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RETROFIT OF REINFORCED CONCRETE COLUMNS BY COMPOSITE JACKETING

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DECLARATION

I certify that all the material in this thesis that is not my own work has been identified, and that no material is included for which a degree has previously been conferred on me.
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We certify that we have read the present work and that in our opinion it is fully adequate in scope and quality as thesis towards the partial fulfillment of the Master Degree requirements in

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ABSTRACT

The strengthening of existing reinforced concrete columns using steel- or fiber-reinforced polymer (FRP) jacketing is based on a well-established fact that lateral confinement of concrete can substantially enhance its axial compressive strength. In recent years, external confinement of concrete using FRP composites has emerged as a popular method of column retrofit; many recent studies have been conducted on the compressive strength of FRP-confined concrete. These studies showed that FRP-confined concrete behaves differently from steel-confined concrete. Consequently, various models for predicting the compressive strength have been developed specifically for FRP-confined concrete, in the context of column strengthening.

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CHAPTER 1

INTRODUCTION

1.1General

Confinement is generally applied to members in compression, with the aim of enhancing their load carrying capacity or, in case of seismic upgrading, to increase their ductility. When concrete cylinder is subject to axial compressive stress, it expands laterally. For confined cylinder, this lateral expansion is resisted by the lateral pressure induced by the jacket which is loaded in tension in the circumferential (hoop) direction. The confinement action enhances the concrete strength, ductility and in additional, prevents slippage and buckling of longitudinal reinforcement.

Traditional confinement techniques rely on the steel hoops or steel jacket for upgrading. In recent years, external confinement of concrete using FRP composites has emerged as a popular method of column retrofit, particularly for circular columns and many recent studies have been conducted on the compressive strength and stress-strain behavior of FRP-confined concrete. Different from steel behavior that shows a large ductility at yield the FRP has only linear behavior up to failure therefore it exerts an increasing pressure on concrete core while steel jacket reaches yield and then from this point it exerts a constant lateral pressure. Thus the confinement provided by FRP jacket to concrete core is active, rather than passive, as the confining pressure from the jacket increases with the axial loading of column. Failure of FRP confined concrete generally occurs when the FRP jacket reaches its hoop rupture strength.

1.2 Objectives

The design of columns wrapped with Fiber Reinforced Polymers necessitates a precise modeling of stress–strain behavior for confined concrete to compute the overall strength of columns. The actually used models for FRP-confined concrete are based on the concept of steel-confined concrete sections. Divided into two categories, design oriented and analysis oriented, those models are mainly affected by corner radius of cross-section, compressive strength of concrete, aspect ratio, thickness of steel. Or the behavior of FRP differs from steel, consequently the confined concrete behavior changes accordingly. This thesis presents a semi-analytical method developed by the author based on Mander equation and implemented to predict the behavior of FRP confined rectangular columns. Increase their axial load-carrying capacity. Axial strengthening for non-slender (i.e., short) reinforced concrete columns. Combined axial and flexural strengthening of short eccentrically loaded reinforced concrete columns will increase their axial and flexural capacity. An axial load-bending moment (*P–M*) strength interaction diagram can be constructed for an FRP-strengthened reinforced concrete column in a fashion similar to that of a non-strengthened column.

1.3 Thesis arrangement

In addition to this chapter six more chapters form this thesis:

Chapter 2, gives a general background to areas related to the topic from some properties and forms of FRP and some basic of reinforced concrete concepts.

Chapter 3, shows different models of confining RC columns and the model developed in this study and procedures used to construct a full stress-strain diagram for confined concrete.

Chapter 4, the relation between the load (P), moment (M) and the curvature (Φ) and procedures to construct a P-M diagram and M- Φ diagram for confined concrete.

Chapter 5, deals with columns retrofits techniques and seismic retrofit design procedure for column using FRP.

Chapter 6, shows results of a numerical analysis for confined and unconfined concrete and the effect of major variable on confinement level.

Chapter 7, summarizes the work done with final conclusion and states points for discussion and recommendation for further works.

CHAPTER 2

BACKGROUND

2.1 Introduction

Fiber-reinforced polymer (FRP) is a common term used by civil engineering community for high-strength composites. Composites have been used by the space and aerospace communities for over six decades and the use of composites by the civil engineering community spans about three decades. In the composite system, the strength and the stiffness are primarily derived from fibers, and the matrix binds the fibers together to form structural and non-structural components. Composites are known for their high specific strength, high stiffness and corrosion resistance. Repair and retrofit are still the predominant areas where FRPs are used in the civil engineering application. The field is relatively young and, therefore, there is considerable ongoing research in this area. American Concrete Institute Technical Committee published 440 documents that are excellent sources for information.

Fiber reinforced polymer (FRP) is a composite made of high-strength fibers and a matrix for binding these fibers to fabricate structural shapes. Common fiber types include aramid, carbon, glass, and high-strength steel; common matrices are epoxies and esters. Inorganic matrices have also been evaluated for use in fire-resistant composites. FRP systems have significant advantages over classical structural materials such as steel that includes low weight, corrosion resistance, and ease of application

Originally developed for aircraft, these composites have been used successfully in a variety of structural applications such as aircraft fuselages, ship hulls, cargo containers, high-speed trains, and turbine blades (Feichtinger 1988, Kim 1972, Thomsen and Vinson 2000). FRP is particularly suitable for structural repair and rehabilitation of reinforced and pre-stressed concrete elements. The low weight reduces both the duration and cost of construction since heavy equipment is not needed for the rehabilitation. The composites can be applied as a thin plate or layer by layer.

Even though use of FRP for civil engineering structures only started in the 1980s, a large number of projects have been carried out to demonstrate the use of this composite in the rehabilitation of reinforced and pre-stressed concrete structures (Hag-Elsafi et al. 2004, Mufti 2003, Täljsten 2003). The composite has been successfully used to retrofit all basic structural components, namely: beams, columns, slabs and walls. In addition, strengthening schemes have been carried out for unique applications such as storage tanks and chimneys. These advanced materials may be applied to the existing structures to increase one or several of the following properties:

• Axial, flexural, or shear load capacities;

• Ductility for seismic performance;

• Durability against adverse environmental effects;

• Fatigue limit;

• Stiffness to reduce deflections under service and design loads (Buyukozturk et al. 2004, Täljsten and Elfgren 2000)

Hand-layup fabrics and sheets have been used in thousands of projects in the United States and around the world Figure (2.1a). Since the 1994 Northridge earthquake in California, hundreds of highway columns have been retrofitted in California and neighboring states. Wrapping of circular columns with FRP is most effective; however, rectangular columns have also been retrofitted successfully for both strength and ductility using FRP wraps. FRP retrofitting of a highway column is shown in Figure (2.2b).

The two major components of a composite are high-strength fibers and matrix that binds these fibers together to form a composite-structural component. The fibers provide strength and stiffness while the matrix (resin) transfers of stresses and strains between the fibers. To obtain full composite action, the fiber surfaces should be completely coated (wetted) with matrix. Two or more fiber types can be combined to obtain specific composite property that is not possible to obtain using a single fiber type. For example, the modulus, strength, and fatigue performance of glass-reinforced polymers (GRP) can be enhanced by adding carbon fibers. Similarly, the resistance energy of carbon fiber reinforced polymers (CFRP) can be increased by the addition of glass or aramid fibers.

The optimized performance that hybrid composite materials offer has led to their widespread growth throughout the world (Hancox 1981, Shan and Liao 2002). In recent years, hybrid composites have found uses in a number of applications such as abrasive resistant coatings, contact lens, sensors, optically active films, membranes and absorbents (Cornelius and Marand 2002).

2.2 Types of Fibers

The primary role of fiber is to resist the major portion of the load acting on the composite system. Depending on the matrix type and fiber configuration, the fiber volume fraction ranges from 30 to 75%. Strength and stiffness properties of commercially available fibers cover a large spectrum and consequently, the properties of the resulting composite have a considerable variation (Mallick 1993). Typical fiber reinforcements used in the composite industry are glass (E- and S-glass), carbon and aramid .The properties and characteristics of these fibers as well as other fiber types such as basalt are presented in the subsequent sections.

2.2.1 Glass fibers

They are the most common reinforcing fibers used in composites. Major advantages of glass fibers include low cost, high tensile strength, chemical resistance and high temperature resistance. The disadvantages are low tensile modulus, sensitivity to abrasion while handling, relatively low fatigue resistance and brittleness.

Glass fibers are produced by fusing silicates with silica or with potash, lime or various metallic oxides. The molten mass is passed through micro-fine bushings and rapidly cooled to produce glass fiber filaments ranging in diameter from 5 to 24 μm. These filaments are then drawn together into closely packed strands or loosely packed roving. During this process, the fibers are frequently covered with a coating, known as sizing, to minimize abrasion-related degradation of the filaments (Miller 1987; Gurit Composite Technologies 2008).

