots (4.34)$$

When steel strain exceeds the yield strain ε_y , the stress is obtained using the relationship (Hasan ,2006):

$$f_s = f_y + H_s E_s (\varepsilon_s - \varepsilon_y) \quad \dots (4.35)$$

where; f_s, ε_s, E_s , and H_s is steel stress, strain, elastic modulus, and strain hardening parameter, respectively; and f_y, ε_y are yield steel stress and strain, respectively.

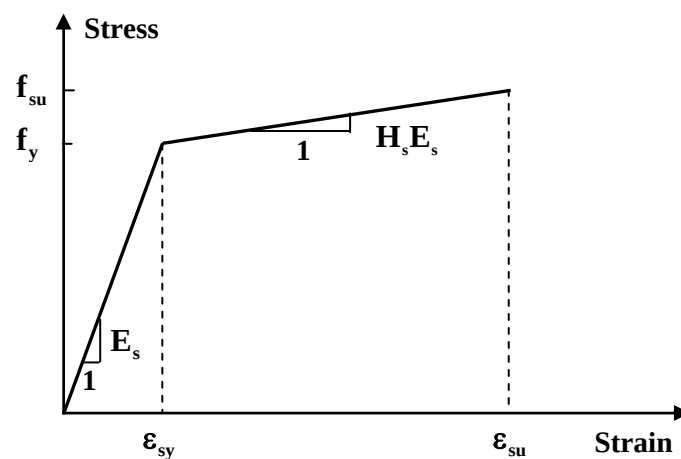


Figure 4.7: Idealized uniaxial stress-strain relation for reinforcing bar
(Hasan ,2006)

4.6.1.2 Concrete Stress-Strain Relation

In order to compute the concrete stress in each layer, a material law describing the concrete stress-strain relation is needed.

▪ *Concrete in Compression*

The uniaxial stress-strain relation is assumed to be linear up to a stress level equal to $c_p f'_c$ followed by a parabolic shape up to the compressive strength f'_c (Naji, 1989),

$$f_c = E \varepsilon_c \quad \text{for} \quad 0 \leq f_c \leq c_p f'_c \quad \dots (4.36)$$

$$f_c = c_p f'_c + E_c \left[\varepsilon_c - \frac{c_p f'_c}{E_c} \right] - \frac{1}{2} \frac{E_c}{\varepsilon_0} \left[\left(\varepsilon_c - \frac{c_p f'_c}{E_c} \right)^2 \right] \quad \text{for} \quad c_p f'_c \leq f_c \leq f'_c \quad \dots (4.37)$$

where E_c is the initial modulus of elasticity and ε_0 is the concrete strain corresponding to the peak stress.

Figure 4.8 shows the idealized uniaxial compressive stress-strain response curve that is adopted to represent the behavior of concrete in compression. It is simulated by an elastic-plastic work hardening response followed by a perfectly plastic plateau, which is terminated at the onset of crushing. Even though more accurate and complete models have been published (Taucer, 1991), this model offers a good balance between simplicity and accuracy (Naji, 1989).

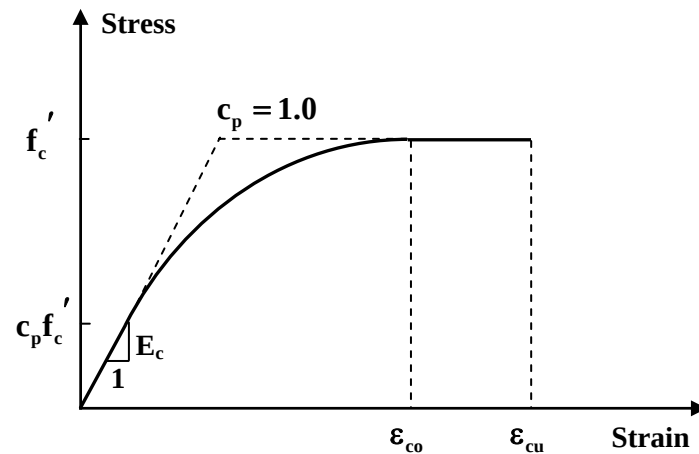


Figure 4.8: Idealized uniaxial stress-strain relation for concrete
(Naji,1989)

▪ *Concrete in Tension*

Concrete is assumed to crack when it reaches its tensile strength calculated according to the ACI-318 Code (2005). After concrete cracks; tension stiffening occurs in concrete reinforced with steel bars or CFRP. Tension stiffening accounts for load transfer mechanisms that exist between reinforcement (steel bars or CFRP fabric) and the surrounding concrete and is generally represented by gradual degradation in the concrete tensile strength after cracking. It is reasonable to assume that CFRP will generate a greater concrete tension stiffening effect compared to steel bars because it is directly attached to a large concrete surface area (El-Tawil and Okeil, 2001). Based on a calibration to beam test results by Shahawy and Beitelman (1999) tension stiffening due to steel bar reinforcement alone is assumed to decrease linearly from 70 percent of the cracking stress to zero at five times the concrete cracking strain. Tension stiffening due to presence of both steel and CFRP combined is assumed to degrade linearly from 70 percent of cracking stress to zero at 20 times the concrete cracking strain as shown in Figure 4.9. This model

was used firstly by El-Tawil and Okeil (2001) then it was adopted by Hasan (2007) and may be given as:

$$f_{ct} = E_{co} \varepsilon_c \quad \text{for} \quad \varepsilon_c < \varepsilon_t \quad \dots (4.38)$$

$$f_{ct} = \alpha_2 f_t \frac{[\alpha_1 - \varepsilon_c / \varepsilon_t]}{\alpha - 1.0} \quad \text{for} \quad \varepsilon_t \leq \varepsilon_c \leq \alpha_1 \varepsilon_t \quad \dots (4.39)$$

$$f_{ct} = 0 \quad \text{for} \quad \varepsilon_c > \alpha \varepsilon_t \quad \dots (4.40)$$

where; E_{co} is the elastic modulus of concrete,

α_1, α_2 are the tension-stiffening parameters, where,

α_1 represents the rate of stress release as the crack widens, (its value is 5 for steel bars only, and 20 for steel bars and CFRP laminate),

α_2 represents the stress at the instance of cracking ($\alpha_2=0.7$),

f_t is the concrete cracking (tensile) stress .

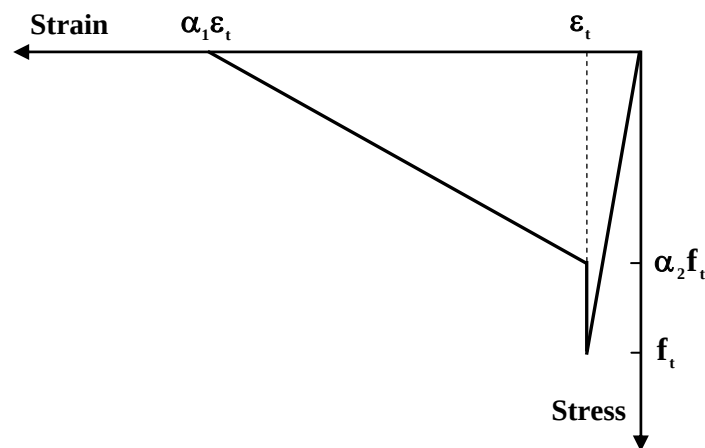


Figure 4.9: Concrete tension –stiffening model (El-Tawil and Okeil , 2001)

4.6.1.3 CFRP Stress-Strain Relation

When loaded in tension, CFRP fibers do not exhibited any plastic behavior (yielding) before rupture as shown in Figure 4.10. The tensile behavior is characterized by a linearly elastic stress-strain relationship up to failure, which is sudden and can be catastrophic. Elastic behavior followed by sudden failure is assumed to occur during loading and may be expressed as:

$$f_{FRP} = E_{FRP} \epsilon_{FRP} \quad \dots (4.41)$$

where f_{FRP} , ϵ_{FRP} and E_{FRP} are tensile stress, strain and Young modulus for FRP laminate, respectively.

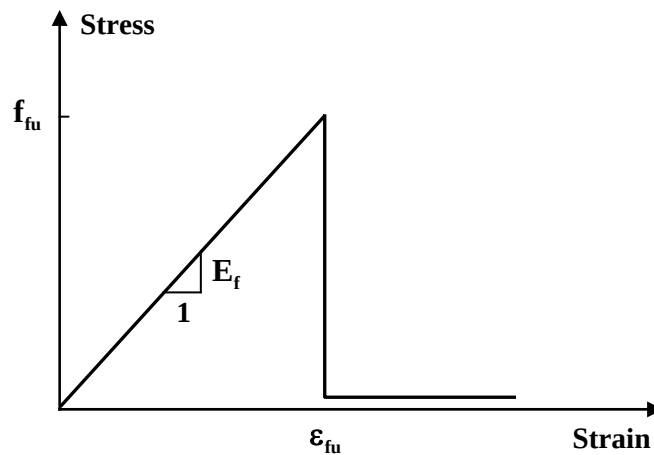


Figure 4.10: Idealized uniaxial stress-strain relation for CFRP laminate
(El-Tawil and Okeil , 2001)

4.6.2 Section Stiffness Matrix

Similar to the element state determination, the section state determination involves the calculation of resisting forces $D_{BR}(x)$ and section stiffness matrix $k_B(x)$ that correspond to deformation $d_B(x)$. The geometric matrix $I(x)$ in Equation 4.33 is first used to obtain the layer strain increments for the given deformation increments $\Delta d(x)$ following the hypothesis that plane section remains plane and normal to the longitudinal axis:

$$\Delta \varepsilon(x) = I(x) \cdot \Delta d(x) \quad \dots (4.42)$$

$$\varepsilon(x) = \varepsilon(x) + \Delta \varepsilon(x) \quad \dots (4.43)$$

The new stress σ_{ilyr} and the tangent modulus E_{ilyr} of all layers are determined from the appropriate uniaxial stress-strain relation: the fiber stresses are then grouped in vector $E(x)$ and the tangent moduli are written in diagonal matrix $E_{B,tan}(x)$. By calling $A_B(x)$ a diagonal matrix with entries the areas A_{ilyr} of all layers and using, the section compatibility matrix $I_B(x)$ of Equation 4.32 well established structural analysis concepts yield the new section tangent stiffness matrix:

$$k_B(x) = I_B^T(x) \cdot (E_{B,tan} A_B) \cdot I_B(x) \quad \dots (4.44)$$

which after carrying out the multiplications results in:

$$k_B(x) = \begin{bmatrix} \sum_{ilyr=1}^{nlyr} E_{ilyr} A_{ilyr} y_{ilyr}^2 & - \sum_{ilyr=1}^{nlyr} E_{ilyr} A_{ilyr} y_{ilyr} & 0 \\ - \sum_{ilyr=1}^{nlyr} E_{ilyr} A_{ilyr} y_{ilyr} & \sum_{ilyr=1}^{nlyr} E_{ilyr} A_{ilyr} & 0 \\ 0 & 0 & E_{FRP} A_{FRP} \end{bmatrix} \quad \dots (4.45)$$

in which E_{ilyr} and A_{ilyr} are the elastic moduli and area of layer i . Similarly; the section resisting forces $D_{BR}(x)$ cannot be obtained directly from the section force-deformation relation, but are determined by summation of the axial force and bending contribution of all fibers as follows:

$$D_{BR}(x) = I_B^T(x) \cdot A_B(x) \cdot \sigma(x) \quad \dots (4.46)$$

alternatively, after carrying out the multiplications:

$$D_{BR}(x) = \left\{ \sum_{ilyr=1}^{nlyr} \sigma_{ilyr} A_{ilyr} y_{ilyr} \quad \sum_{ilyr=1}^{nlyr} \sigma_{ilyr} A_{ilyr} \quad \sigma_{FRP} A_{FRP} \right\}^T \quad \dots (4.47)$$

4.6.3 Bond-Slip Relation

With the development of the technology of upgrading the existing concrete structures by using externally bonded FRP composites, a number of issues related to conventional structural behaviors of the concrete structures after being upgraded have been studied in the last decade. Among them is a keen interest to clarify the mechanism of the interface bond between FRP composites and concrete substrates, because the bonding interface is relatively weak in comparison with the neighboring materials in the whole upgraded system. In most of strengthening cases, the interface bond is critical in transferring stresses from the existing concrete structures to the externally bonded FRP composites.

Lu et al. (2005) proposed a set of three new bond-slip models of different level of sophistication. A unique feature of these three models is not based on axial strain measurements on the FRP plate; instead, they are based on the predictions of a meso-scale finite element model, with appropriate adjustment to match their predictions with the experimental results for a few key parameters. Through comparisons with the large test database, all three bond-slip models are shown to provide accurate predictions of both the bond strength and the strain distribution in the FRP plate. Lu et al. classified these models as:

- Precise model.
- Simplified model.
- Bilinear model.

It is decided here to adopt a combination of the last two models and may be given in Figure 4.11. The mixed bond-slip model consists of a curvilinear ascending branch given by:

$$s = s_{\max} \sqrt{\frac{d_{\text{FRP}}}{d_0}} \quad \text{if} \quad d_{\text{FRP}} \leq d_0 \quad \dots (4.48)$$

and a bilinear descending branch given by:

$$s = s_{\max} \frac{(d_f - d_{\text{FRP}})}{(d_f - d_0)} \quad \text{if} \quad d_0 \leq d_{\text{FRP}} \leq d_f \quad \dots (4.49a)$$

$$s = 0 \quad \text{if} \quad d_{\text{FRP}} \succ d_f \quad \dots (4.49b)$$

where

$$d_f = 2G_f / s_{\max} \quad \dots (4.50a)$$

$$s_{\max} = 1.50\beta_w f_t \quad \dots (4.50b)$$

$$d_o = 0.0195\beta_w f_t \quad \dots (4.50c)$$

$$G_f = 0.308\beta_w^2 \sqrt{f_t} \quad \dots (4.50d)$$

$$\beta_w = \sqrt{\frac{2.25 - b_f/b_c}{1.25 + b_f/b_c}} \quad \dots (4.50e)$$

in which, s is the local bond stress; $s_{\max}$ is the maximum local bond stress; d_{FRP} is the local slip; d_f is the local slip when bond stress reduces to zero; d_o is the local slip at maximum bond stress; G_f is the interfacial fracture energy; and β_w is the FRP-to-concrete width ratio factor.

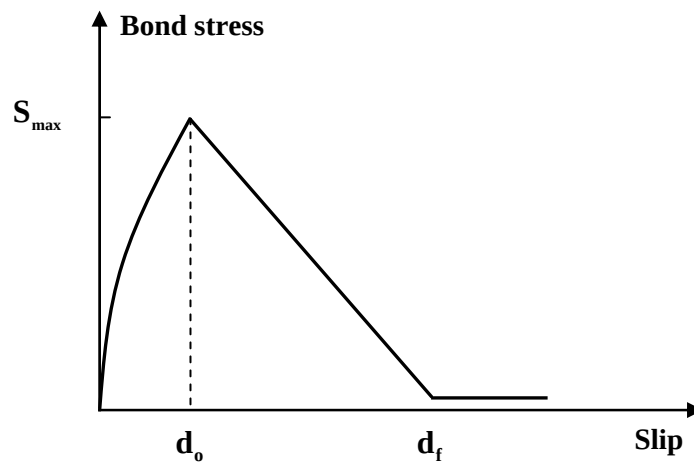


Figure 4.11: Idealized FRP-concrete bond-slip relation

CHAPTER FIVE

NONLINEAR SOLUTION TECHNIQUES AND COMPUTER PROGRAM

5.1 Introduction

The nonlinear response of most reinforced concrete structures can easily be verified from the examination of the load-displacement history. There can be several causes for such a response. These can be broadly divided into three categories:

- Large deformation in the structure, geometrical nonlinearity.
- Nonlinear material behavior, material nonlinearity.
- Contact nonlinearity (Bond-slip at steel-concrete interface and/or debonding at FRP-concrete interface).