2.2.2 Carbon fibers

They offer the highest modulus of elasticity among all reinforcing fibers. Among the advantages of carbon fibers are their exceptionally high tensile-strength-to-weight ratios as well as high tensile-modulus-to-weight ratios. In addition, carbon fibers have high fatigue strengths and very low coefficient of linear thermal expansion and in some cases, even negative thermal expansion. This feature provides dimensional stability, which allows the composite to achieve near zero expansion to temperatures as high as 570 °F (300 °C) in critical structures such as spacecraft antennae. If protected from oxidation, carbon fibers can withstand temperatures as high as 3600 °F (2000 °C). Above this temperature, they will thermally decompose. Carbon fibers are chemically inert and not susceptible to corrosion or oxidation at temperatures below 750°F (400°C).

Carbon fibers possess high electrical conductivity which is quite advantageous to the aircraft designers who are concerned with the ability of an aircraft to tolerate lightning strikes. However, this characteristic poses a severe challenge to the carbon textile manufacturer since carbon fiber debris generated during weaving may cause –shorting” or electric shocks in unprotected electrical machinery. Other key disadvantages are their low impact resistance and high cost (Amateau 2003; Mallick 1993).

2.2.3 Aramid fibers

Aramid fiber is a synthetic organic polymer fiber (an aromatic polyamide) produced by spinning a solid fiber from a liquid chemical blend. Aramid fiber is bright golden yellow and is commonly known as Kevlar®, its commercial trade name. These fibers have the lowest specific gravity and the highest tensile strength-to-weight ratio among the reinforcing fibers used today. They are 43% lighter than glass and approximately 20% lighter than most carbon fibers. In addition to high strength, the fibers also offer good resistance to abrasion and impact, as well as chemical and thermal degradation. Major drawbacks of these fibers include low compressive strength, degradation when exposed to ultraviolet light and considerable difficulty in machining and cutting (Smith 1996).

2.3 Properties of FRP Materials

This section reviews the basic mechanical properties of FRP materials. Density, coefficient of thermal expansion, tension and compression behavior and durability of FRP materials are briefly summarized.

Typical Densities of FRP Materials (kN/m^3)

Steel	GFRP	CFRP	AFRP
79	12-21	15-16	12-15

Typical Coefficients of Thermal of Expansion for FRP Materials x 10^{-6}

Direction	GFRP	CFRP	AFRP
Longitudinal	6 to 10	-1 to 0	1.2-1.5
Transverse	19 to 23	22 to 50	60 to 80

Typical Tensile Properties for Fibers Used in FRP Systems

Fiber Type	Elastic modulus (GPa)	Ultimate strength (N/mm^2)	Rupture strain (%)
Carbon			
General purpose	220-240	2050-3790	1.2
High strength	220-240	3790-4820	1.4
Ultra high strength	220-240	4820-6200	1.5
High modulus	340-520	1720-3100	0.5
Ultra high modulus	520-690	1380-2400	0.2
Glass			
E-glass	69-72	1860-2680	4.5
S-glass	86-90	3440-4140	5.4
Aramid			
General purpose	69-83	3440-4140	2.5
High performance	110-124	3440-4140	1.6

Table 2.1 Mechanical properties of different types of FRP

2.3.1 Density

FRP materials have densities ranging from 12 to 21 kN/m^3, which is four to six times lower than that of steel (Table2.1). The reduced density leads to lower transportation costs, reduces added dead load on the structure and can ease handling of the materials in the project site.

2.3.2 Coefficient of Thermal Expansion

The coefficients of thermal expansion of unidirectional FRP materials differ in the longitudinal and transverse directions, depending on the types of fiber, resin and volume fraction of fiber. Table lists the longitudinal and transverse coefficient of thermal expansion for typical unidirectional FRP materials. Note that a negative coefficient of thermal expansion indicated that the material contracts with increased temperature and expands with decreased temperature. Steel has an isotropic coefficient of thermal expansion of $11.7x10^{-6}/C$.

2.3.3 Tensile Behavior

When loaded in direct tension, FRP materials do not exhibit any plastic behavior (yielding) before rupture. The tensile behavior of FRP materials consisting of one type of fiber material is characterized by a linearly elastic stress-strain relationship until failure, which is sudden and can be catastrophic.

The tensile strength and stiffness of an FRP material depends on several factors. Because the fibers are the main load-carrying constituent, the type of fiber its orientation and its quantity primarily govern the tensile properties of the composite material. The tensile properties of some commercially available FRP strengthening system are given in Table (2.2) and shown schematically in Figure (2.2) .The key mechanical property of the FRP jacket system to provide confinement are the elastic jacket modulus in the hoop direction, the ultimate unidirectional tensile strength and the ultimate unidirectional tension failure strain.

2.3.4 Compressive Behavior

Compressive strengths ratio of 55, 78 and 20% of the tensile strength have been reported for GFRP, CFRP and AFRP, respectively (Wu 1990). In general, compressive strengths are higher for materials with higher tensile strengths, except in the case of AFRP where the fibers exhibit nonlinear behavior in compression at a relatively low level of stress.

The compressive modulus of elasticity is usually smaller than the tensile of elasticity for FRP materials. The compressive modulus of elasticity is approximately 80% for GFRP, 85% for CFRP, and 100 % for AFRP of the tensile modulus of elasticity for the same product (Ehsani 1993).

2.3.5 Durability

Many FRP systems exhibit reduced mechanical properties after exposure to certain environmental factors, including temperature, humidity and chemical exposure. The exposure environment, duration of the exposure, resin type and formulation, fiber type, and resin-curing method are also important factors that influence the extent of the reduction in mechanical properties table (2.2)

Advantages	Disadvantages
• Electromagnetic neutrality • High strength to weight ratio • Corrosion resistance • High Fatigue endurance (for Carbon and aramid fibers)	• Lack of ductility • Low transverse strength • Low modulus of elasticity for glass and aramid fibers

Table 2.2 Advantages and disadvantages of FRP

2.4 Forms of FRP

All the fiber types are available in a variety of forms to serve a wide range of processes and end-product requirements. Fibers supplied as reinforcement include continuous spools of tow (carbon), roving (glass), milled fiber, chopped strands, chopped or thermo-formable mat, and woven fabrics. Reinforcement materials can be tailored with unique fiber architectures and be performed (shaped) depending on the product requirements and manufacturing process (Figure 2.3).

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2.4.1 Filament

A filament is an individual fiber as drawn during processing (drawing and spinning). It can be considered as the smallest unit of fiber reinforcement. Depending on the material, the filament diameter can range from 1 to 25 μm. It is standard practice to use a specific alphabet designation when referring to a specific filament diameter. Very fine fibers typically used in textile applications range from AA to G. Conventional composite reinforcements consist of filaments with diameters ranging from G to U. Individual filaments are rarely used as reinforcement, they are typically gathered into strands of fibers (either continuous or chopped) for use in fibrous composites (Watson and Raghupathi 1987).

2.4.1.2 Yarn

A yarn is a generic term for closely associated bundle of twisted filaments, continuous strand of fibers or strands in a form suitable for knitting, weaving or otherwise to form a textile fabric.

2.4.1.3 Tow

A tow is an untwisted bundle of continuous filaments. Also known as continuous strand or an “end,” it is commonly used when referring to manufactured fibers especially carbon. Tow designations are based upon the number of thousands of fibers. For example a “12k HMC Tow” refers to a high modulus carbon tow consisting of 12,000 fibers.

2.4.1.4 Roving

Unlike yarns, a roving is a *loosely* assembled bundle of untwisted parallel filaments or strands. Each filament diameter in a roving is the same and is usually 13–24 μm. Rovings have varying weights and the Tex range is usually between 300 and 4800.

2.4.1.5 Chopped strands

Chopped strands are produced by cutting continuous strands into short lengths. The ability of the individual filaments to remain together during or after the cutting process depends on the type and amount of sizing applied during manufacturing. Strands of high integrity that remain together are referred to as being “hard” while those that separate more easily are called “soft.”

2.4.1.6 Milled fibers

Milled fibers are produced by grinding continuous strands in a hammer mill into very short lengths. Fiber lengths typically range from particulates to screen opening with dimensions ranging from 1 to 3 mm. They are primarily used in the plastics industry as inexpensive filler. Although they provide increased stiffness and dimensional stability to plastics, they do not provide significant reinforcement value. Typical applications include reinforced reaction injection molding (RRIM), phenolics and potting compounds.

2.4.1.7 Fiber mats

A fiber mat, also known as “omni-directional reinforcement” is randomly oriented fibers together with a small amount of adhesive binder. Fiber mats can be used for hand lay-up as prefabricated mat or for the spray-up process as chopped strand mat.

Three typical types mat reinforcements are:

1. Randomly oriented chopped filaments (chopped strand mat)
2. Swirled filaments loosely held together with binder (continuous strand mat)
3. Very thin mats of highly filamentized glass (surfacing mat).