In the present study, material and contact nonlinearities are considered.

The finite element method is a numerical technique in which continuous system with an infinite number of degrees of freedom is represented by an assemblage of discrete members, which has a finite number of degrees of freedom. The development in the finite element method depends on the proper element properties and on the availability of an efficient means of solving the resulting system of linear and nonlinear simultaneous equations.

On the other hand, the use of finite element discretization in nonlinear structural problems results in a nonlinear strain-dependent equation of the form,

$$[K]\{U\} = \{P\} \quad \dots (5.1)$$

in which,

- $[K]$ is the strain-dependent structure stiffness matrix ,
- $\{U\}$ is the unknown nodal displacements vector, and,
- $\{P\}$ is the external load vector.

The objective of a structural analysis is to determine the structural response under loading. This can be achieved relatively easily in linear problems. In nonlinear problems, however, more sophisticated solution strategies have to be employed. In general, the problem is reduced to the solution of a series of linear problems (Owen and Hinton ,1980). An incremental technique coupled with an iterative process until convergence is achieved is usually used in this context.

In this chapter, the nonlinear solution technique utilized in the finite element analysis of one dimensional reinforced concrete beams. The numerical integration, convergence criteria, and termination of the analysis are dealt with in section 5.3, 5.4 and 5.5, respectively. Finally, the computer program and its flow chart implemented for this purpose are well described.

5.2 Basic Techniques for Solving Nonlinear Equations

The finite element discretization of a nonlinear structure leads to a system of algebraic equations of the form,

$$\{R\}_i^k = \{P\}^k - \{F\}_i^k \quad \dots (5.2)$$

in which,

- $\{R\}$ is the out of balance or residual load vector ,
- $\{F\}$ is the internal resisting nodal load vector, and,

k, i is the number of load increment and iteration, respectively.

The satisfaction of equilibrium at the nodes requires that the external load vector equal the internal load vector, i.e. the out of balance force vector be zero,

$$\{R\} = 0 \quad \dots (5.3)$$

The solution of Equation 5.3 is usually attempted using one of the following three basic techniques,

- i. incremental technique,
- ii. iterative technique, or
- iii. incremental- iterative technique

These three techniques are shown for a single degree of freedom in Figure 5.1, and described briefly below,

- i. Incremental technique

The basic idea of this technique is to linearize Equation 5.1 by subdividing the load $\{P\}$ into several small increments. The behavior of the structure within each increment is assumed to be linear. Hence, a constant value of the global stiffness matrix is used at each load increment to obtain the displacement increment, as follows:

$$\{\Delta U\} = \{K\}^{-1} \{\Delta P\} \quad \dots (5.4)$$

The total displacement at any stage of loading is found by adding the displacements. The global stiffness matrix is modified at the

beginning at each load increment to account for the nonlinear effects generated during the preceding load increment as shown in Figure 5.1.a.

The incremental technique in its original form does not account for force redistribution during the application of the load increments. Therefore it suffers from a progressive and uncorrected tendency to drift from the true equilibrium path (Naji, 1989), see Figure 5.1.a.

ii. Iterative technique

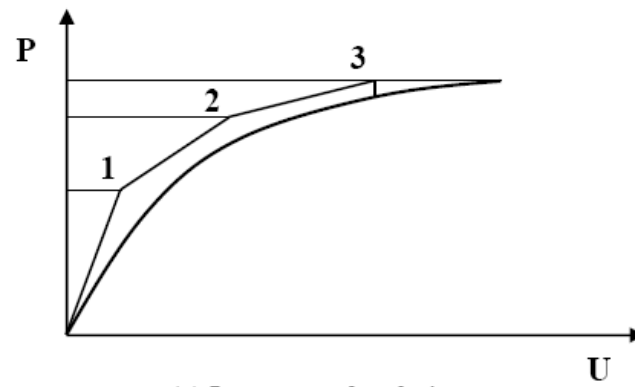
This technique is based on applying the whole load immediately to obtain an approximate solution. This solution is refined by successively applying the unbalanced or residual forces $\{R\}$, as shown in Figure 5.1.b. Different approaches are adopted for evaluating the global stiffness matrix, such as the constant or initial stiffness method and the tangent stiffness method in which the global stiffness is updated at the beginning of each iteration.

For problems where the entire response of the structure is required, purely, iterative technique cannot be used because they fail to produce information about the intermediate stages.

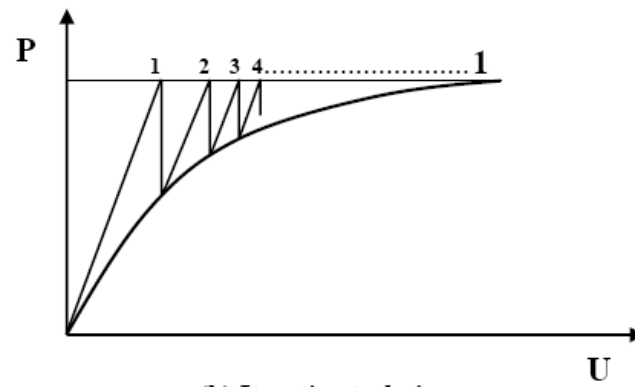
iii. Incremental-iterative technique

In this technique, the external loads are applied in increments with iterations performed within each increment until equilibrium is satisfied, i.e. Equation 5.3, according to some selected convergence criteria. The stiffness matrix is recalculated for the first iteration of each load increment, as shown in Figure 5.1.c. This mixed technique is widely used in the analysis of the reinforced concrete structure for capability to provide information throughout the loading history (Naji, 1989).

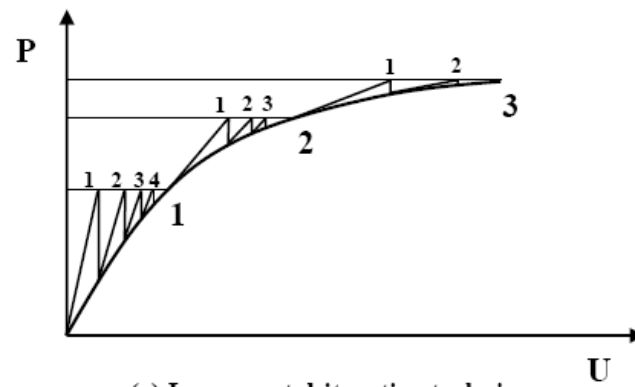
It was decided here to adopt the incremental-iterative technique in the finite element analysis developed in this study.



(a) Incremental technique



(b) Iterative technique



(c) Incremental-iterative technique

Figure 5.1: General techniques for the solution of nonlinear problems (Naji, 1989)

5.3 Numerical Integration

All integration in the element formulation are evaluated numerically with the Gauss-Lobato integration scheme that based on the expression:

$$I = \int_0^L f(x) dx = \sum_{h=1}^m w_h f(x_h) \quad \dots (5.5)$$

where h denotes the monitored section and w_h is the corresponding weight factor .

The Gauss-Lobatto scheme with m integration pointes permits the exact integration of polynomials of degree up to $(2m - 3)$. This procedure is superior to the classical Gauss integration method, when it is important to include in the integration the end points of the element (Fabio et al, 1991). Since inelastic behavior in beam element often concentrates at the ends of the member, the monitoring of the end sections of the element offers advantages from the standpoint of accuracy and numerical stability. Table 5.1 lists the values of ξ_h and w_h for different values m of the number of integration points.

Table 5.1: Integration points and weights for Gauss-Lobatto scheme
(Monti et al., 1993)

m	ξ_h	w_h
2	± 1.0000000000	1.0000000000
3	± 1.0000000000 0.0000000000	0.3333333333 1.3333333333
4	± 1.0000000000 ± 0.4472135955	0.1666666666 0.8333333333
5	± 1.0000000000 ± 0.6546536707 0.0000000000	0.1000000000 0.5444444444 0.7111111111

5.4 Convergence Criteria

The nonlinear solution algorithm in this study is based on a set of linear matrix relations that are repeatedly applied until convergence is achieved. Theoretically, this occurs when the applied loads are perfectly balanced by the internal resisting forces of the elements. Numerically, however, such perfect balance is either impossible or too expensive to attain. In a nonlinear solution algorithm it is normally accepted that convergence is reached when some control parameters, such as the unbalanced forces, are smaller than a specified threshold or tolerance. Norms or absolute values of these variables may be selected as control parameters as many studies to date have discussed (Fabio et al, 1991). Here, a norm of the unbalanced load criterion is used as a convergence criterion, as follows (Owen and Hinton, 1980):

$$\frac{\sqrt{\sum_{i=1}^n (R)^2}}{\sqrt{\sum_{i=1}^n (P)^2}} \times 100 \leq \text{Tolerance} \quad \dots (5.6)$$

where n is the total number of nodal points. This criterion states that convergence occurs if the norm of the residual forces becomes less than Tolerance times the norm of the total applied forces. A tolerance of 1.0 percent is taken throughout this study. In addition to the convergence tolerance, a ceiling is provided to limit the number of iteration performed for each load step in case convergence tolerance provided is too stringent.

5.5 Termination of the Analysis

The finite element analysis is terminated whenever

- i. the number of iterations exceeds a prescribed number,

- ii. the stiffness matrix is no longer positive definite,
- iii. steel reinforcing bar or CFRP plate has fractured, or,
- iv. extensive CFRP debonding takes place.

5.6 Brief Summary of the Computer Program

As stated in Chapter One, the main objective of the present work is to develop a realistic tool for the nonlinear analysis of reinforced concrete beam strengthened with CFRP composites. A computer program NBSRCB (Nonlinear Behavior of Strengthened Reinforced Concrete Beams) has been developed to trace the behavior characteristics of FRP strengthened reinforced concrete beam including the FRP debonding effects. The program, whose flow charts are shown in Figures 5.1 and 5.2, is coded in FORTRAN language and consists of a main program in addition to twenty one subroutines. The input data contain information concerning the geometry and material properties of the beam element as well as information about the applied loads.

Several numerical examples including comparison with available experimental results are presented in Chapter Seven to demonstrate the applicability of the developed computer program.

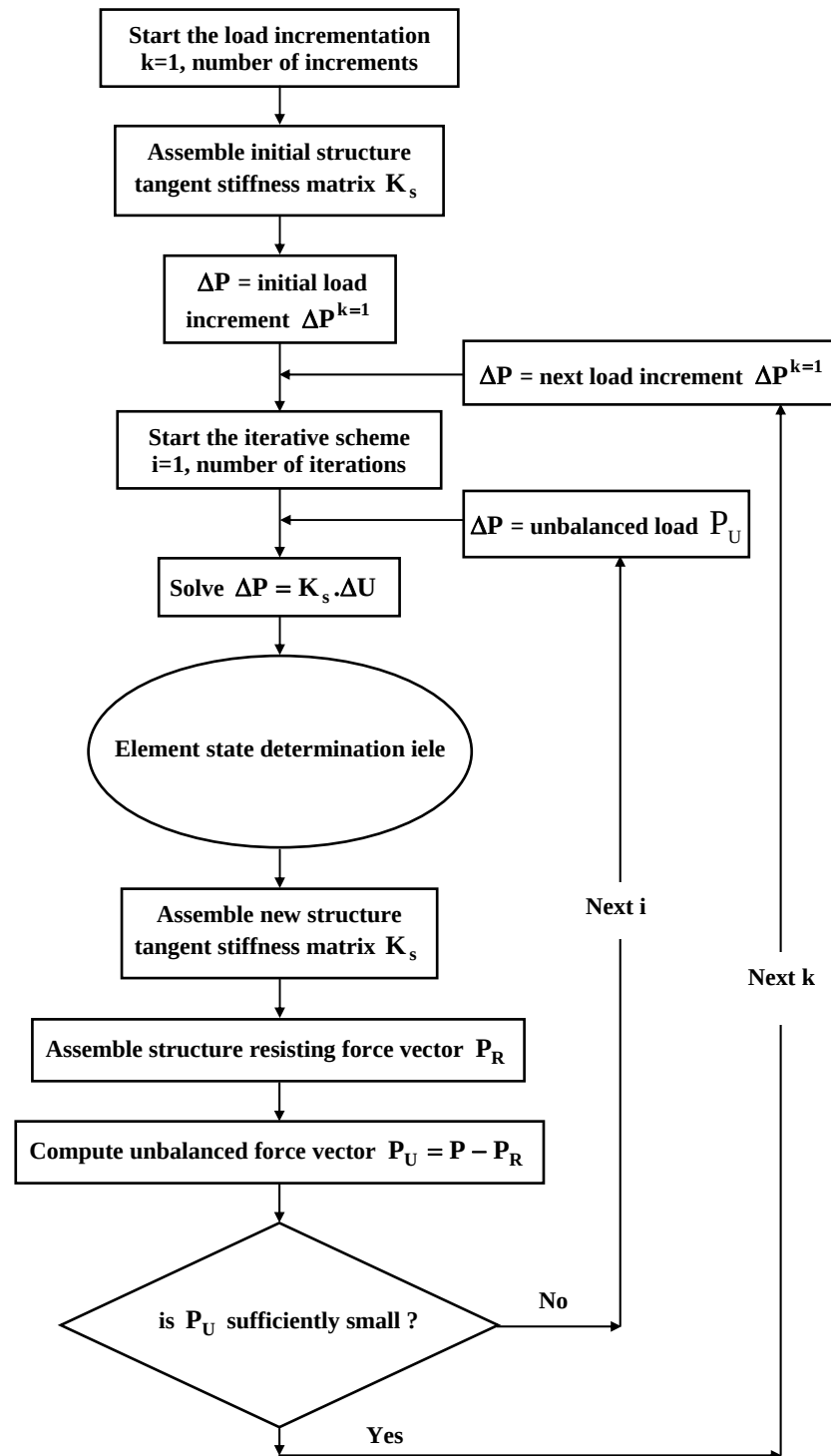


Figure 5.2: Flow chart of structure state determination

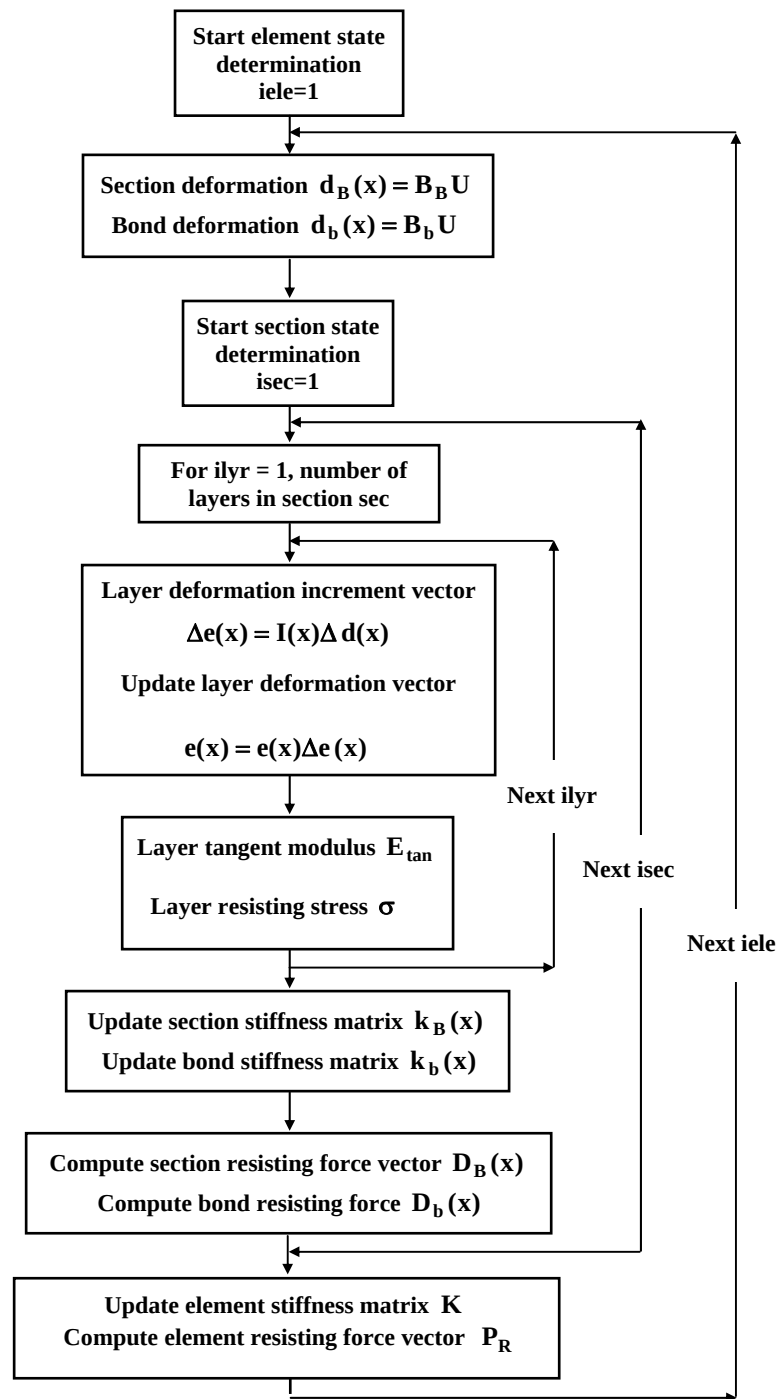


Figure 5.3: Flow chart of element state determination

CHAPTER SIX

ANALYSIS OF TEST RESULTS AND DISCUSSION

6.1 General

The main objective of the current research work is to investigate the effect of CFRP plates on the flexural behavior of strengthened RC beams. In this chapter, test results from the experimental program will be presented. They will be compared to investigate the significance of the considered experimental variables. These experimental variables include, steel reinforcement ratio, amount of CFRP plates, effect of loading type, monotonic and repeated. In the current research program, six beam specimens were tested to study the influence of different variables considered, four beams were strengthened with CFRP laminates and two beams were tested without strengthening. The unstrengthened specimens act as reference beams for comparing the performance of CFRP strengthened beams. All the reinforced concrete beams had the same length and cross-sectional area. Test results were analyzed based on cracking behavior, vertical midspan deflection, ductility, strain distribution across the depth of the reinforced concrete beam at mid-span section, CFRP strain, and failure mode.

6.2 Cracking Behavior

Crack formation was monitored throughout testing to assess the behavior of the strengthened specimens in comparison with the behavior of unstrengthened control beams.

However, this study focused on the debonding behavior, the formation of cracks that led to debonding of the CFRP plates is discussed in Section 6.6. Only the first cracking load and cracking patterns of all specimens are presented in the following subsections.

6.2.1 First Cracking Loads

External strengthening of RC beam showed better enhancement in first cracking loads when compared with reference control beams, Table 6.1. Beam specimen S10M1, S10M2 showed an increase in first cracking load of 25 and 50 percent, respectively, when compared with the control beam specimen U10M0. While both the beam specimens S12ME and S12CE showed a maximum increase of 67 percent in the first cracking load.

Table 6.1: First cracking load for the tested beams

Specimen	First cracking load , kN	Increase in cracking load , %
U10M0	12	N/A
S10M1	15	25
S10M2	18	50
U12M0	12	N/A
S12ME	20	67
S12CE	20	67

6.2.2 Cracking Patterns

The cracking behavior of each beam specimen is discussed in the following:

- *Control Beam Specimen U10M0*

The control beam specimen U10M0 was tested in order to have an unstrengthened beam to compare to the CFRP strengthened beams. The control beam behaved in expected fashion under flexural loading. It was gradually loaded until the initiation of cracking. The appearance of

flexural cracks was first at 12 kN within the constant moment region. Flexural cracks formed and widened as loading proceeded. At 20 kN, new flexural cracks formed and widespeared along the span. The flexure-shear crack appeared at a load of 30 kN. Further flexure-shear cracks occurred near each end of the specimens as the load was increased to 40 kN while the flexural cracks in the mid span widened and grew to approximately 180mm from the soffit of the beam. At 50 kN, the cracks did not grow for the reminder of the test but the flexural cracks in the constant moment region widened .Failure of the test specimen was by yielding of steel followed crushing of the concrete at the compression fiber, as shown in Figure 6.1.



Figure 6.1: Crack pattern after failure for beam U10M0

- *Strengthened Beam Specimen S10M1*

For Strengthened beam specimens S10M1, which strengthened with only one CFRP plate, the first cracking was observed at the load of 15 kN with initial crack forming in the constant moment region. As the load was increased new cracks formed along the beam specimen. In fact, cracks did not open widely during the post-cracking behavior of the beam, since the presence of the CFRP plate delay the crack initiation and arrested their propagation. Also, distinct reduction in the number of cracks was observed (Figure 6.2).The flexure-shear cracks formed at 55

kN .At this point, the flexural cracks had grown to approximately 140 mm from the soffit of the beam. For the fully developed crack pattern, the crack spacing within beam span was approximately 70 mm.

The formation of flexural cracks that occurred as a result of the yielding of embedded steel reinforcement generated high stresses in the CFRP plates across the cracks. Since the concrete could not maintain the interface shear stresses, cracks at the interface between the CFRP and the concrete were formed. Peeling of the CFRP plates from the concrete beam occurred at 70 kN. The CFRP plates ripped adhesive and a thin layer of the concrete off from the beam soffit.



Figure 6.2: Crack pattern after failure for beam S10M1

- *Strengthened Beam Specimen S10M2*

The strengthened beam specimen S10M2, which was strengthened with two CFRP plates, loaded with the first cracking observed at 18 kN. Similar to the control beams specimen, the initial crack formed in the constant moment region. As the load was increased to 25 kN new cracks formed within the constant moment region. Distinct reduction in the number of cracks was observed (Figure 6.3). Loading was continued and the cracks marked until a load of 45 kN .At this point, the initial flexure-shear cracks formed at approximately 80mm from CFRP anchorages at a load 45 kN .Flexure-shear cracks had grown outside of the constant

moment region after 60 kN. Finally, debonding failure occurred at 90 kN in the same as for S10M1.

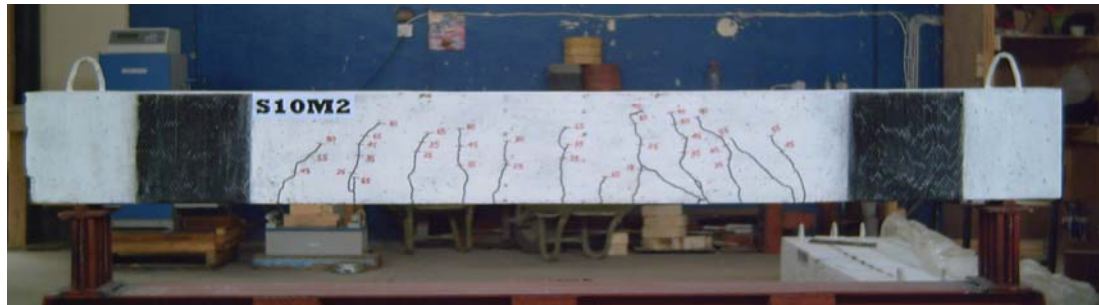


Figure 6.3: Crack pattern after failure for beam S10M2

- *Control Beam Specimen U12ME*

Specimen U12ME was a control specimen for the second group of tests (steel reinforcement ratio 0.81 percent). The beam was loaded gradually until the first crack was observed. The flexural cracks appeared in the constant moment region at a load of approximately 12 kN. Further flexural cracks formed and widened as the loading increased. At 20 kN, flexural cracks were widespread along the beam between 500 mm from the mid span towards each support. The cracks in the constant moment region had developed to approximately 120 mm from the soffit of the beam. When the tension reinforcement began to yield, shear and flexural-shear cracks began forming in the shear region of the beam. Flexure-shear cracks formed each support at a load of 40 kN. The flexural cracks in the constant moment region widened with increasing load and at 89 kN the crack pattern was fully developed. Failure of the test specimen was by yielding of steel followed crushing of the concrete at the compression fiber, as shown in Figure 6.4.



Figure 6.4: Crack pattern after failure for beam U12M0

- *Strengthened Beam Specimen S12ME*

Beam specimen S12ME was strengthened with two CFRP plate were installed on the tension bottom face of the reinforced concrete beam. The cracks pattern of this beam specimen is shown in Figure 6.5. First crack occurred at a higher load than of the unstrengthened beam specimen U12M0, which was observed at an applied load of 20 kN .The flexural cracks appeared in the constant moment region at a load 30 kN . The flexure shear cracks formed at 60 kN. At this point, the flexural cracks had grown to approximately 120mm from the soffit of the beam. At 120 kN, horizontal cracks appeared at the level of steel reinforcement. Failure was by mixed interfacial crack-induced debonding with full cover separation.

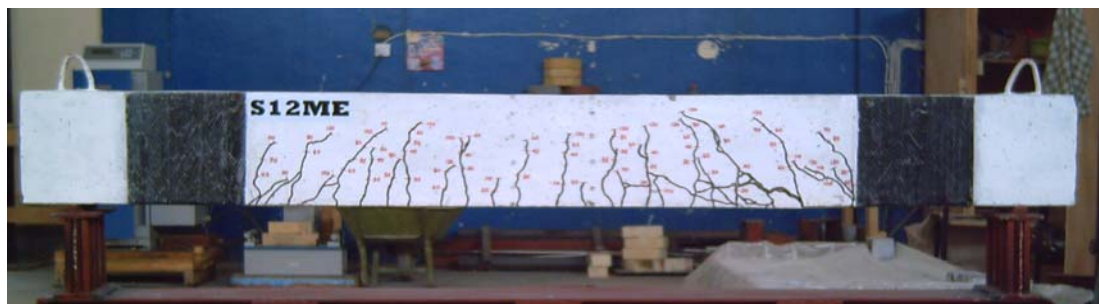


Figure 6.5: Crack pattern after failure for beam S12ME

- *Strengthened Beam Specimen S12CE*

To study the effect of the cyclic loading on the response of the strengthened RC beams, cyclic load was applied to the specimen S12CE. During the first cycle (15 kN), no creaking was observed. Then; the specimen was loaded with the first cracking observed at 20 kN. Loading was continued and the cracks marked until a load 25 kN (cycle 2). The beam was then unloaded to zero and reloaded up to 35 kN (cycle 3). On the resumption of loading, at 45 kN (cycle 4), new flexural cracks occurred between the cracks. The cracks were not fully closed during the unloading and widened during the reloading. Flexure-shear cracks had grown outside of the constant moment region after the cycle 5 (55 kN). During cycle 10 (105 kN), horizontal cracks appeared at the level of steel reinforcement. After this, a sudden debonding failure occurred at 120 kN. Failure was by mixed ICI debonding with full cover separation. Crack pattern after the failure of the beam specimen S12CE is shown in Figure 6.6.



Figure 6.6: Crack pattern after failure for beam S12CE

6.3 Load-Deflection Curves

Six beam specimens were cast, and tested under monotonic and cyclic loading to examine the response of CFRP strengthened RC beams.

The structural behaviors of these beam specimens are represented here by their load versus midspan deflection shown in Figures 6.7 to Figure 6.9.

To study the effect of external strengthening on the monotonic behavior of RC beams, three beam specimens were cast with the two different steel reinforcement ratios (0.56 percent for S10M1 and S10M2; and 0.81 percent for S12ME). The beam specimen S10M1 was strengthened with one CFRP plate, while the two others beam specimen strengthened by two CFRP plate. The unstrengthened specimens, U10M0 and U12ME act as reference beam for comparing the performance of CFRP strengthened beams. The responses for these three beam specimens are shown in Figure 6.7 and Figure 6.8.

Generally, it can be observed that the load versus midspan deflection response can be divided into three stages of behavior. The first stage was characterized by an approximately linear relationship between the load and the midspan deflection. During this stage of behavior, the section was uncracked and both the concrete and steel, in addition to the CFRP plate, behave essentially elastic.

The second stage represents the behavior beyond the initial cracking of the composite section where the stiffness of the beam was decreased as indicated by the reduced slope of the load versus midspan deflection curve. The end of this stage was distinguished when the main steel reinforcement start to exhibit inelastic behavior.

The third stage was characterized by a decreasing slope of the curve, where the tension steel reinforcement reaches the strain hardening stage. For the unstrengthened control specimens (U10M0 and U12ME) a large increase in deflection was noticed in the third stage, while the applied load changed little. While the strengthened beam specimens failed suddenly and exhibited no ductility.