2.4.2 Fabrics

A fabric is defined as a manufactured assembly of long fibers of carbon, aramid, glass, other fibers or a combination of these to produce a flat sheet of one or more layers of fibers. These layers are held together either by mechanical interlocking of the fibers themselves or with a secondary material to bind these fibers together and hold them in place, giving the assembly sufficient integrity for handling. Consequently, fabrics are the preferred choice of reinforcement since the fibers are in a more convenient format for the design engineer and fabricator. Fabric types are categorized by the orientation of the fibers used, and by the various construction methods used to hold the fibers together.

2.4.2.1 Unidirectional fabrics

A fabric made with a weave pattern designed for strength in only one direction is termed unidirectional. The pick count of a unidirectional fabric is very small and most of the yarns run in the warp direction.

2.4.2.2 Multi-axial fabrics

Multi-axial fabrics, also known as non-woven, non-crimped, stitched or knitted, have optimized strength properties because of the fiber architecture. Stitched fabrics consist of several layers of unidirectional fiber bundles held together by a nonstructural stitching thread, usually polyester. The fibers in each layer can be oriented along any combination of axes between 0 and 90°. Multiple orientations of fiber layers provide a quasi-isotropic reinforcement. The entire fabric may be made of a single material or different materials can be used in each layer. A layer of mat may also be incorporated into the construction.

2.4.3 Matrix types

The primary functions of the matrix (or resin) in a composite are:

1. Transfer stresses between fibers.
2. Provide a barrier against the environment.
3. Protect the surface of fibers from mechanical abrasion.

The matrix plays a major role in a composite and influences the inter-laminar shear as well as the in-plane shear properties of the material. The interaction between fibers and matrix is important when designing damage-tolerant structures. Furthermore, the ability to manufacture the composite and defects within it, depends strongly on the matrix's physical and thermal characteristics such as viscosity, melting point and curing temperature (Mallick 1993). There are generally two types of matrix: organic and inorganic.

2.5 Strength

The actual strength is the maximum load that can be sustained by a structural member before failure. In other words, the actual strength is the ability of a structural member to resist the internal actions induced by load until failure. The performance of a reinforced concrete member subjected to seismic action can be evaluated based on the degradation of strength through loading cycles.

For designing purposes, the computed strength is used for proportioning reinforced concrete members. One should distinct between different strength definitions such as nominal, required and design strengths. On the other hand, the required strength is the strength of a member required to resist factored loads. The strength design method requires that nominal strength be reduced by specified strength reduction factors, i.e. design strengths, equal to or exceed the factored load effect, i.e. required strengths.

($\sigma_{\text{actual}} <= \sigma_{\text{allowable}}$)

2.6Stiffness

Stiffness is a property used to quantify and control the deformation under the action of certain force. Relationships for computing stiffness are readily established from first principles of structural mechanics using members geometry, material properties and members end conditions. In reinforced concrete structures, these relationships are not quite simple but can be achieved using some assumptions. If a serviceability criterion is to be satisfied, the extent and influence of cracking in members and contribution of concrete in tension must be considered in calculating the stiffness.

A typical relationship between load and displacement, describing the response of a reinforced concrete element subjected to monotonically increasing load, is shown in Figure (2.4). For design purposes, the real response may be approximated to bilinear relationship, idealized response, where P_y, defines the yield strength of the member. The slope of the idealized linear elastic response, $K=P_y / \Delta_y$, is used to quantify stiffness. This should be based on the effective secant stiffness of the real load-displacement curve at a load of $0.75P_y$ as shown in the Figure2.4.

Stiffness will be of concern when estimating response for serviceability limit state such as checking of typical inter-story drift. Also the resistance decay of a reinforced concrete member subjected to cyclic loading can be evaluated in terms of the degradation of stiffness.

2.7 DUCTILITY

Ductility is an important consideration in design of all structures, particularly when they are subjected to overloads. In view of its significance, a discussion on the basic concepts of ductility and an analogous concept, deformability, is presented here after.
All materials deform under loads. These deformations can be elastic or plastic, small or large. A material that is capable of undergoing a large amount of plastic deformation is said to be *ductile*. Examples of ductile materials include mild steel, aluminum, copper, magnesium, lead, molybdenum, nickel, brass, bronze, nylon, Teflon and many others alloys. A material that is capable of undergoing very little plastic deformation before rupture is said to be *brittle*. Ordinary glass is a nearly ideal brittle material because it exhibits almost no ductility whatsoever (i.e. no plastic deformation before its failure load).

After attaining certain levels of deformations, all materials fail. A structural material is considered to have failed when it becomes incapable of performing its design function, either through fracture as in brittle materials or by excessive deformation as in ductile materials. Ductile materials are capable of absorbing large amounts of energy prior to failure; they are capable of undergoing large deformations before rupture. Because elastic deformations of materials are generally small, the usual measure of ductility (or brittleness) is the total percent elongation up to rupture of a tensile test specimen.

Ductility is an important attribute of materials in that visible deformations can occur if the loads become too large, thus providing an opportunity to take remedial action before fracture occurs. In the context of structural members as slabs, beams and columns, the inherent ductility, large deformations can become visible, giving ample warning of an impending structural failure. This is particularly important for the performance of structures in high seismic regions where structures must undergo large cyclic displacements (often inelastic) without structural collapse. Satisfactory structural response under such loading conditions relies on the capacity of a structure (or structural members) to displace in-elastically through several cycles of response without a significant degradation of strength or stiffness is a quality termed *ductility* (Pristley et al. 1996).

Because of its significance, ductility must be quantified. The usual practice is to express ductility as a *ductility factor* or a *ductility index*. Referenced to displacements, the ductility factor is often mathematically defined as the ratio of maximum displacement to displacement at yield [Pristley et al. 1996]:

$$\mu = \frac{m}{y} \tag{2.1}$$

where

μ= the ductility factor (the subscript refers to displacement)

Δ_m = the maximum displacement (inelastic response)

Δ_y = the displacement at yield

The maximum displacement quantity, Δ_m (inelastic response) can be any prescribed value greater than yield displacement (Δ_y) so that the ductility factor is always greater than unity. Note that the key word here is inelastic response, because $\Delta_m > \Delta_y$. For example, for seismic design considerations Δ_m can be defined as the maximum displacement (e.g., post-yielding, post-buckling) expected under the design-level earthquake. For a different design consideration, such as in concrete structures, the maximum displacement can be the displacement that may be attained at the ultimate force level, Δ_u, and the corresponding ductility factor can be defined as

$$\mu = \frac{u}{y} \tag{2.2}$$

Although Equation (2.1) and (2.2) define ductility factors in terms of displacements in the context of overall structural response (structure or structural system displacement ductility factor), they can also be expressed to define a member response to loads (member displacement ductility factor). In general, ductility can be defined in a number of different ways depending on design considerations. Measures of ductility can be expressed as displacement ductility, rotational (curvature being the rotation over the depth of a section) ductility and strain ductility to quantify the structural response under maximum loads (e.g., ultimate loads or design-level earthquake loads), which are useful indicators of structural response (Figure 2.5).

The relationship between the curvature and displacement ductility factors depends on the structural geometry and is important for determining safe levels of inelastic displacements for the structure as a whole. A large number of concrete structures (buildings and bridges) in California either collapsed completely or were significantly damaged during earthquakes in 1933, 1971, 1987, 1989, and 1994; the main reason was a lack of ductility. Providing sufficient ductility or improving ductility of structures that either lack or have a little of it is of utmost significance to structural engineers. A comprehensive discussion ductility factors has been presented by Pristley, Seible, and Calvi [1996].

The ductility factor (ductility index) of concrete beams reinforced with steel bars (member ductility factor) can be defined as the ratio of deformation at ultimate to that at yield. From a design perspective, the ductility index of a concrete beam Reinforced with steel bars provides a measure of the energy absorption capability Jaeger et al. 1997, Naaman and Jeong 1995].

Because deflections, curvatures, and rotations in a beam are all proportional to external moment, the ductility factor for materials such as metals that exhibit a post yielding plateau can be expressed in terms of the ratio of any of these quantities. Thus,

$$\text{Ductility factor} = \frac{\text{Deflection (or curvature,or rotation) at ultimate}}{\text{Deflection (or curvature,or rotation) at steel yield}} \tag{2.3}$$

2.8 Plastic hinges

The plastic hinge is a region of plasticity over which inelastic rotation will occur without significant strength loss. Reinforcing detailing requirement must be met over this region to ensure inelastic rotation capacity. At maximum flexural response of a column, curvature over the plastic hinge region is the summation of yield curvature and plastic curvature, which is substantially larger than the yield curvature. For a simple structural element, such as a cantilever beam, the distribution of curvature and plasticity can be illustrated in Figure (2.6).

Plastic hinges are an extension of the ductile design concept in building seismically resistant structures. Energy is dissipated through the plastic deformation of specific zones at the end of a member without collapsing the rest of the structure. In conventional reinforced concrete columns, this plastic hinge action can result in damage and permanent strain in the column, necessitating replacement of the entire member and possibly the entire structure. However, through the use of specially designed plastic hinge zones, damage due to large seismic displacements can be localized and repaired after an earthquake.

In reinforced concrete columns, the detailed plastic hinge consists of a weakened portion of the column near the top and bottom where the longitudinal reinforcement is decreased, allowing yielding in this zone before the rest of the column is damaged. These specially weakened steel bars are termed fuse-bars since they are designed to yield and thus protect the rest of the column during repeated ground motion

2.9 Rehabilitation

Structural rehabilitation represents an important aspect of the construction industry and its significance is increasing. Several methods are available, each with different advantages and handicaps. However, little information is available and insufficient code guidelines are accessible. In fact, most repair and strengthening designs are based on the assessment of engineers only and, often, empirical knowledge and current practice have an important role in the decisions to be made.

Rehabilitation and strengthening of reinforced concrete structures is a dynamically growing division of structural engineering. In recent years an increased application of new repair and strengthening systems of reinforced concrete load-carrying structures has been noted.

The problem of strengthening the reinforced concrete structures appeared for the first time when their proper function was modified or they were used in a different manner than previously planned. Assumptions made in the design are closely connected with a specific function of the structure. The designers of the existing reinforced load-carrying structures constructed many years ago could not predict their use in practice and determine all deterioration effects produced by external factors during their service. It was not rare to find that in this way some structural members could be deteriorated or even damaged. In most cases the increased dead and live loading that should be safely carried by the structures, as well as their poor technical condition necessitate strengthening procedures.

Repairing a RC element may be defined as an attempt to restore the original strength and stiffness of a damaged or deteriorated RC element. Ramirez et al.(1993) while Strengthening a RC element is defined as an intervention to increase the original strength and stiffness of the RC element. In the case of an undamaged element, there cannot be, by definition, a need to repair the element. In this case, there can only be a need to strengthen this element, due to one or more causes previously referred to.

The strengthening of concrete structures with externally bonded reinforcement is generally done using either steel plates or CFRP laminates. Steel plates have been used for many years due to their simplicity in handling, applying and to their effectiveness for strengthening. The properties and behavior of steel-concrete structures are well known. Steel plates are very effective to be used as bending reinforcement. The high tensile strength and stiffness lead to an increase in bending capacity and a reduction of the deformations. Steel plates can also be used as external shear reinforcement. However, labor costs might rise quickly. Steel stirrups have to be bent or welded and very often anchored with bolts in the concrete compression zone. When several stirrups per meter are needed, these costs can make this technique economically less interesting.

Wrapping with an FRP jacket can also provide strength enhancement for a member subjected to combined axial compression and flexure (Nosho 1996; Saadatmanesh et al.1996, Chaallal and Shahawy 2000, Sheikh and Yau 2002, Iacobucci et al. 2003, Bousias et al. 2004, Elnabelsy and Saatcioglu 2004, Harajli and Rteil 2004, Sause et al. 2004, Memon and Sheikh 2005).

Externally bonded FRP composite jackets provide effective confinement for circular concrete columns. However, these composite jackets are less effective in increasing the axial compressive strength of square and rectangular columns as verified in tests by Rochette and Labossière (2000) and Pessiki et al. (2001). Stress concentrations at the corners and ineffective confinement at the flat sides result in premature rupturing of the FRP composite jacket; thus, the high strength of the FRP composite material is not fully utilized. One approach for increasing the effectiveness of FRP confinement for rectangular concrete columns is shape modification of the cross section into an elliptical or circular section Teng and Lam 2002. A new method had been developed recently where prefabricated shells made of FRP composite layers are post tensioned with expansive cement grout and serve as stay-in-place formwork for casting the expansive cement grout around an existing square or rectangular column. The modified cross section of the original square section is circular and that of the original rectangular section is elliptical. Expansive cement consists of Portland cement and calcium sulfo aluminate anhydrite components, the hydration of the latter causes expansion (Klein et al. 1961). The advantage of using expansive cement grout is that post tensioning can be applied to the FRP composite shell through chemical expansion; the post tensioned FRP shell participates immediately and protects the core concrete by exerting an inward pressure, which postpones crack formation and growth, Confinement is modified from ―passive” to ―active,” with a higher confinement efficiency and a lower cost due to the fewer number of FRP layers required and the stay-in place formwork offered by the post tensioned FRP shell.

Figure 2.1a Installation of FRP wrap. (Courtesy of Sika Corporation.)

Figure 2.1bFRP wrap being installed on a highway column. (Courtesy of Sika Corporation)