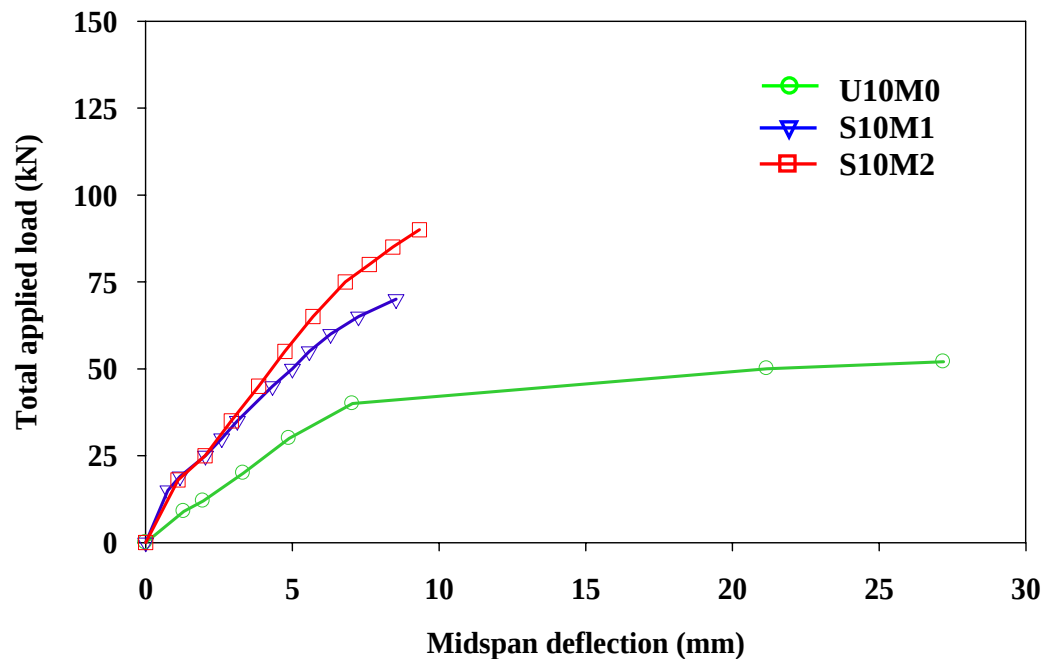


Figure 6.7: Load vs. deflection for beams U10M0, S10M1 and S10M2

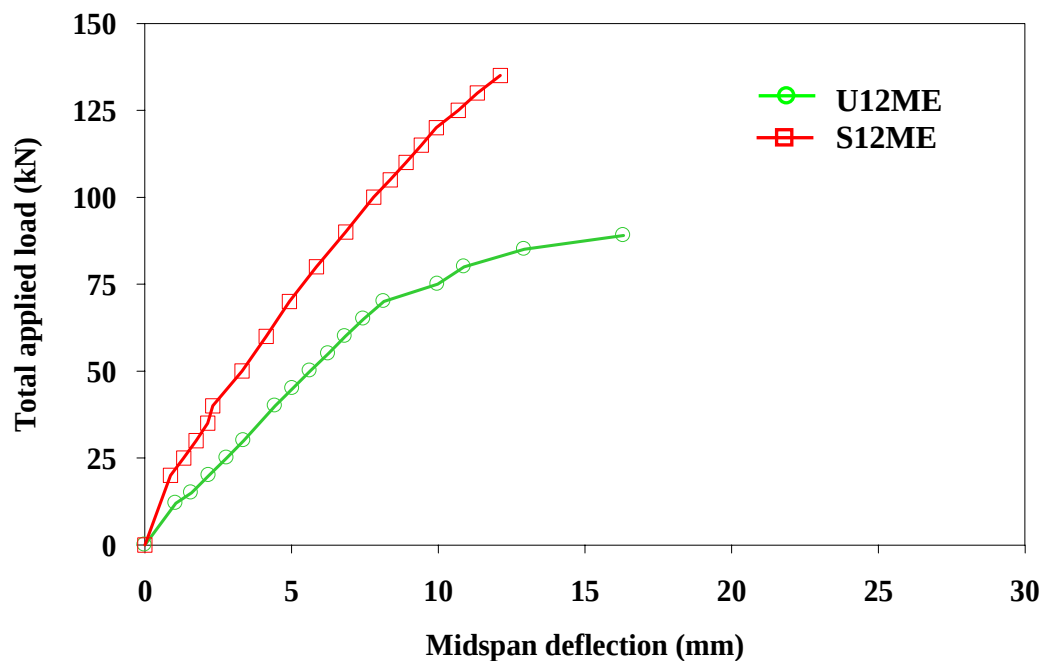


Figure 6.8: Load vs. deflection for beams U12M0 and S12ME

Generally, the deflections of the beams strengthened with CFRP were lower than the deflections of the corresponding control beam. The reduction in the maximum deflections was 69 and 66 percent for the beam specimen S10M1 and S10M2, respectively.

The performance of strengthened members under cyclic loading is of great importance for seismic retrofit applications. Figure 6.9 shows a comparison of performance for two strengthened beams, one of which tested under monotonic load (labeled as S12ME) and the other under cyclic loading (labeled as S12CE). The cyclic tests are performed as force controlled with 10 kN increments. It is seen from the figure that performance of CFRP strengthened beam under cyclic loading may fall below that under monotonic loading. Material and bond degradation is likely to occur at high amplitude cyclic loading, which reduces the performance of the beam specimen and accelerate the beam premature failure.

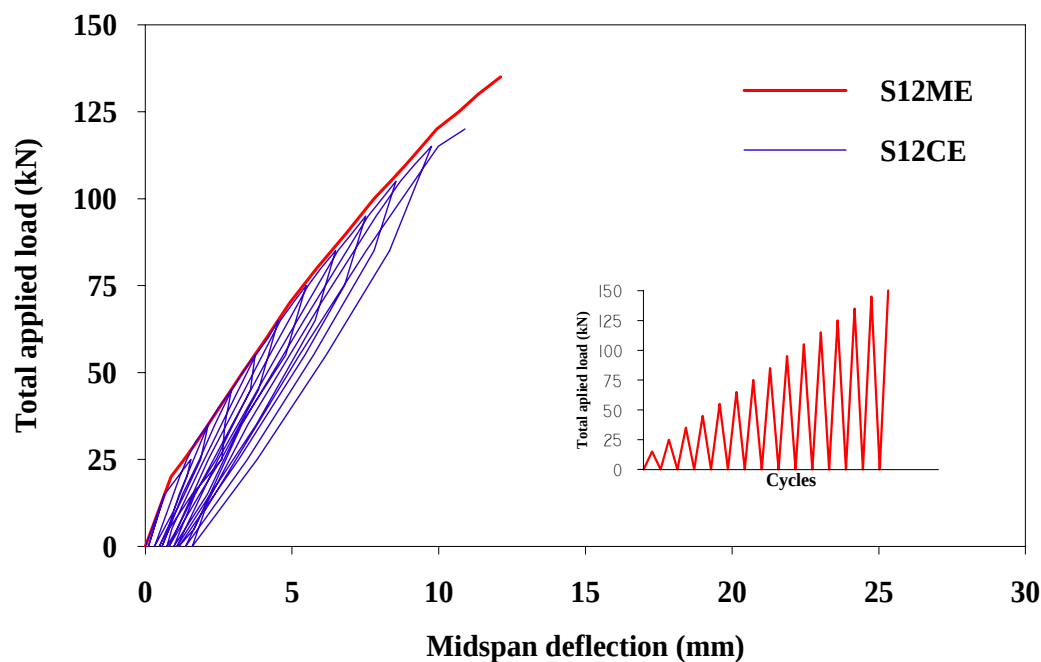


Figure 6.9: Load vs. deflection for beams S12ME and S12CE

6.4 Ductility Ratios

The ductility of a beam can be defined as its ability to sustain inelastic deformation without loss in its load carrying capacity prior to failure. Following this definition, ductility can be expressed in terms of deformation. Ductility can be calculated as the ratio of ultimate deformation to deformation at yield. The deformation can be strains, deflections, or curvatures.

For the beam specimens S10M1 and S10M2, the ductility ratio is decreased by 64 and 67 percent, respectively, in comparison with control beam specimen U10M0 as shown in Table 6.2. Also, the ductility ratios for the strengthened beam specimen S12ME and S12CE were reduced by 35 and 30 percent, respectively, in comparison with the corresponding control beam specimen U12ME.

Table 6.2: Ductility ratio for the tested beams

Specimen	Deflection , mm		Ductility ratio	Reduction in ductility ratio, %
	Yielding	Ultimate		
U10M0	7.0	27.2	3.9	N/A
S10M1	6.0	8.5	1.4	64
S10M2	7.0	9.3	1.3	67
U12M0	8.0	16.3	2.0	N/A
S12ME	9.0	12.1	1.3	35
S12CE	8.0	10.9	1.4	30

The overall and clear message of the data given in Table 6.2 is that the premature and brittle nature of debonding failure may reduce the level of safety of the strengthened RC beam by decreasing its ductility.

6.5 Concrete Strains

The distribution of concrete strains at mid span section of the tested beam specimens was measured using twelve Demec discs over the depth of each beam. The concrete strain distribution over the depth of all the tested beams at different load levels is shown in Figure 6.10 to Figure 6.15.

Generally, it can be seen that the strain distribution remained approximately linear in compression zone through out loading range. While in the tension zone, the strain distribution is approximately linear at low loads and becomes nonlinear at higher loads due to cracking. The same result can be seen in the second test group. In addition to that, the presence of CFRP laminates at the bottom tension zone surface reduced the concrete strains, and this reduction was reflected to strains in the bottom tension steel bar reinforcement (i.e., reducing the tension steel bar strains), and that means increasing the tension strength and some tensile stresses were carried by CFRP laminates.