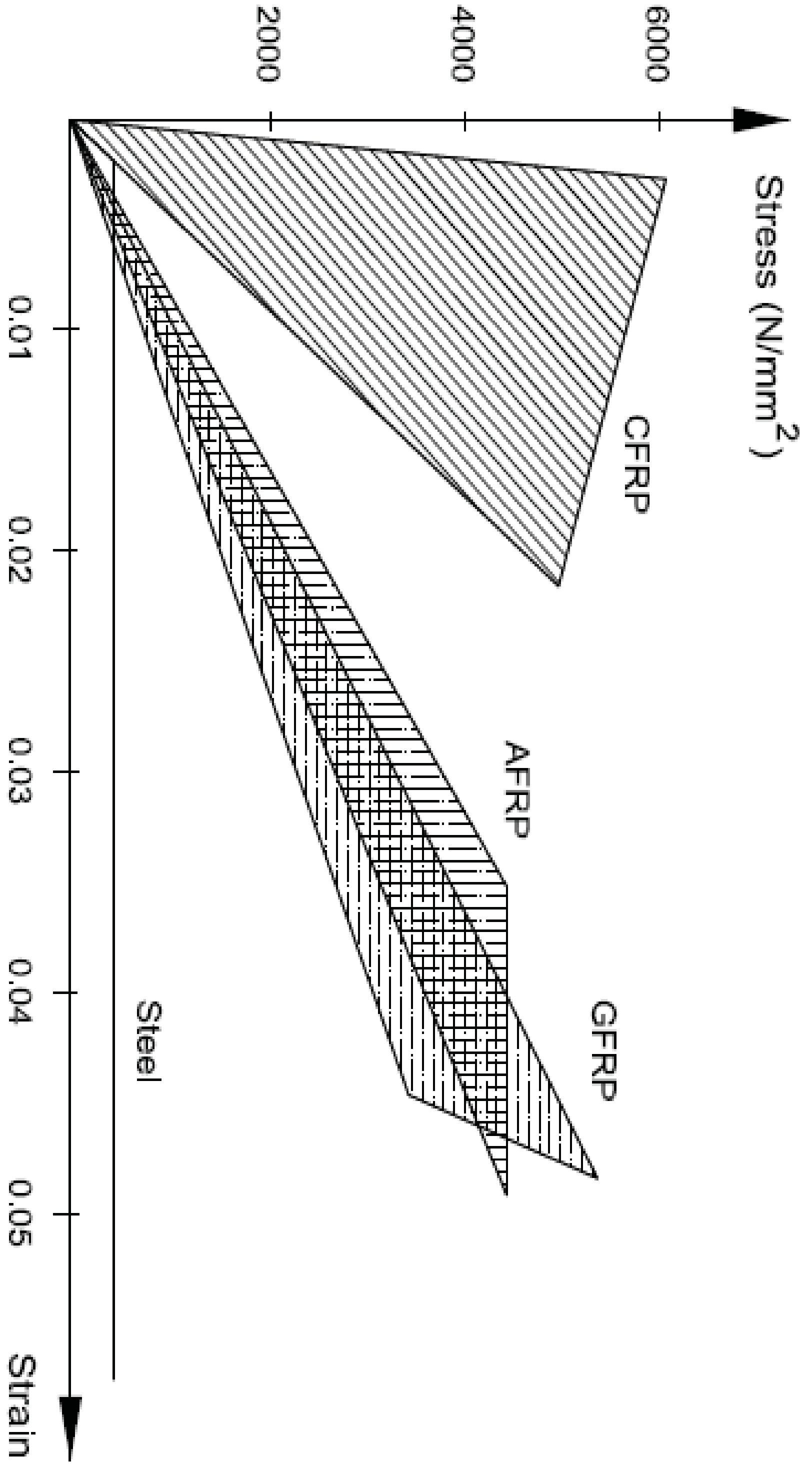


Figure 2.2 : Different stress strain diagrams of FRB to steel

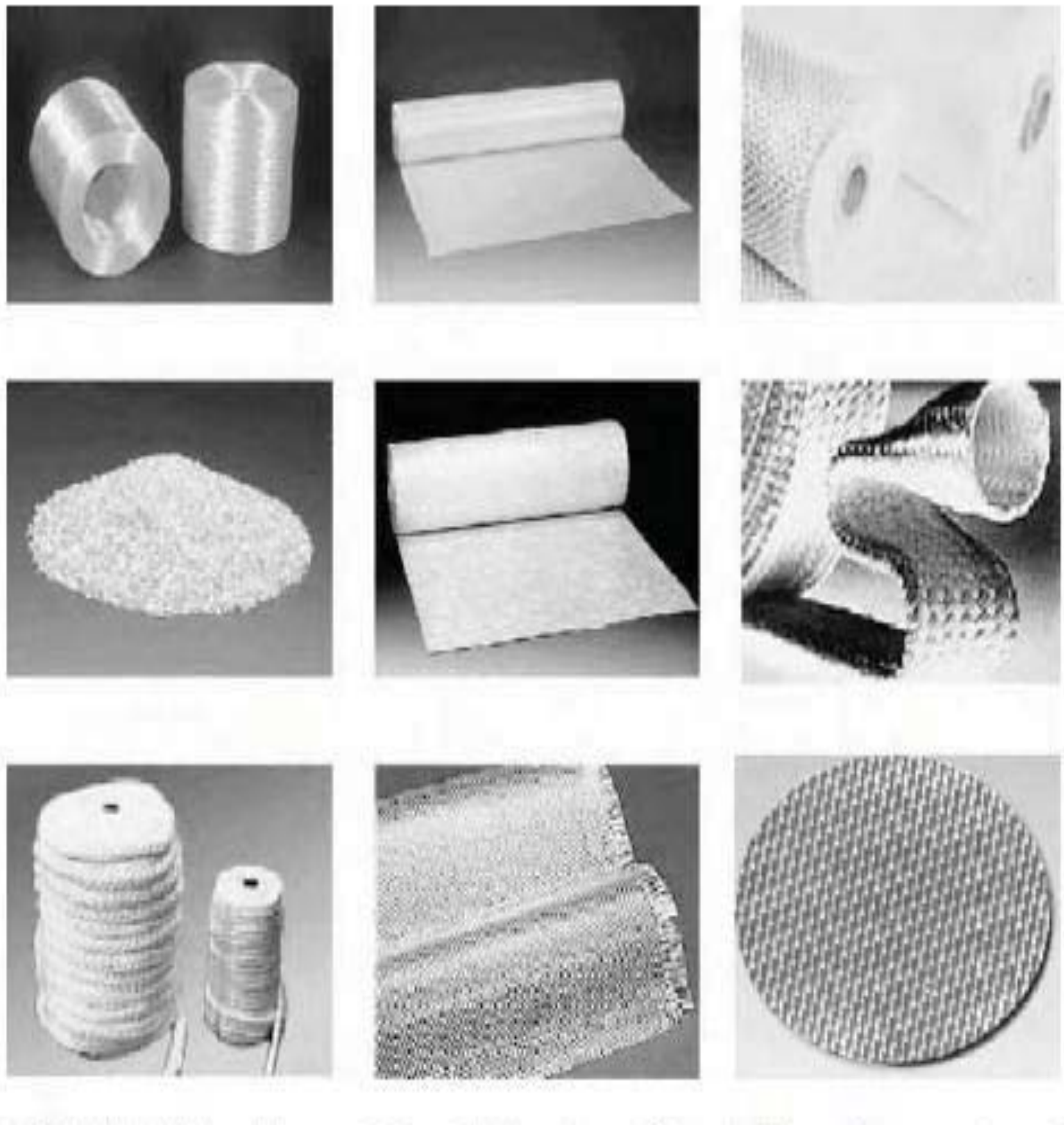


Figure 2.3: Different types of glass fabrics

From left to right in each row — top: glass roving, oven cloth roving, bidirectional cloth; middle: chopped strand, chopped strand mat, braided glass fiber sleeve; bottom: knitted glass fiber rope, glass cloth, and woven glass cloth.

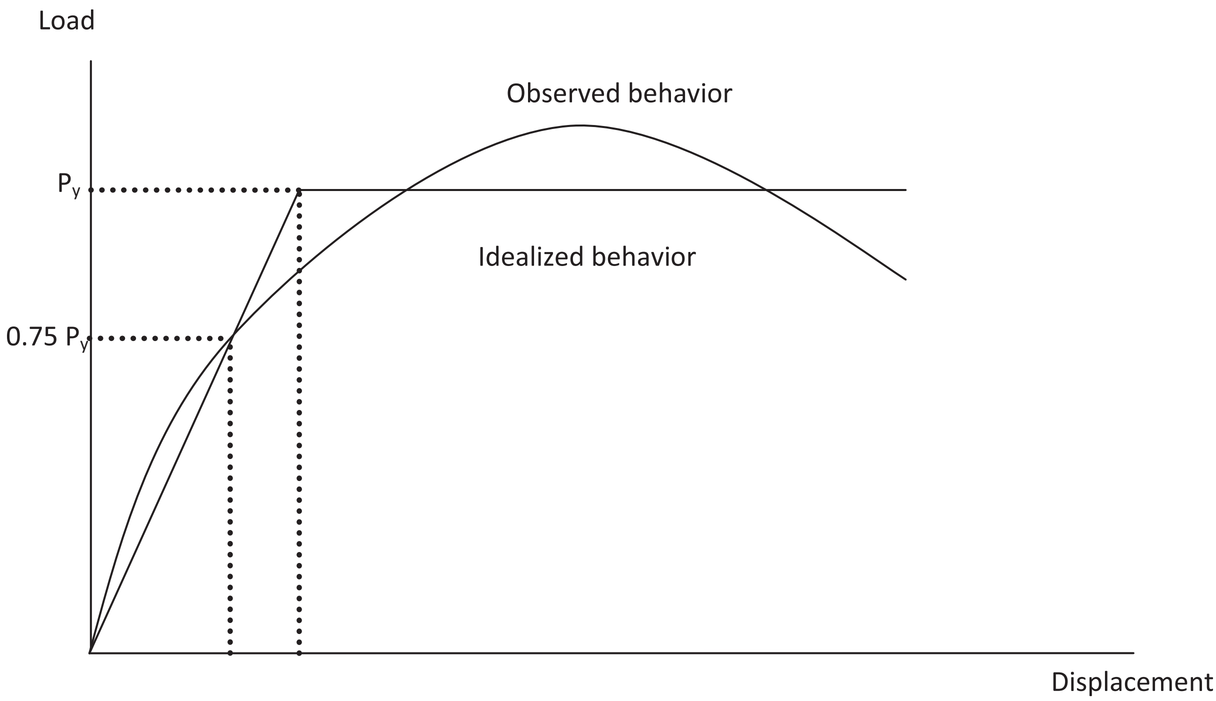


Figure 2.4 Load-Displacement curve for a Reinforced Concrete Member

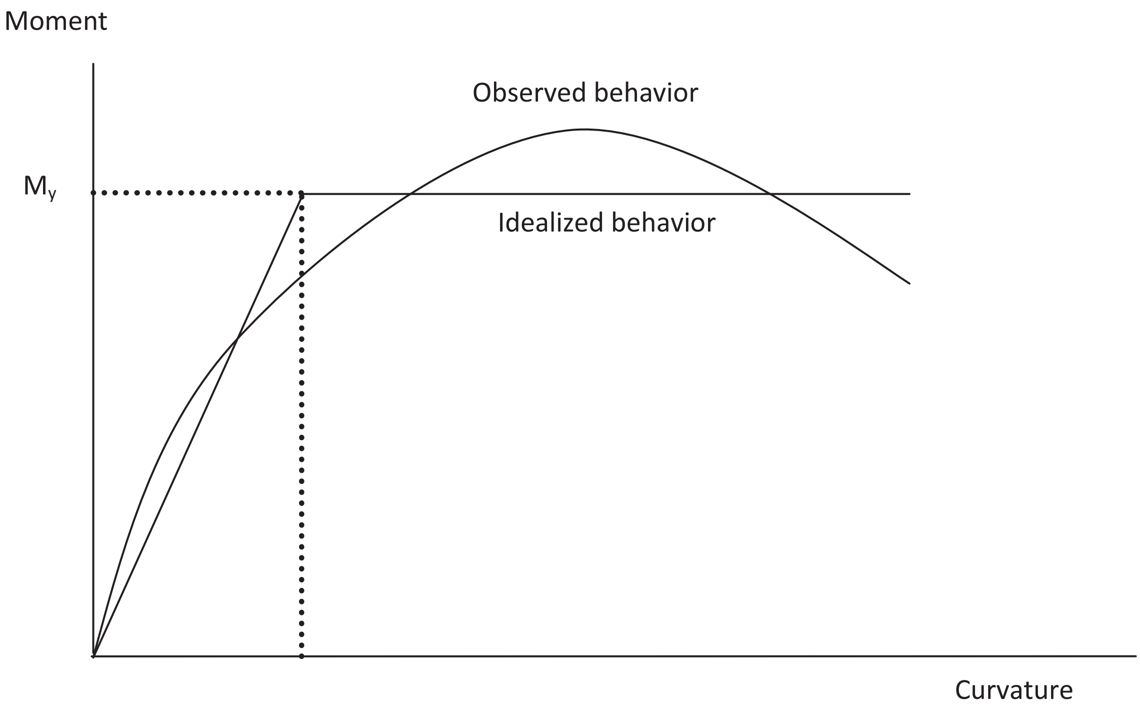


Figure 2.5Moment Curvature curve for a Reinforced Concrete Member

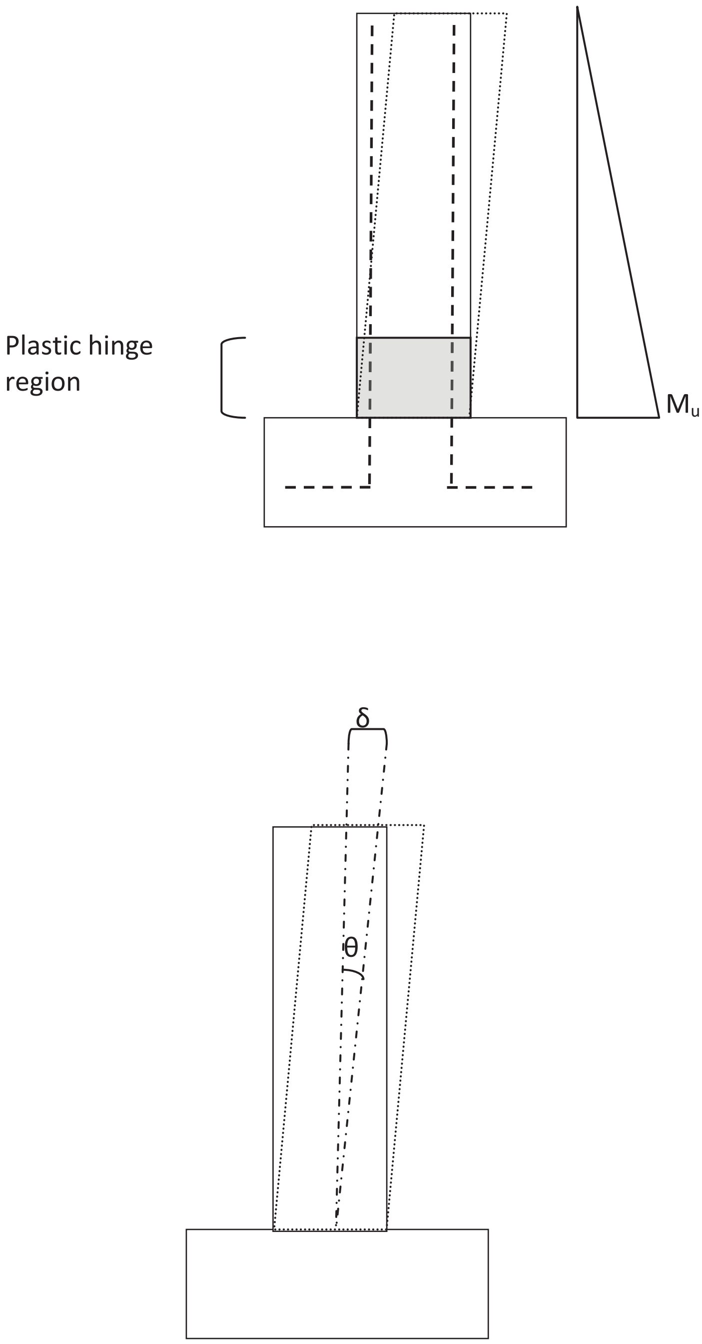


Figure 2.6 Plastic hinge region

CHAPTER 3

STRENGTHING OF RC RECTANGULAR COLUMNS CONFINED WITH FRP JACKETS

3.1Confinement

Confinement is generally applied to members in compression, with the aim of enhancing their load carrying capacity or, in case of seismic upgrading, to increase their ductility.

Traditional confinement techniques rely on their steel hoops or steel jacket for up grading. The confinement action enhances the concrete strength and ductility and, in additional, prevents slippage and buckling of longitudinal reinforcement. The confinement provided by an FRP jacket to a concrete core is passive rather than active, as the confining pressure from the jacket increases with the expansion of the concrete core

Different from steel behavior that shows a large ductility at yield the FRP has only linear behavior up to failure therefore it exerts an increasing pressure on concrete core up to rapture of FRP while the steel jacket reaches yield and then from this point it exerts a constant lateral pressure

3.1.1Lateral confining pressure

When concrete cylinder is subject to axial compressive stress, it expands laterally. For confined cylinder, this lateral expansion is resisted by the lateral pressure induced by the jacket which is loaded in tension in the circumferential (hoop) direction .The confining pressure provided by the FRP jacket increases continuously with the lateral strain of concrete because of the linear elastic stress–strain behavior of FRP, in contrast to steel-confined concrete in which the confining pressure remains constant when the steel is in plastic flow. Failure of FRP confined concrete generally occurs when the hoop rupture strength of the FRP jacket is reached. The confining action in FRP-confined concrete can be schematically illustrated in Figure 3.1.for equilibrium the lateral (radial) confining pressure acting on the concrete core f_l is given by

$$f_l = \frac{\sigma_j t}{R} \qquad (3.1)$$

where f_j= stress in the FRP jacket; t=thickness of the FRP jacket and R=radius of the confined concrete core. For FRP jackets with fibers predominantly in the hoop direction only the hoops stress is considered for jacket and due to the linearity of FRP behavior the stress in the jacket f_j is related to the strain ε_j by

$$f_j = E_j \varepsilon_j \qquad (3.2)$$

Where E_j=elastic modulus of FRP in the direction of fiber and the lateral confining pressure is related to the jacket strain by:

$$f_l = \frac{E_j \varepsilon_j t}{R} \tag{3.3}$$

3.2 Classification of stress-strain models for confined columns

In literature large number of studies on FRP-confined concrete has been out with many stress strain model developed. These models can be classified into two categories:

a) Design oriented models

b) Analysis oriented models

In the first category stress strain models are presented in closed form expression, while in second category, stress strain curves of FRP confined concrete are predicted using incremental numerical procedure.

3.2.1 Design oriented models

Various simple stress strain models in closed form expression have been proposed based directly on stress strain curves of tested FRP confined circular concrete specimens (Fardis & Khalaili 1982,Samaan et al. 1998,Saafi et. al. 1999, Xiao & Wu 2000, Lam & Teng 2002) most of these models predicts only the behavior during loading phase of column, while the models of Miyauchi et.al.(1999) and Xiao and Wu(2000) predict the stress strain relation during loading and unloading phases. Such models are particularly suitable for direct application in design calculation by hand or spreadsheets and are thus referred to as design oriented models.

3.2.2 Analysis-Oriented Models

A number of stress–strain models have been developed based on incremental iterative numerical approach in which the interaction between the concrete core and the confining FRP is explicitly accounted for. Although incremental approach are difficult to be adopted in hand calculations for design, they give more accurate results and can be adopted in advanced computer analysis such as nonlinear finite element analysis, particularly if the lateral strain of the confined concrete is also required in the analysis. These models are referred to as analysis-oriented stress–strain models.

The main advantage of this approach is:

- The behavior of both well confined and weakly confined can be predicted by the same model without difficulty
- Stress in the FRP can be explicitly evaluated throughout the loading process and related to the condition of FRP rupture.

In recent years, external confinement of concrete using FRP composites has emerged as a popular method of column retrofit, particularly for circular columns and many recent studies have been conducted on the compressive strength and stress-strain behavior of FRP-confined concrete. These studies showed that FRP-confined concrete behaves differently from steel-confined concrete. Consequently, various models for predicting the compressive strength and the stress-strain behavior have been developed specifically for FRP-confined concrete, in the context of both column strengthening and concrete-filled FRP tubes for new construction.
The majority of models exist in literature take one of the two forms:

$$f_{cc} = f_{co} + kf_l \qquad (3.4)$$

Or

$$\frac{f_{cc}}{f_{co}} = 1 + k\frac{f_l}{f_{co}} \qquad (3.5)$$

Where $f\,cc$ and fco are the compressive strengths of the confined and unconfined concrete respectively, f_l is the lateral confining pressure, and k is the confinement effectiveness coefficient. This form was first proposed by Richart et al. (1928) for actively confined concrete with a value of 4.1 for k. however the value of k varies considerably between the models table 1 and its measured based on each author experimental conditions FRP-confined concrete, f_l is related to the thickness and strength of the FRP by:

$$f_l = \frac{f_j\ t}{R} \qquad (3.6)$$

Where, f_j is the tensile strength of FRP in the hoop direction t is the total thickness of the FRP jacket, and R is the radius of the confined circular column.

The lateral maximum stress acting on the concrete as a result of the confinement action can then be obtained by equilibrium of forces E_l is called confinement modulus or lateral modulus it is a measure of the stiffness of the confining device. Figure 3.5 shows the peak stresses for a confined concrete column using equations in table 3.2

$$f_l = \frac{E_j\, \varepsilon_{ju} t}{R} = E_l\, \epsilon_{ju} \qquad (3.7)$$

The maximum value of the confinement pressure that the FRP can exert is attained when the circumferential strain in the fibers reaches its ultimate strain (ε_{ju}) and the fibers rupture leading to brittle failure of the cylinder

Mander et al. [1988a] proposed a model for concrete confined by transverse steel. This model is one of the most popular models and is the basis for most current codes. What makes this model attractive to designers and researchers is the fact that it uses a single equation for the entire range of concrete compressive strains. In addition, it can be applied to circular and rectangular columns confined by circular or rectangular shaped transverse reinforcement it can also adapt to account for the confinement due to FRP and/or steel hoops. The model gives good results compared to experiences and represents the behavior of confined concrete Figure (3.3) shows the result of this model for the same concrete confined at different levels of constant lateral pressure.