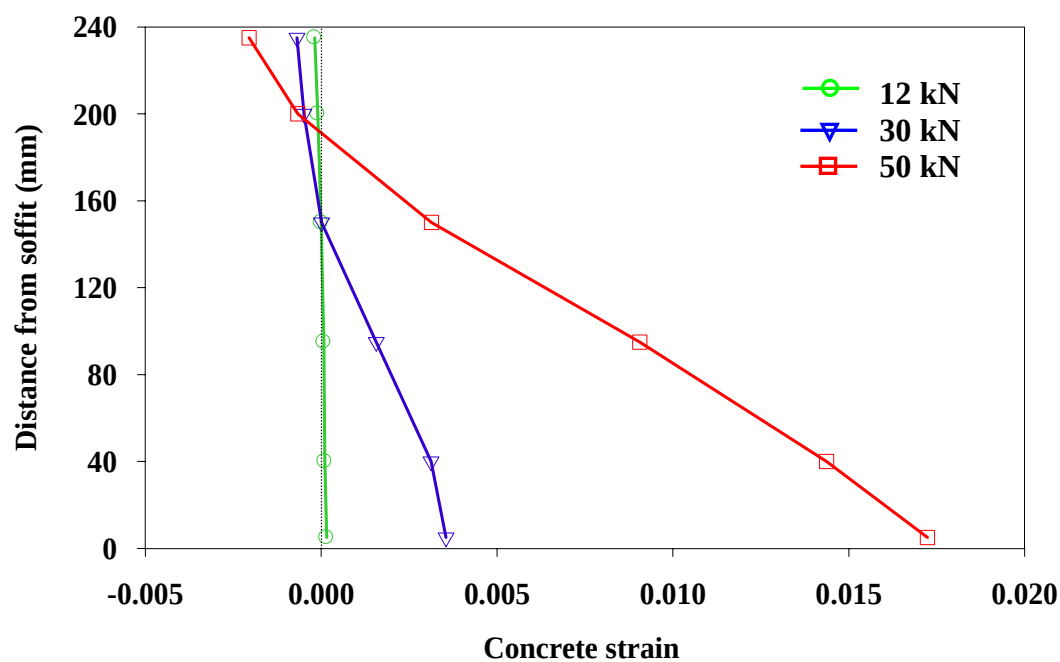


Figure 6.10: Concrete strain distribution for beam U10M0

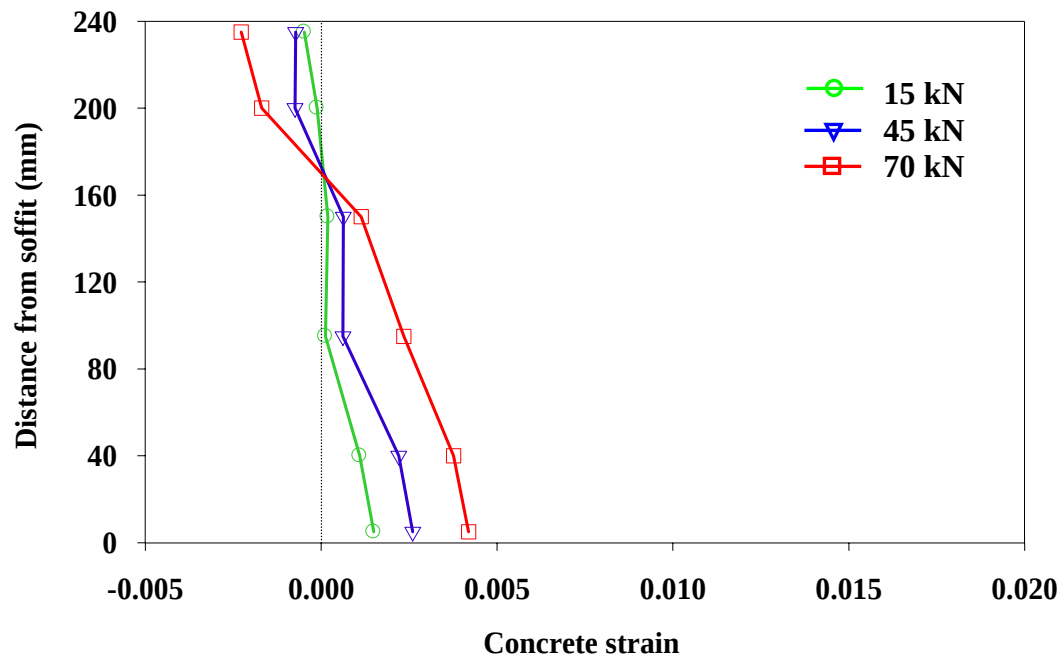


Figure 6.11: Concrete strain distribution for beam S10M1

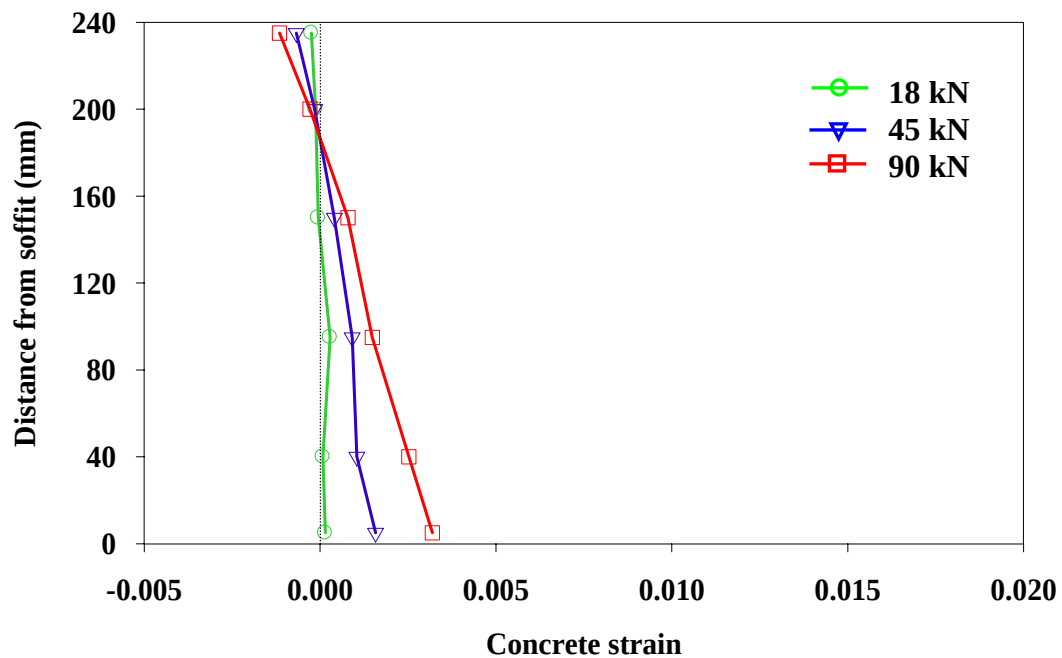


Figure 6.12: Concrete strain distribution for beam S10M2

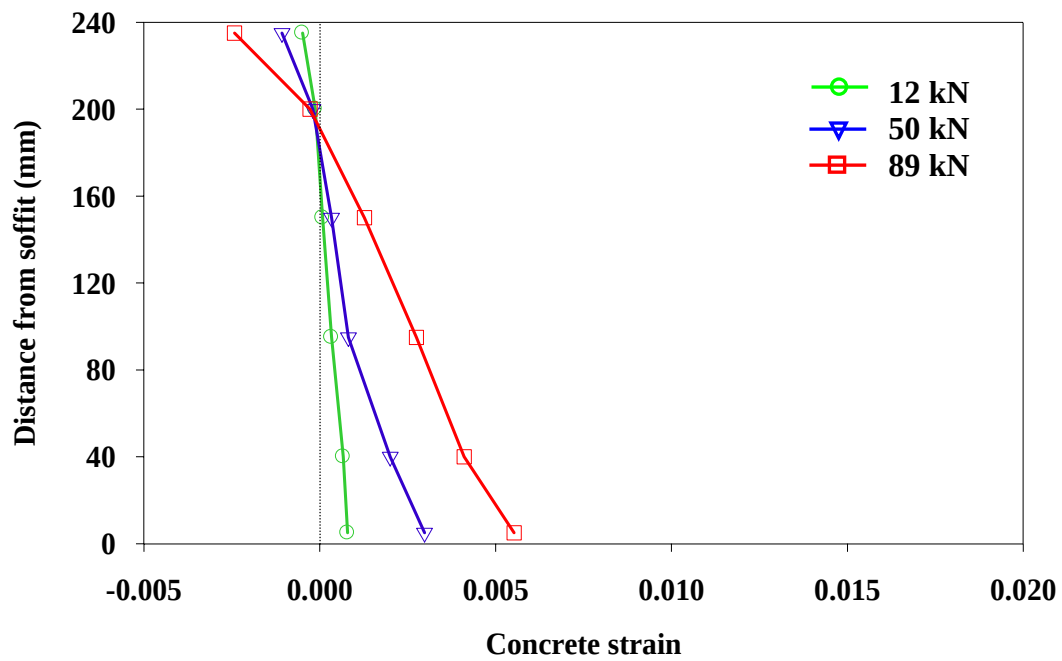


Figure 6.13: Concrete strain distribution for beam U12ME

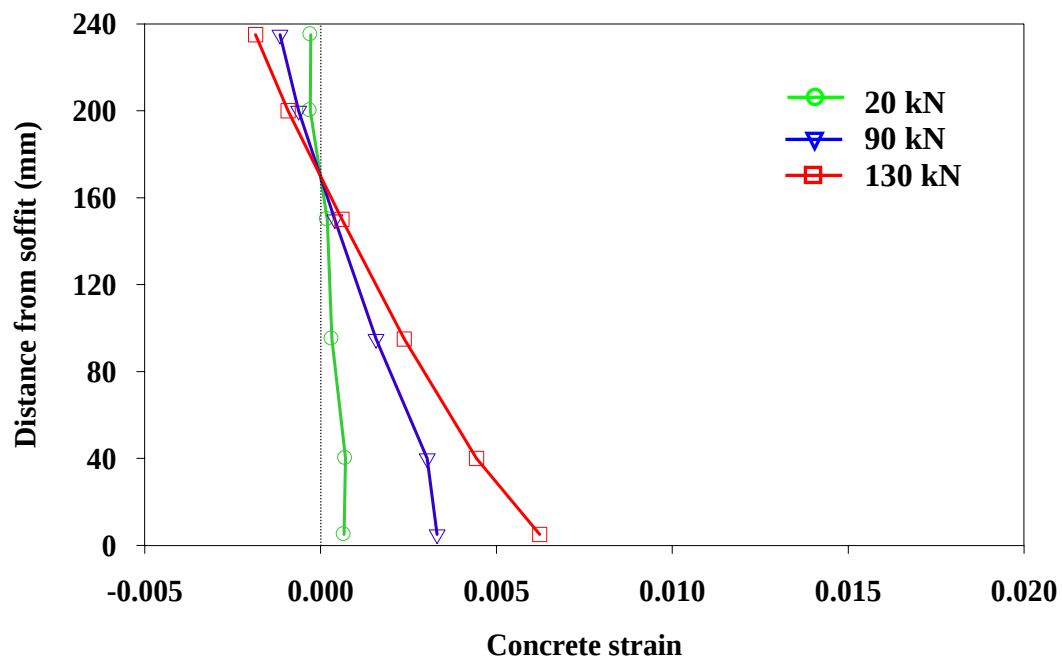


Figure 6.14: Concrete strain distribution for beam S12ME

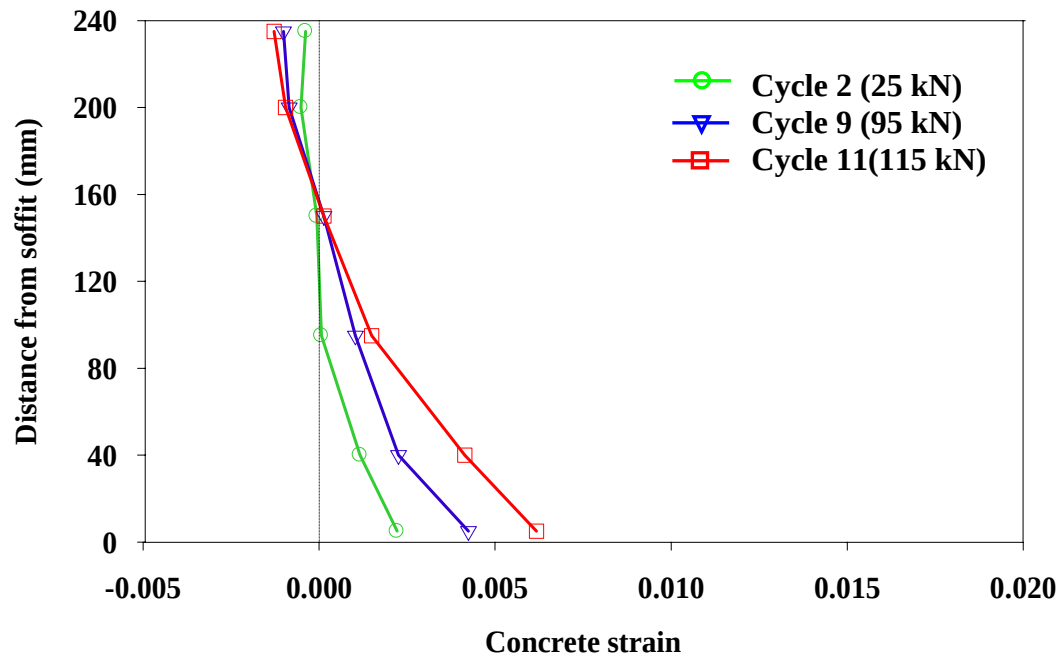


Figure 6.15: Concrete strain distribution for beam S12CE

Figures 6.16 and 6.17 clearly show the effect external CFRP strengthening on the maximum concrete compressive strain at the extreme fiber. At the same load level, the presence of the CFRP plate on the tension face of the strengthened RC beam reduces the maximum concrete strain at the extreme fiber. This reduction in concrete strain may be increased with increasing the amount of the external CFRP plate.

Also, it can be observed that the extreme fiber of the compressive concrete may not reach its ultimate strain at failure. The concrete strength in the compression zone cannot fully developed in this case and the compression force resisted by the concrete will be overestimated if the compressive stress block used in the design of conventional RC beams is adopted.

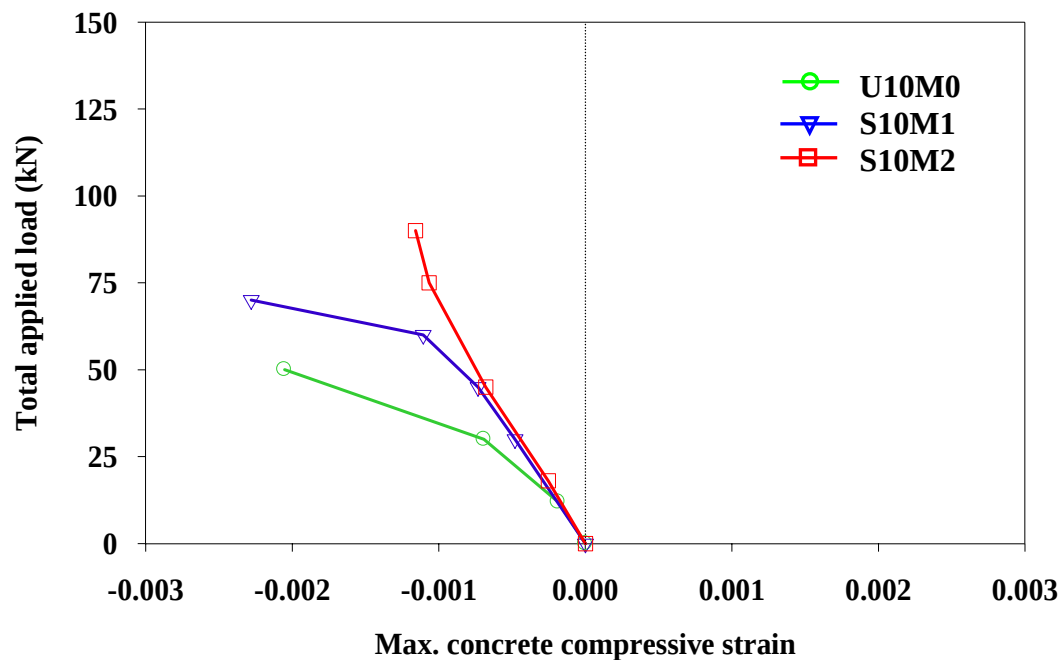


Figure 6.16: Load vs. maximum concrete compressive strain for beams U10M0, S10M1, and S10M2

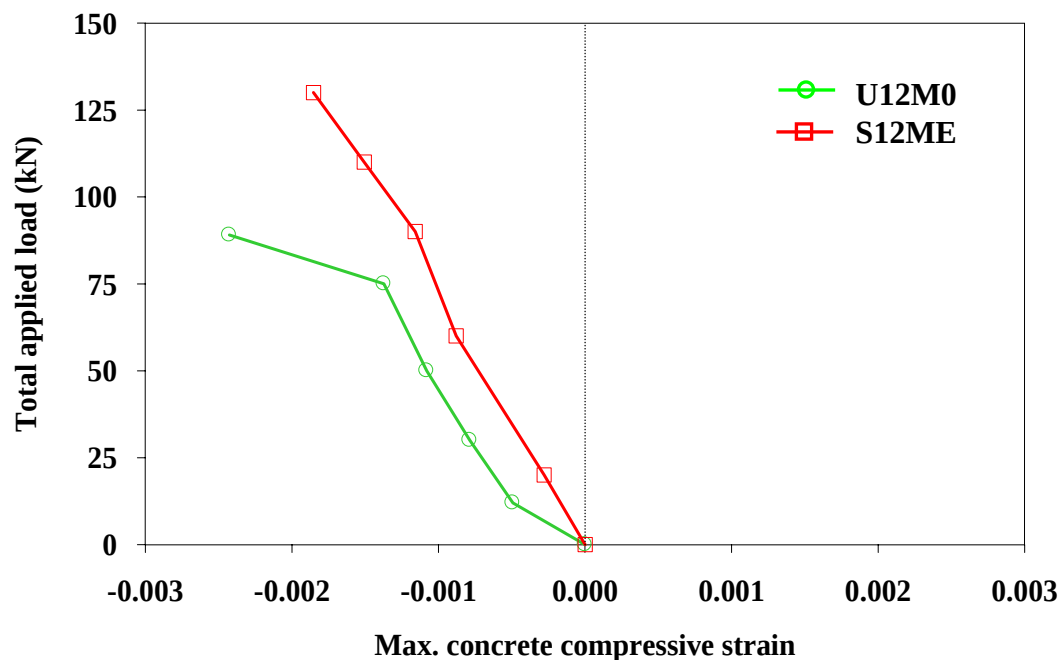


Figure 6.17: Load vs. maximum concrete compressive strain for beams U12M0 and S12ME

6.6 Ultimate Load and Failure Modes

All the unstrengthened and strengthened RC beam specimens were tested up to failure. The recorded ultimate loads and failure modes for these beam specimens are presented in Table 6.3.

Table 6.3: Ultimate load capacity and failure mode for tested beams

Specimen	Ultimate load , kN	Increase in ultimate load ,%	Failure mode
U10M0	52	N/A	Typical flexural failure
S10M1	70	35	Debonding
S10M2	90	73	Debonding
U12M0	89	N/A	Typical flexural failure
S12ME	135	52	Debonding
S12CE	120	35	Debonding

For the beam specimen S10M1, which strengthened with one CFRP plate, the increase in ultimate load capacity was about 35 percent when compared with the corresponding control beam specimen U10M0. While the beam specimen S10M2, which strengthened with two CFRP plate, showed an increase of 73 percent in ultimate load capacity when compared with the control beam. Further, beam specimen S12ME, which reinforcement ratio of 0.92 percent and strengthened with two CFRP plates, showed an increase of only 52 percent in ultimate load capacity over the control beam U12ME. This increase in ultimate load was much lower than that obtained with steel reinforcement ratio of 0.51 percent of strengthened beam specimen S10M2.

Effect of cyclic loading on the ultimate load capacity can be

explained through comparing the results of beams S12ME (which subjected to monotonic increasing load up to failure) with companion beam specimens. Strengthened beam specimen S12CE results in 35 percent increase in its ultimate load capacity when compared with the control beam specimen, as shown in Table 6.3. On the other hand, beam specimen S12CE showed a reduction of 11 percent in ultimate load capacity after twelve increasingly load cycles when compared with the monotonically tested beam specimen S12ME. This reduction may be attributed to the degradation of material and bond characteristics.

The control beam specimens, U10M0 and U12ME behaved in expected fashion under flexural loading. The failure due to yielding of tension steel reinforcement followed by crushing of concrete at midspan. The control beam specimens were used as a baseline to compare the remaining strengthening beam specimen. Figure 6.18 shows the flexural failure mode of control beam specimen U10M0.

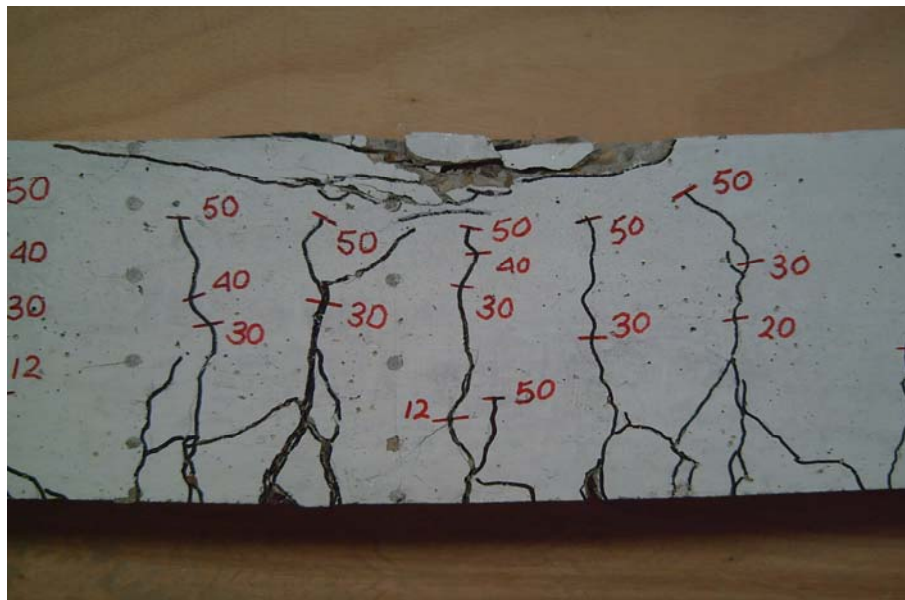


Figure 6.18: Flexural failure mode of specimen U10M0

Two kinds of failure modes occurred on the CFRP-strengthened beams in this experiment. The first is the intermediate flexure crack-induced debonding failure which was occurred in beam specimen S10M1, the second, in the main, is the intermediate shear crack-induced debonding near one of the end anchorages with or without concrete cover separation.