3.3 Stress-Strain Relationship of Concrete Confined by both Hoops and FRP

3.3.1 Equilibrium Considerations of Jacketed Rectangular Column

It is well established that external confinement of a concrete column enhances its strength and ductility. In the past, confinement was usually provided by external jackets or tubes made of metal. Currently, the availability of composite materials with improved physical properties has opened the door for opportunity to rehabilitate concrete structures. In recent years, tests performed by several researchers [Karbhari et al. 1993], [Saadatmanesh et al. 1994] have confirmed that use of FRP sheets to provide additional confinement to the concrete compression members could be an effective technique to strengthen further their load carrying capacity and to repair deteriorated columns for the designed load considered. As an attempt to the application of composite fabrics where strengthening or retrofitting of structural member is the prime concern, several researchers have proposed models for assessment of gain in strength and ductility of confined concrete.

Figure (3.4a) shows a cross section of a reinforced concrete rectangular column confined by an external jacket. The jacket confines the column core, introducing pressure through the rounded corners with an arching action occurring in the horizontal direction. It is assumed that the jacket is reinforced only in the direction transverse to the column axis. Figure (3.4b) shows the section confined by internal reinforcing steel hoops; the steel hoops induce a confining pressure at the nodes intersection between hoops and the column longitudinal bars, whereas in the concrete confined by steel hoops, arching action occurs in the horizontal and in vertical directions. It should be noted that when both sources of confinement are present, a large area of the concrete core might be confined by both the jacket and the steel hoops. The load-deformation response of concentrically loaded column confined with external FRP jacket and internal steel hoops can be traced using the method proposed by Wang and Restrepo (2001) with some modifications as follow.

The area of the concrete core confined by both steel hoops and FRP jacket is equal to the area confined by steel hoops; it is calculated considering parabolic arching that takes place between two steel hoops in the vertical direction and between two longitudinal bars restrained by the hoops in the transverse direction:

$$A_{cjs} = \left(d_x d_y - \sum \frac{w_s^{\,2}}{6} \right) \left(1 - 0.5 \frac{s}{d_x} \right) \left(1 - 0.5 \frac{s}{d_y} \right) \tag{3.8}$$

The area of unconfined concrete is equal to:

$$A_0 = \frac{w_{jx}^{\,2} + w_{jy}^{\,2}}{3} \tan \theta_j \tag{3.9}$$

while area of unconfined concrete is given by:

$$A_{cj} = t_x t_y - (4r^2 - \pi r^2) - A_s - A_{cjs} - A_0 \tag{3.10}$$

Figure (3.4) defines the variables used in above equations. The term s' is the clear spacing of the hoops and A_S is the area of longitudinal reinforcement and the term r is the radius of rounded corner of column cross section. The term w_S is clear distance between the longitudinal bars. It is assumed that the concrete core area confined by the internal steel hoops is inside the area confined by the external jacket. The horizontal arching angle θ_j in Eq. (3.9), due to the presence of the jacket, can be derived using experimental work. The angle θ_s, due to the steel hoop confinement, is assumed to be 45 degree.

3.3.2 Numerical Procedure

Mander et al. (1988a) proposed a unified stress-strain approach for confined concrete applicable to both circular and rectangular shaped transverse reinforcement. The model was based on an equation suggested by Popvics (1973). In this approach the relationship between longitudinal compressive concrete stress f_c and longitudinal compressive strain ε_c takes the forms:

$$f_c = \frac{f_{cc} x g}{g - 1 + x^g} \qquad \textbf{(3.11)}$$

$$x = \frac{\varepsilon_c}{\varepsilon_{cc}} \qquad \textbf{(3.12)}$$

$$\varepsilon_{cc} = \varepsilon_{co}\left[1 + 5\left(\frac{f_{cc}}{f_c} - 1\right)\right] \qquad \textbf{(3.13)}$$

$$g = \frac{E_c}{E_c - E_{\sec}} \qquad \textbf{(3.14)}$$

$$E_{\sec} = \frac{f_{cc}}{\varepsilon_{cc}} \qquad \textbf{(3.15)}$$

Where f_{cc} = compressive strength of confined concrete, ε_c = longitudinal compressive concrete strain. f_c is the compressive strength of concrete and ε_{co} is the corresponding strain (generally ε_{co}=0.002 can be assumed), and $E_c = 5000\sqrt{f_c}$ (N/mm^2).in his model Mander expressed the compressive strength of confined concrete f_{cc} by

$$f_{cc} = \alpha_1 \alpha_2 f_c \quad \textbf{(3.16)}$$

in which α_1 is a strength enhancement factor that considers the concrete to be subjected to a tri-axial stress state with bi-equal confining stresses, and α_2 is a reduction factor that considers any deviation from the bi-equal confining stress concept.

The factors α_1 and α_2 were proposed by Mander et al. (1988) and by Wang and Restrepo, 2001, as follows:

$$\alpha_1 = 1.25\left(1.8\sqrt{1+7.94\frac{F_l}{f_c'}} - 1.6\frac{F_l}{f_c'} - 1\right) \quad \textbf{(3.17)}$$

$$\alpha_2 = \left(1.4\frac{f_l}{F_l} - 0.6\,(\frac{f_l}{F_l})^2 - 0.8\right)\sqrt{\frac{F_l}{f_c}} + 1 \quad \textbf{(3.18)}$$

where F_l and f_1 are the maximum and minimum confining stresses in x and y directions respectively, The lateral confining stresses due to by the FRP jacket in the x and y directions $f_{l,jx}$ and $f_{l,jy}$ are derived from equation 1for rectangular section as:

$$f_{l,jx} = \rho_{jx} f_j$$

$$f_{l,jy} = \rho_{jy} f_j \quad \textbf{(3.19(a,b))}$$

Where f_j is the confining stress and is obtained from

$$f_j = E_j \varepsilon_t \quad \textbf{(3.20)}$$

For $(0 \le \varepsilon_j \le \varepsilon_{ju})$

$$f_j = 0$$

For $(\varepsilon_j > \varepsilon_{ju})$

Where E_j and ε_{ju} are the modulus of elasticity and the ultimate strain of the jacket material respectively, and ε_j is the circumferential strain in the jacket.

The ratios ρ_{jx} and ρ_{jy} are defined as:

$$\rho_{jx} = 2\frac{t_j}{t_y} \qquad \rho_{jy} = 2\frac{t_j}{t_x} \tag{3.21(a,b)}$$

Where t_j is the jacket thickness and t_x and t_y are the overall cross-sectional dimensions.

Similarly the confining stresses due to the steel hoops in the x and y directions $f_{l,sx}$ and $f_{l.sy}$ take the forms:

$$f_{l,sx} = \rho_{sx} f_{sh} \qquad\qquad f_{l,sy} = \rho_{sy} f_{sh} \tag{3.22(a,b)}$$

Where f_{sh} is the confining stress in the hoops and is obtained as

$$f_s = E_s \varepsilon_t \qquad \text{for } (\varepsilon_t < \varepsilon_y) \tag{3.23}$$

$$f_s = f_{sy} \qquad \text{for } (\varepsilon_t \geq \varepsilon_y)$$

Where E_s is the modulus of elasticity of steel reinforcement and ε_y and f_{sy} are the yield strain and yield stress of the steel reinforcement, respectively.

The confinement reinforcement ratios ρ_{sx} and ρ_{sy} are defined as

$$\rho_{sx} = \frac{A_{t,x}}{s\,d_y} \qquad \rho_{sy} = \frac{A_{t,y}}{s\,d_x} \tag{3.24(a,b)}$$

in which d_x and d_y are the distances between the centerlines of the perimeter hoop in the x and y directions, respectively; $A_{t,x}$ and $A_{t,y}$ are areas of transverse steel parallel to the x- and $y-$ axis, respectively; and s is the spacing between sets of hoops.

In general, the confined concrete strength of the area of the column solely confined by the jacket $f_{cc,j}$ is found by substituting Eq. (3.24) into Eq. (3.21).

The lateral confining stress f_l acting upon the area confined by both the jacket and the steel hoops is the summation of the lateral confining stress due to steel hoops $f_{l,s}$ and the lateral confining stress due to jacket $f_{l,j}$

$$f_l = f_{l,s} + f_{l,j} \qquad \textbf{(3.25)}$$

And, the confined concrete strength of the area of the column confined by the jacket and the transverse steel reinforcement $f_{cc,js}$ is found substituting Eq. (3.30) into Eq. (3.21).