No evidence of debonding of the CFRP plates was observed until after yielding of the reinforcement. The formation of secondary cracks adjacent to existing flexural cracks was an indication of initiation of debonding of the CFRP plates. These cracks were followed by the formation of longitudinal debonding cracks along the sides of the CFRP plate.

The debonding in beam specimens was very sudden and the only indication of incipient failure was few popping sounds as the debonding cracks propagated quickly to the end of the plate.



Figure 6.19: Debonding failure mode of specimen S10M1

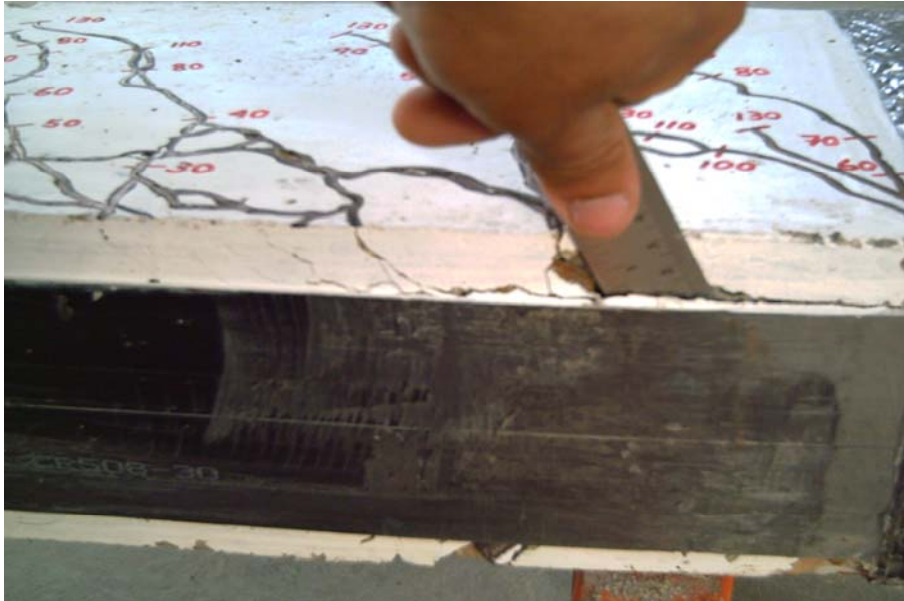


Figure 6.20: Debonding failure mode of specimen S12ME



Figure 6.21: Debonding failure mode of specimen S12CE

6.7 CFRP Strains

To prevent debonding of the FRP laminate, ACI Committee 440 (2002) proposed limiting the strain level developed in the laminate. For the particular CFRP external strengthening used, the limiting effective strain is calculated in accordance with the ACI Committee 440 (2002) as $\varepsilon_{fu} = 0.0081$, which is shown with dashed line as the ACI limit in Figure 6.22. Comparison of this limit with the experimental results indicated that this calculated strain limit is nonconservative for all strengthened beam presented in this study which failed through debonding at a CFRP strain level below 0.005.

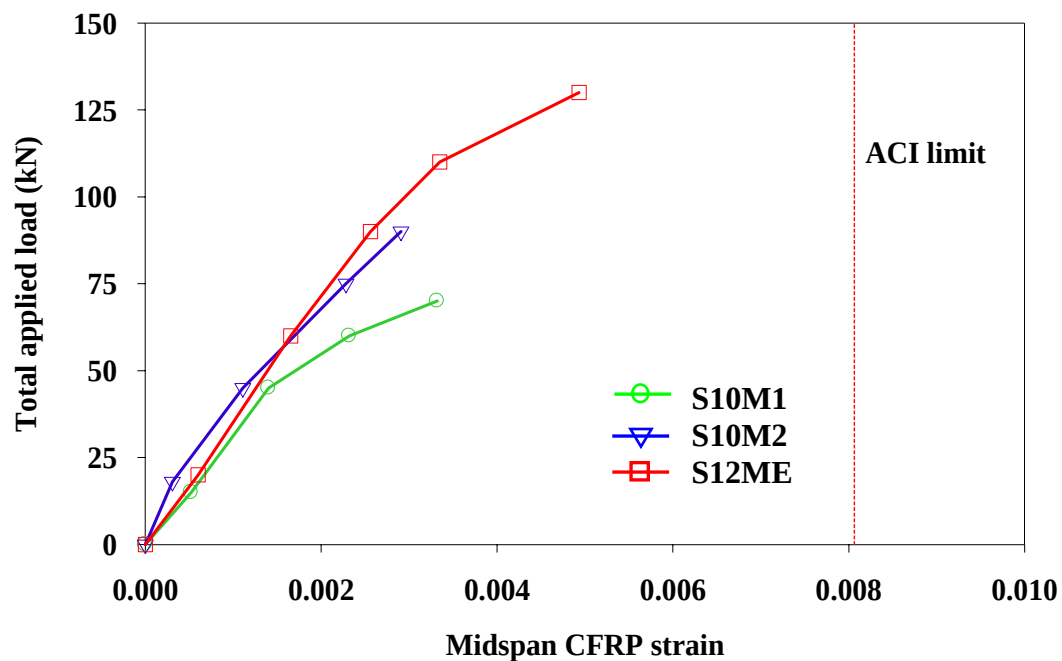


Figure 6.22: Load vs. midspan CFRP strain for tested beam specimens

CHAPTER SEVEN

VERIFICATION OF THE FINITE ELEMENT MODEL

7.1 General

A series of examples are presented in this chapter to demonstrate the ability of the beam element presented in Chapter Four to describe the behavior of unstrengthened and strengthened RC beam under monotonic and cyclic load. The numerical results in this chapter are obtained with the stand alone computer program NBSRCB (Nonlinear Behavior of Strengthened Reinforced Concrete Beams) whose numerical implementation was discussed in the previous chapter.

7.2 Element and Section Discretization

An important step in the finite element modeling is the selection of the mesh density. A convergence of results is obtained when an adequate number of elements are used in a structure. This is practically achieved when an increase in the mesh density has a negligible effect on the results. Therefore, in this finite element modeling, a convergence study was carried out to determine an appropriate mesh density. The convergence study was made by increasing the number of elements as well as the number of the layer in each section .When an increase in mesh has a negligible effect on the results of mid-span deflection, it is assumed that the convergence of results is obtained.

Thanks to the symmetry of the test setup, only half the specimen was considered for the finite element analysis. A mesh containing a total of 12 elements was used for the two unstrengthened control beam specimens as shown in Figure 7.1.

For the CFRP strengthened beam specimens, a mesh containing a total of 20 elements was used to obtain a sufficiently smooth response in terms of bond-stress distribution, although the overall applied load-midspan deflection response converged with as few as 15 elements.

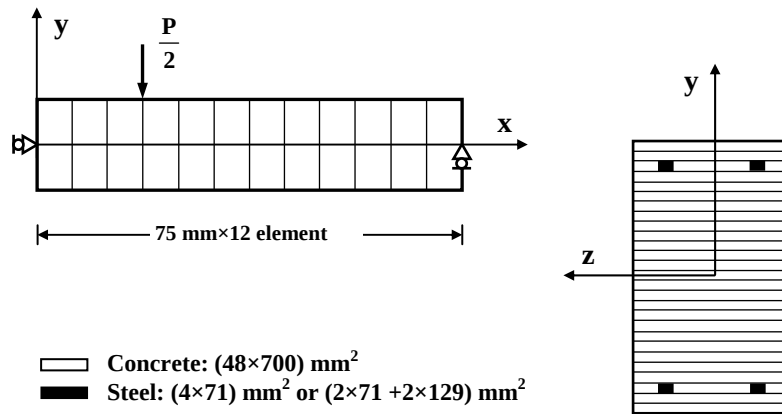


Figure 7.1: Element and section discretization for control beams

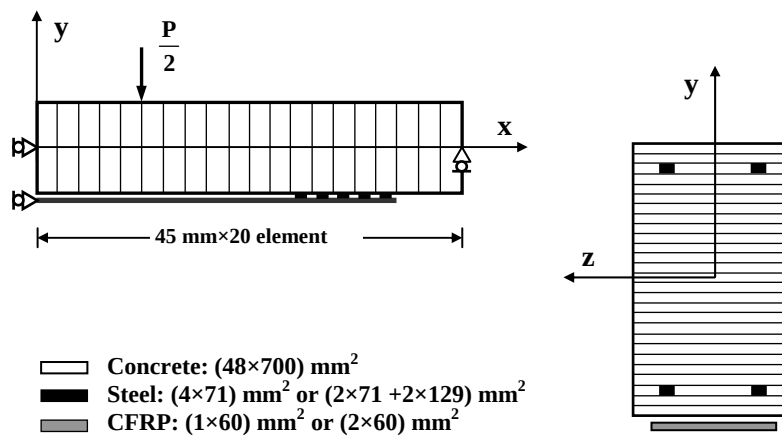


Figure 7.2: Element and section discretization for strengthened beams

7.3 Results of the Finite Element Model

To illustrate the validity of the developed nonlinear finite element model for the analysis of RC strengthened with CFRP plates, all tested beam specimens will be analyzed by using NBSRCB developed computer program, as mentioned previously. The analysis follows the same procedure as that given in Chapter Four and Chapter Five taking into account the variation in material properties, dimensions and other specifications described previously in Chapter Three.

7.3.1 First Cracking Loads

Table 7.1 shows the numerical obtained results that are related with first cracking load. The first cracking load was increased by 31 and 58 percent in S10M1 (strengthened with one CFRP plate) and S10M1 (strengthened with two CFRP plate) beam specimens, respectively. It can be observed that the first cracking load in strengthened beams increases with the increasing the CFRP amount when compared with the corresponding unstrengthened control beams.

Table 7.1: Numerical first cracking loads for the tested beam

Specimen	First cracking Load (kN)			Increase in numerical first cracking load, %
	Experimental	Numerical	Num./Exp.	
U10M0	12	12	1.00	N/A
S10M1	15	17	1.13	31
S10M2	18	19	1.06	58
U12ME	12	13	1.08	N/A
S12ME	20	19	0.95	46

On the other hand, the first cracking load in strengthened beam specimen S12ME beam specimen was increased by 46 percent when compared with its unstrengthened control beam specimen U12ME.

The first cracking load obtained from the numerical results showed good agreement with experimental data recorded with difference ranging from 0 to 13 percent.

7.3.2 Load Deflection Curves

A comparison between experimental and numerical load versus midspan deflection curves are shown in Figures 7.3 to 7.7. Also, the ductility ratios calculated based on the numerical results is presented in Table 7.2.

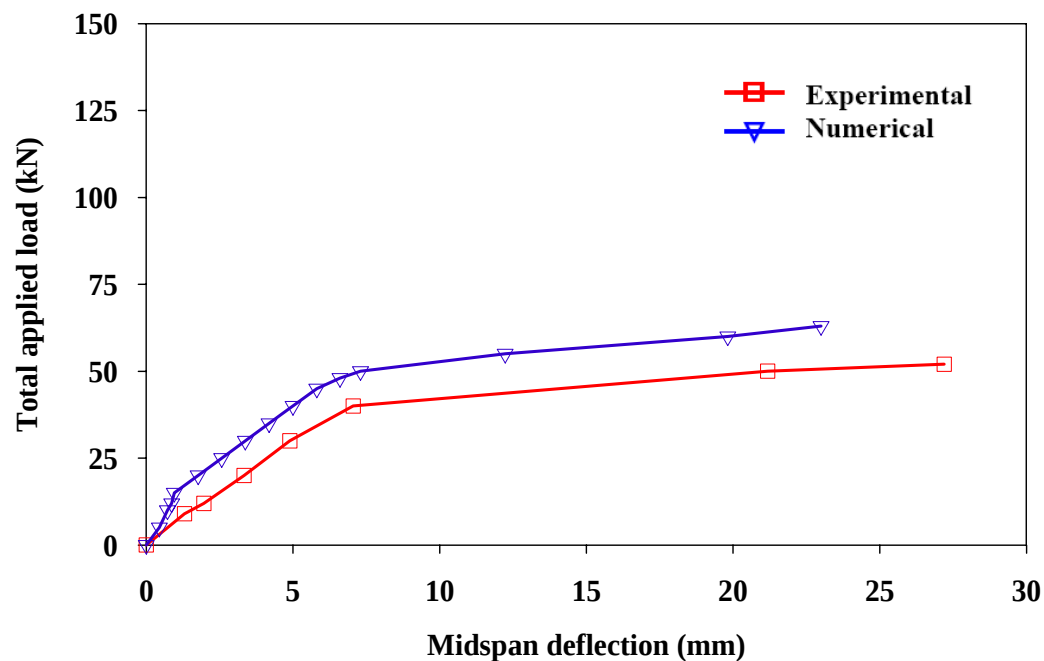


Figure 7.3: Load versus midspan deflection for beam U10M0

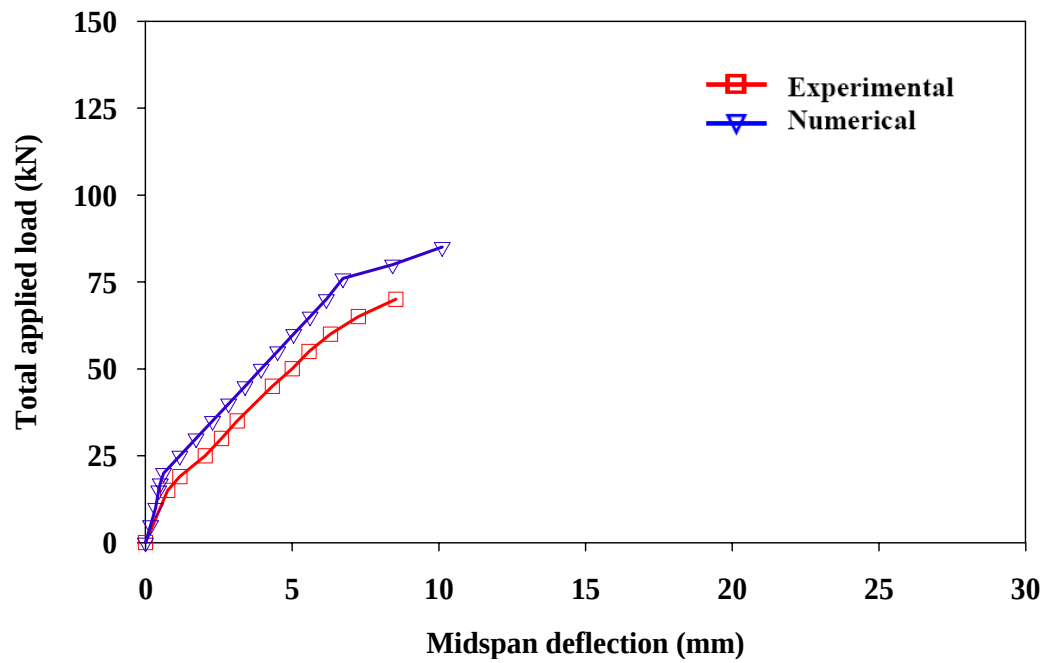


Figure 7.4: Load versus midspan deflection for beam S10M1

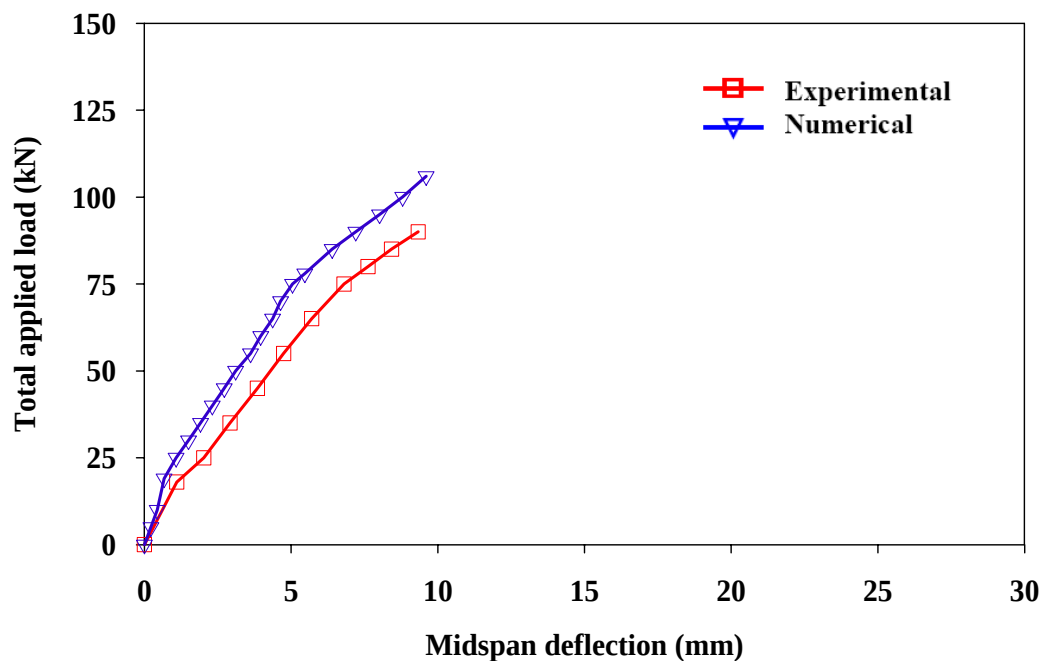


Figure 7.5: Load versus midspan deflection for beam S10M2

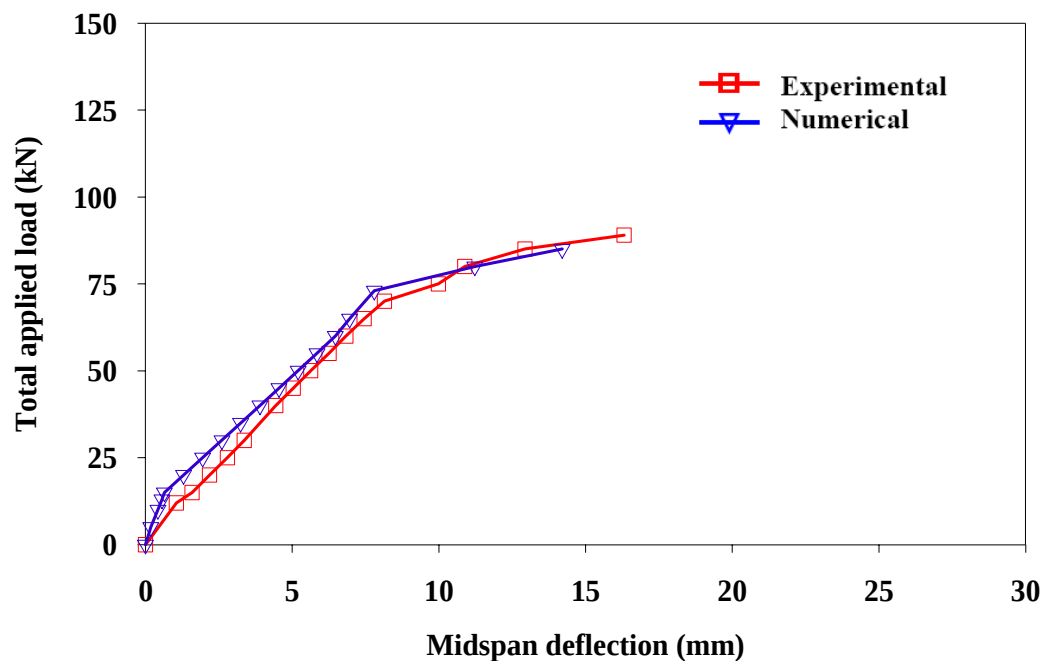


Figure 7.6: Load versus midspan deflection for beam U12ME

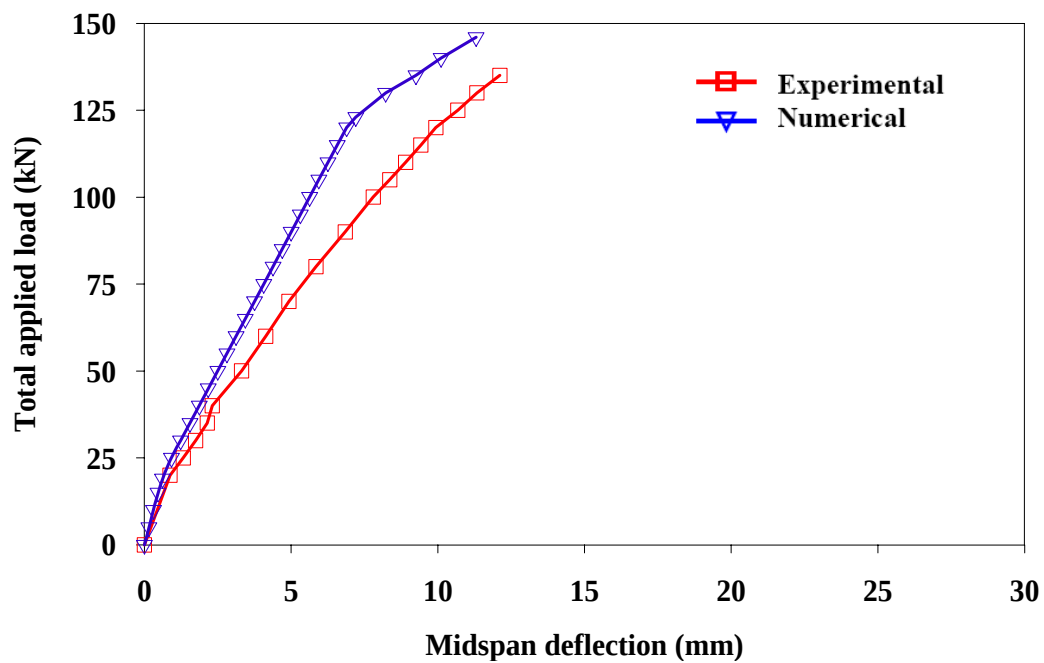


Figure 7.7: Load versus midspan deflection for beam S12ME

Table 7.2: Numerical ductility ratios for the tested beams

Specimen	Deflection , mm		Ductility ratio	Reduction in ductility ratio, %
	Yielding	Ultimate		
U10M0	6.6	23.1	3.5	N/A
S10M1	6.7	10.1	1.5	57
S10M2	5.6	9.6	1.7	51
U12M0	7.8	14.2	1.8	N/A
S12ME	7.2	11.3	1.6	11

The finite element results for the control beam specimen U10M0, Figure 7.3, shows stiffer response compared with the experimentally recorded curve for the pre-cracking and post-cracking as well as for the ultimate stage. The computed ductility ratio, which is defined as the ratio of ultimate to yield deflections is 3.5 and is lower than the experimental ductility ratio.

Figures 7.4 and 7.5 show a comparison between the load versus midspan deflection curves for experimental and numerical results for the strengthened beam specimens S10M1 and S10M2, respectively. Again, slightly stiffer behaviors are obtained in the numerical results when compared with that obtained from experiments. The computed ductility ratios are 1.5 and 1.7, respectively. Numerically, the use of the CFRP to strength these two beam specimens reduce its ductility ratios by 57 and 51 percent, respectively.

An experimental and numerical load versus midspan deflection comparisons are made in Figures 7.6 and 7.7 for the control beam specimen U12ME and corresponding strengthened beam specimen S12ME, respectively. In spite of the fact the numerical curve in Figure

7.6 shows a slightly stiffer behavior in the pre-cracking and post-cracking stages, but it shows also a relatively lower ultimate load.

The numerical ductility ratio for the strengthened beam specimen S12ME is 11 percent less than for the corresponding control beam (U12ME).

7.3.3 Concrete Strain Distribution

Concrete strain distribution along the depth of the tested beam specimens in the constant moment zone has been experimentally measured using Demec gauges. Experimental and FE numerical results are plotted for selected loading levels during the loading stages, as shown through Figure 7.8 to Figure 7.12.

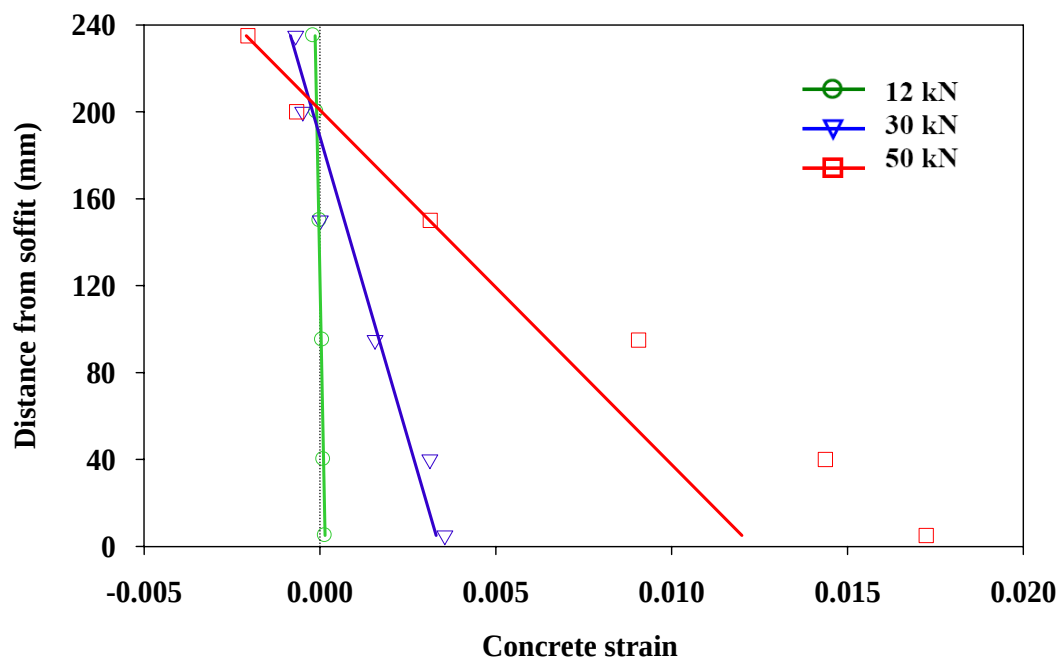


Figure 7.8: Concrete strain distribution for beam U10M0

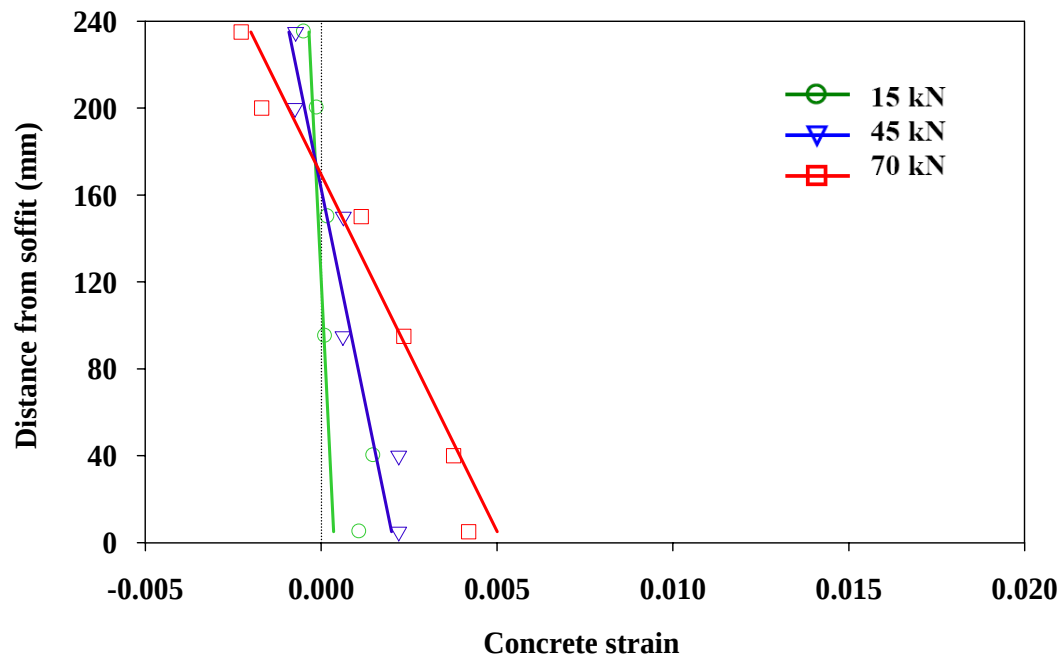


Figure 7.9: Concrete strain distribution for beam S10M1

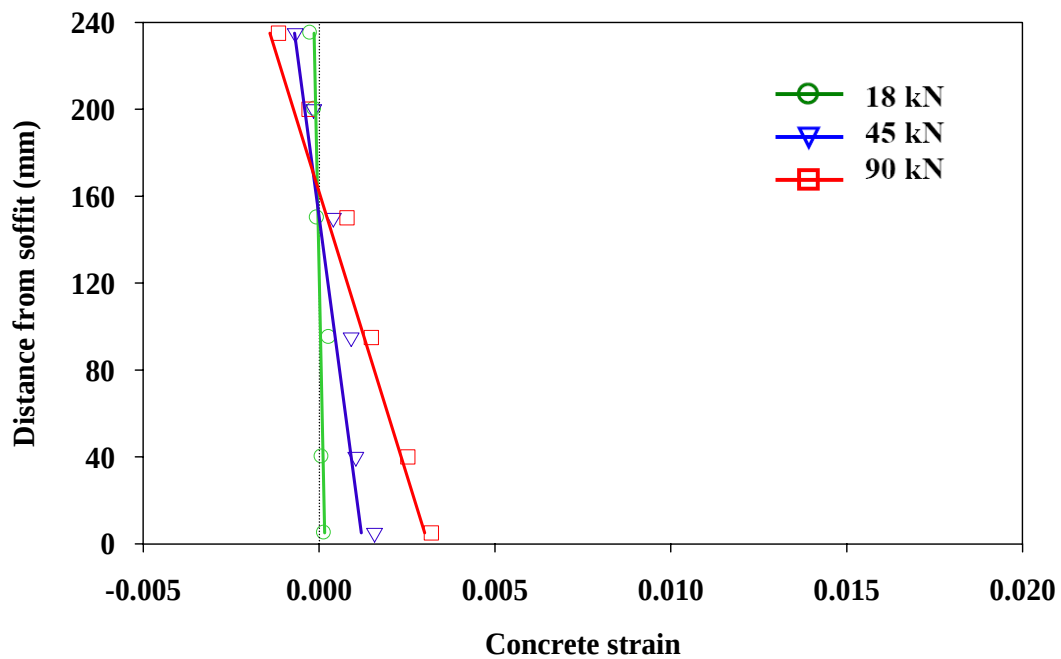


Figure 7.10: Concrete strain distribution for beam S10M2

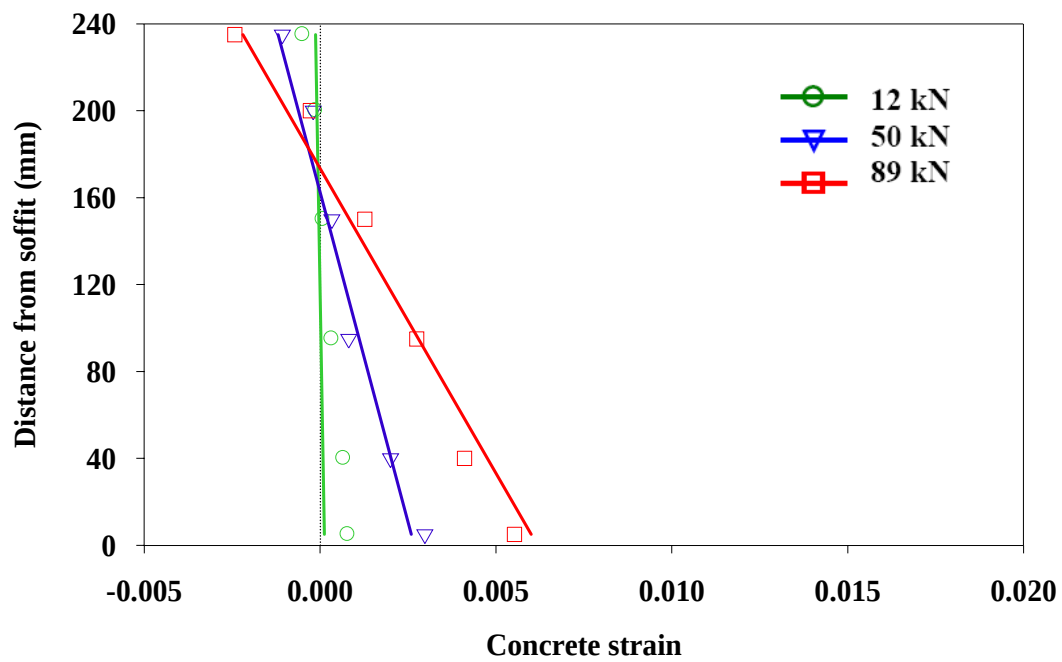


Figure 7.11: Concrete strain distribution for beam U12ME

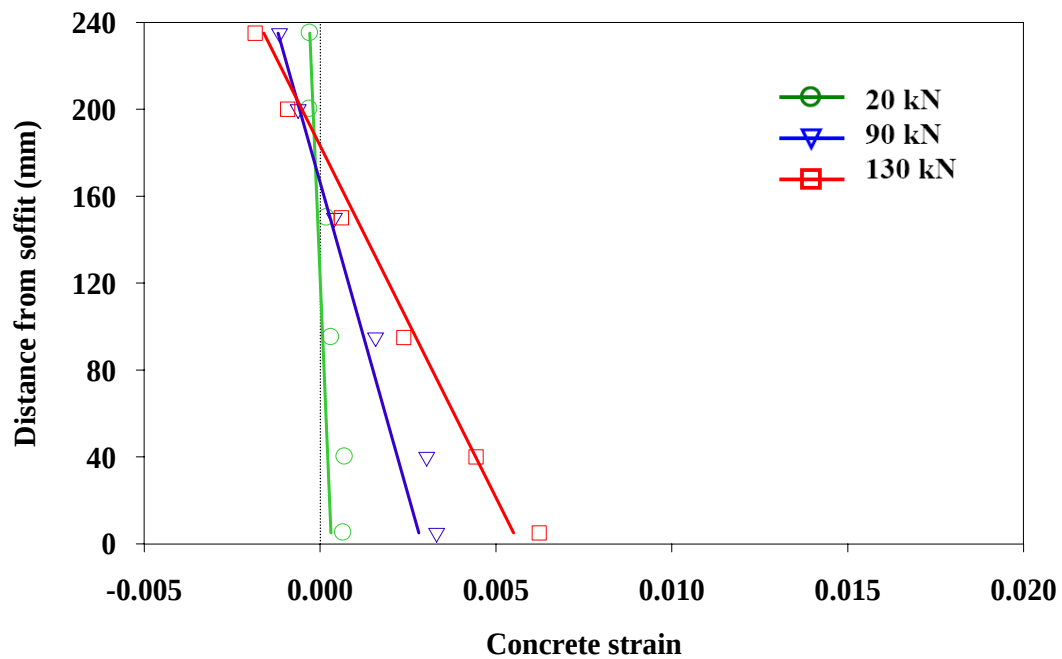


Figure 7.12: Concrete strain distribution for beam S12ME

Generally, it can be observed that as the load increases the neutral axis shifts upward slightly. This can be attributed to crack initiation within the tensile zone of concrete cross section.

As shown in Figure 7.8, the average value of the maximum tension strain is 0.0120 mm/mm, while it is equal to 0.0050 mm/mm for strengthened beam specimen S10M1 (Figure 7.9), and it is equal to 0.0031 mm/mm for strengthened beam specimen S10M2 as shown in Figure 7.10. These figures explain the benefit achieved from CFRP strengthening and its effect on the behavior of concrete in tension zone. The same conclusion can be observed in Figures 7.11 and 7.12, which is related with the beam specimen U12ME and S12ME, respectively.

7.3.4 Ultimate Load and Failure Modes

The numerical results obtained for the beam specimens at failure are shown in Table 7.3. The numerical failure modes for the tested beam specimens are also presented in the same table.

Table 7.3: Numerical failure loads for the tested beam

Specimen	Ultimate Load (kN)			Increase in numerical ultimate load, %
	Experimental	Numerical	Num./Exp.	
U10M0	52	63	1.21	N/A
S10M1	70	85	1.21	35
S10M2	90	106	1.18	68
U12ME	89	85	0.96	N/A
S12ME	135	146	1.08	72

The ultimate load capacity increased by 35 percent in the beam specimen strengthened with one CFRP plate (S10M1) and increased by 68 percent in the beam specimen strengthened with two CFRP plate (S10M2). While the ultimate load capacity increased by 72 percent with beam specimen S12ME when compared with the corresponding control beam specimen.

The two control beam specimens U10M0 and U12ME are failed with the typical flexural failure mode (yielding of steel followed by crushing of concrete), while all the strengthened beam specimens (S10M1, S10M2, and S12ME) are failed with the interfacial debonding failure mode in the constant moment region. These mode failures are coincide with that obtained in the experimental results.

CHAPTER EIGHT

CONCLUSIONS AND FUTURE RESEARCH

8.1 General

The main objective of this research is to study the flexural behavior of externally strengthened RC beams with carbon fiber reinforced polymer (CFRP) plates. This research includes an experimental program as well as a nonlinear finite element model has been developed in order to predict the flexural behavior of these tested beams.

In this chapter, the conclusions drawn from the experimental, analytical results are described. The limitations of the present work and suggestions for future work are also presented.

8.2 Conclusions

Based on the results obtained from the experimental work and by finite element analysis for the externally strengthened RC beams with CFRP plates, the following conclusions can be drawn:

1. For the tested beams externally strengthened with bottom FRP plates subjected to monotonic loading up to failure, the ultimate flexural loads are increased by 35 to 73 percent.
2. The presence of CFRP plates on the tension face restrained cracking propagation which caused increase in load carrying capacities prior to and beyond the first cracking. The cracking loads are found to increase by 25 to 67 percent.
3. The presence of CFRP laminates at the bottom tension zone surface reduced the tensile concrete strains, and this reduction is reflected to strains in the bottom tension steel bar reinforcement (i.e., reducing the tension steel bar strains), and that means increasing

the tension strength and some tensile stresses would be carried by CFRP laminates.

4. The structural ductility of the CFRP strengthened RC beams is very low when compared with the corresponding control specimen. The reduction in ductility ratio was about 30 to 67 percent based on deflection.
5. The Plate End Interfacial (PEI) debonding and/or Concrete Cover Separation (CCS) can be easily prevented by using CFRP U-jackets at the CFRP plate ends. The instillation of such anchors at the plate ends is very convenient in practice.
6. Under repeated loading conditions, the stiffness and load capacity of CFRP strengthened beams undergoes a progressive reduction compared to monotonic loading case. The reduction in ultimate load capacity is about 11 percent.
7. No evidence of debonding of the CFRP plates was observed until after yielding of the reinforcement. The formation of secondary cracks adjacent to existing flexural cracks is an indication of initiation of debonding of the CFRP plates.
8. The debonding in beam specimens is very sudden and the only indication of incipient failure is few popping sounds as the debonding cracks propagated quickly to the end of the plate. The clear message of the experimental results is that the premature and brittle nature of debonding failure may reduce the level of safety of the strengthened RC beam by decreasing its ductility.
9. At debonding failure, the extreme fiber of the compressive concrete may not reach its ultimate strain. The concrete strength in the compression zone cannot be fully developed in this case and the compression force resisted by the concrete will be overestimated if

the compressive stress block used in the design of conventional RC beams is adopted.

10. The responses of concrete beams reinforced with lightly steel ratios are more sensitive to the application of CFRP strengthening.
11. The design equation suggested by ACI Committee 440 (2002) is evaluated based on the comparison made along with the present experimental results. Generally, it is found that this equation is unconservative and needs further refinement.
12. The finite element model used in the present study is able to simulate the strengthened reinforced concrete beams with CFRP laminates. The cracking loads, crack patterns, and ultimate loads predicted are very close to that measured during the experimental testing.

8.3 Recommendations for Future Research

The debonding failure in reinforced concrete members strengthened with externally bonded CFRP requires further investigations. The areas, which are to be of particular importance, are listed here in:

1. Further studies should focus on expanding the experimental database of reinforced concrete beams strengthened with CFRP through full scale experimental tests and on long-term performance.
2. Further tests covering a range of beam geometry different CFRP strengthening schemes under the combined effect of bending, shear and torsion are required.

3. Bond durability in strengthening systems may be thoroughly investigated both experimentally and theoretically in order to properly account for environmental bond degradation.
4. Investigate the structural behavior of prestressed reinforced concrete beams strengthened with FRP laminates. The extension of the computer program to prestress concrete is required to make it applicable to wider range of FRP strengthened members.
5. The critical portion of the strengthened RC beam is the plate-to-concrete interface. This should be examined extensively including use of different FRP composites, epoxies, surface preparations, and application processes.
6. Time-dependent effects are urgently needed to be incorporated in the present study. These effects should include concrete shrinkage, creep and aging of concrete.
7. Behavior of FRP strengthened reinforced concrete beams subjected to dynamic and impact loading.

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جمهورية العراق
وزارة التعليم العالي والبحث العلمي
جامعة بابل
كلية الهندسة
قسم الهندسة المدنية

سلوك العتبات الخرسانية المسلحة المقواة بصفائح من ألياف الكاربون البوليمرية

رسالة

مقدمة إلى كلية الهندسة في جامعة بابل
كجزء من متطلبات نيل درجة دكتوراه فلسفة في الهندسة المدنية
(أنشاءات)

من قبل

خالد كريم شدهان

بكالوريوس هندسة مدنية ١٩٩٧

ماجستير هندسة أنشائية ٢٠٠٠

إشراف

الأستاذ المساعد الدكتور عمار ياسر علي

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كانون الثاني ٢٠٠٨

الخلاصة

أن الغرض من هذا البحث هو التحري عن تصرف وأداء العتبات الخرسانية المسلحة و المقواة بألياف الكربون البوليمرية وذلك من خلال تقديم دراسة تحليلية ومختبرية للموضوع.

يتضمن العمل المختبري أعداد وفحص ستة عتبات خرسانية مسلحة، أربعة منها مقواة باستخدام صفائح البوليمر المسلحة بألياف الكربون، تتضمن متغيرات الفحص كمية حديد التسليح وكمية التسليح الخارجي بصفائح البوليمر المسلحة بألياف الكربون بالإضافة الى نوع التحميل سواء كان تزايديا أو دوريا، ومما تجدر الإشارة إليه أن كل العتبات قد تم فحصها تحت ظروف الأسناد البسيط.

أظهرت النتائج المختبرية زيادة مهمة في التحمل الأقصى تراوحت بين ٣٥% الى ٧٣% في العتبات المقواة بالمقارنة مع نماذج السيطرة، كذلك فإن العتبات المقواة أظهرت هطولا أقل عند نفس مستوى التحميل مقارنة مع عتبات السيطرة، إضافة الى ذلك فإن وجود صفائح البوليمر المسلحة بألياف الكربون في الوجه المعرض للشد سوف يقيد اتساع التشققات وبالتالي يسبب زيادة التحمل الأقصى قبل وبعد حدوث التشقق الأول. أن حمل التشقق الأول ازداد بمقدار يتراوح بين ٢٥% إلى ٦٧%. أن مقدار النقصان بالمطيلية هو بحدود ٣٠% إلى ٦٧% بالاعتماد على الهطول.

المواصفات التصميمية لمعهد الخرسانة الأمريكي توصي باستخدام حدود للأنفعال في صفائح البوليمر المسلحة عن طريق استخدام معامل الربط لمنع حدوث الفشل بالربط لصفائح البوليمر المسلحة ، الدراسة قامت بتقييم المعادلة التصميمية المقترحة من قبل المعهد الأمريكي للخرسانة بمقارنة النتائج المختبرية الحالية.

العمل التحليلي قدم المعادلات التقاضلية الحاكمة للمسألة بالإضافة الى طريقة العناصر المحددة، والتي تم اشتقاقها من مبدأ الطاقة الكامنة. يتكون العنصر المقترح من عنصر عتبة ذو عقدتين زائدا عنصر قضيب ذو عقدتين، حتى تسمح بالأنزلاق بالنسبة للعتب الخرساني، درجات الحرية العقدية سواء كانت في العتب أو العضو

المحوري مختلفة عن بعضها البعض, لتسمح بالانزلاق, حيث تم فرض مجالات
أزاحة منفصلة لكل من العتب الخرساني والعضو المحوري وبالتالي فإن انزلاق
الربط قد تم اشتقاقه من التوافقية, كما تم استخدام متعددة حدود من الدرجة الثالثة
للطول الشاقولي, بينما استخدمت متعددة حدود من الدرجة الأولى للأزاحات
المحورية في العتب الخرساني والعنصر المحوري, وبالتالي فإن هذه الفرضيات
أدت الى أن يكون توزيع الربط الأنزلاقي من الدرجة الثانية.

تم إنشاء برنامج حاسبة لتتبع علاقة الحمل-الأزاحة باستخدام لغة الفورتران.
ولغرض إثبات صحة النموذج المقدم فقد تم مقارنة نتائجه مع نتائج الفحوصات
المختبرية, النتائج المستحصلة من تحليل العناصر المحددة أظهر موافقة جيدة مع
النتائج المختبرية.