In recent years, researchers have attempted to extend Mander's model to predict the behavior of concrete, accounting for the effect of confinement provided by elastic FRP jackets. A major obstacle of the model is due to the use of constant value for confining pressure throughout the loading history. FRP behaves elastically until failure, and the inward pressure increases continuously, so this assumption is not appropriate. Based on Pantazopoulu and Mills (1995) constitutive model for unconfined concrete under uniaxial loading, Spoelstra and Monti (1999) proposed the an equation that explicitly shows the dependence of concrete f_c and lateral strain ε_l on the current strain ε_c and the confining pressure f_l

$$\varepsilon_l(\varepsilon_c, f_l) = \frac{E_c\varepsilon_c - f_c(\varepsilon_c, f_l)}{2\ \beta\ \ f_c(\varepsilon_c, f_l)} \qquad \textbf{(3.26)}$$

$$\beta = \frac{5700}{\sqrt{f_{co}}} - 500 \qquad (\ f_{co} \text{ in N/mm}^2) \qquad \textbf{(3.27)}$$

It is assumed that under concentric compressive loading, the transverse strain in the concrete surface of the column, ε_l, the strain in the jacket ε_j, and the strain in the steel hoops ε_s are all equal that is

$$\varepsilon_l = \varepsilon_j = \varepsilon_s \tag{3.28}$$

This model represent an incremental approach to define the stress and strain during loading ,At each incremental of lad ε_t is computed from Eq. (3.31), the strain ε_j in the confining jacket can be found along with its current stress, f_l. This updated value of f_l can be used for a new estimate of ε_t through Eq. (3.31), giving rise to an iterative procedure (Figure 3.3) until f_l converges to a stable value. The whole procedure is repeated for each ε_c, over the complete stress-strain curve. This latter can be regarded as a curve crossing a family of Mander's curves, each one pertaining to the level of confining pressure corresponding to the current lateral strain(Figure 3.4) The stress-strain characteristics of the confining mechanism are explicitly accounted for, while the lateral strain of concrete is implicitly included in the model. Figure 3.5and 3.6 shows some results from applying Mander equation on a square concrete column using GFRP and CFRP. It can be shown that confining using GFRP give more ductility but less strength. While in Figure 3.7 and 3.8 the comparison was between same cross-section but using different concrete compressive strengths 30MPa and 45MPa respectively and using different FRP thickness. As shown in the figures the more the column is wrapped the higher the strength is gained.

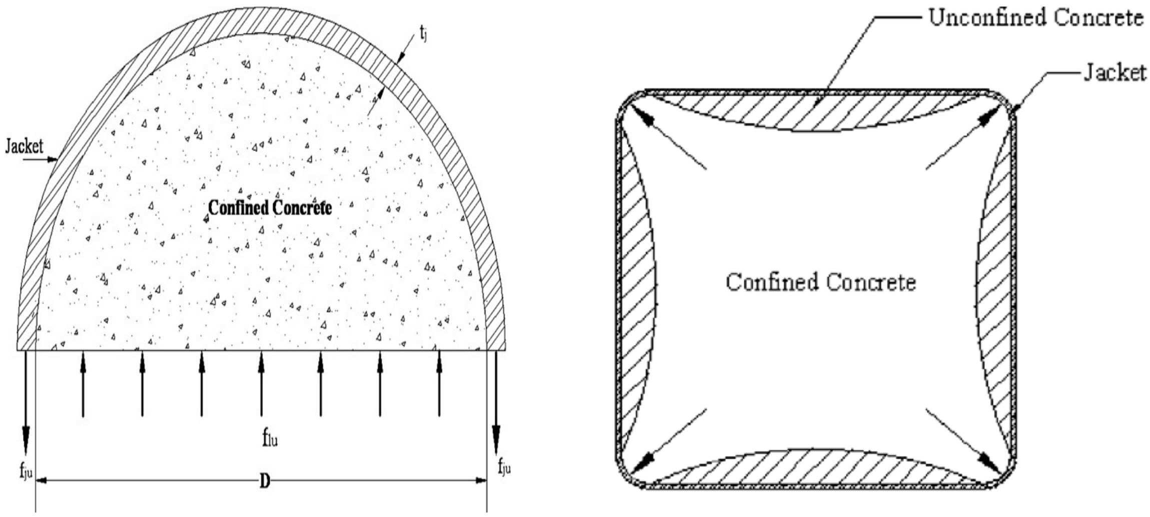
In Figures 3.9 and 3.10 the ECP equation is compared with the Mander equation with different corner radii and FRPs. The ECP gives a higher stress compared to Mander due to the modification on the equation and the safety limits in the ECP.

Model	Equation	k Factor
Samaan et al.(1998)	$f_{cc} = f_{co} + 6f_l^{0.6}$	$6f_l^{0.6}$
Fardis and Khalili(1981)	$f_{cc} = f_{co} + 4.1f_l$	4.1
Saafi et al. (1999)	$f_{cc} = f_{co} + 2.2f_l^{0.84}f_{co}^{0.16}$	$(\frac{f_l}{f_{co}})^{-0.16}$
Miyauchi et al.(1997)	$f_{cc} = f_{co} + 3.485f_l$	3.485
Lam and Teng(2002)	$f_{cc} = f_{co} + 2f_l$	2
Youssef(2003)	$f_{cc} = f_{co} + 2.1(f_l)^{0.84}(f_{co})^{-0.25}$	$2.25(\frac{f_l}{f_{co}})^{0.25}$
Kabahari and Gao(1997)	$f_{cc} = f_{co} + 2.1(f_l)^{0.87}(f_{co})^{-0.13}$	$2.1(\frac{f_l}{f_{co}})^{-0.13}$
Toutanji(1999)	$f_{cc} = f_{co} + 3.5f_l^{0.84}f_{co}^{0.15}$	$3.5(\frac{f_l}{f_{co}})^{-0.15}$
Saadatmanesh al(1994)	$f_{cc} = f_{co}(-1.254 + 2.254\sqrt[2]{1 + 2.254\left(\frac{7.94f_l}{f_{co}}\right)} - 2(\frac{f_l}{f_{co}}))$	——

Table 3.1 FRP Confined Models K factor

Model	f_{cc}	ε_{cc}
Fardis and Khalili	$f_{cc} = f_{co}(1 + 4.1\left(\frac{f_l}{f_{co}}\right))$	$\varepsilon_{cc} = \varepsilon_{co} + 0.0005(\frac{E_l}{f_{co}})$
Saafi et al.	$f_{cc} = f_{co}(1 + 2.2\left(\frac{f_l}{f_{co}}\right)^{0.84})$	$\varepsilon_{cc} = \varepsilon_{co}(2 + 537\varepsilon_{fu} + 2.6)(\frac{f_{cc}}{f_{co}} - 1)$
Xiao and Wu	$f_{cc} = f_{co}(1.1 + [4.1 - 0.75\left(\frac{f_{co}{}^2}{E_l}\right)](\frac{f_l}{f_{co}}))$	$\varepsilon_{cc} = \frac{\varepsilon_{fu} - .0005}{7(\frac{E_l}{f_{co}})^{0.8}}$
Sopelstra and Monti	$f_{cc} = f_{co}(0.2 + 3\left(\frac{f_l}{f_{co}}\right)^{0.5})$	$\varepsilon_{cc} = \varepsilon_{co}(2 + 1.25\frac{E_{co}}{f_{co}}\varepsilon_{fu}\sqrt[2]{\frac{f_l}{f_{co}}})$
Mander	$f_{cc} = f_{co}(2.254\sqrt[2]{1 + 7.94\left(\frac{f_l}{f_{co}}\right)} - 2(\frac{f_l}{f_{co}}) - 1.254)$	$\varepsilon_{cc} = \varepsilon_{co}(1 + 5(\frac{f_{cc}}{f_{co}} - 1)$
Kono	$f_{cc} = f_{co}(1 + 0.0572f_l)$	$\varepsilon_{cc} = \varepsilon_{co}(1 + 0.28f_l)$
Miyauchi et al.	$f_{cc} = f_{co}(1 + 3.485\frac{f_l}{f_{co}})$	$\varepsilon_{cc} = \varepsilon_{co}\left(0.2 + 3\left(\frac{f_l}{f_{co}}\right)^{0.363}\right), f_{co} = 30MPa$ $\varepsilon_{cc} = \varepsilon_{co}\left(0.2 + 3\left(\frac{f_l}{f_{co}}\right)^{0.525}\right), f_{co} = 50MPa$
ACI(Tang and Lam)	$f_{cc} = f_{co}(1 + 3.3\left(\frac{f_l}{f_{co}}\right))$	$\varepsilon_{cc} = \varepsilon_{co}(1.75 + 12\left(\frac{f_l}{f_{co}}\right)\left(\frac{\varepsilon_{fu}}{\varepsilon_{co}}\right)^{0.45})$
ECP208-2010	$f_{cc} = f_{co}(2.25\sqrt[2]{1 + 9.875\left(\frac{f_l}{f_{co}}\right)} - 2.5(\frac{f_l}{f_{co}}) - 1.25)$	$\varepsilon_{cu} = 1.37\left(\frac{5f_{cc} - 4f_{cu}}{E_c}\right)$

Table 3.2 Summary of FRP – Confined Models



a b

Figure 3.1 Confinement in different sections

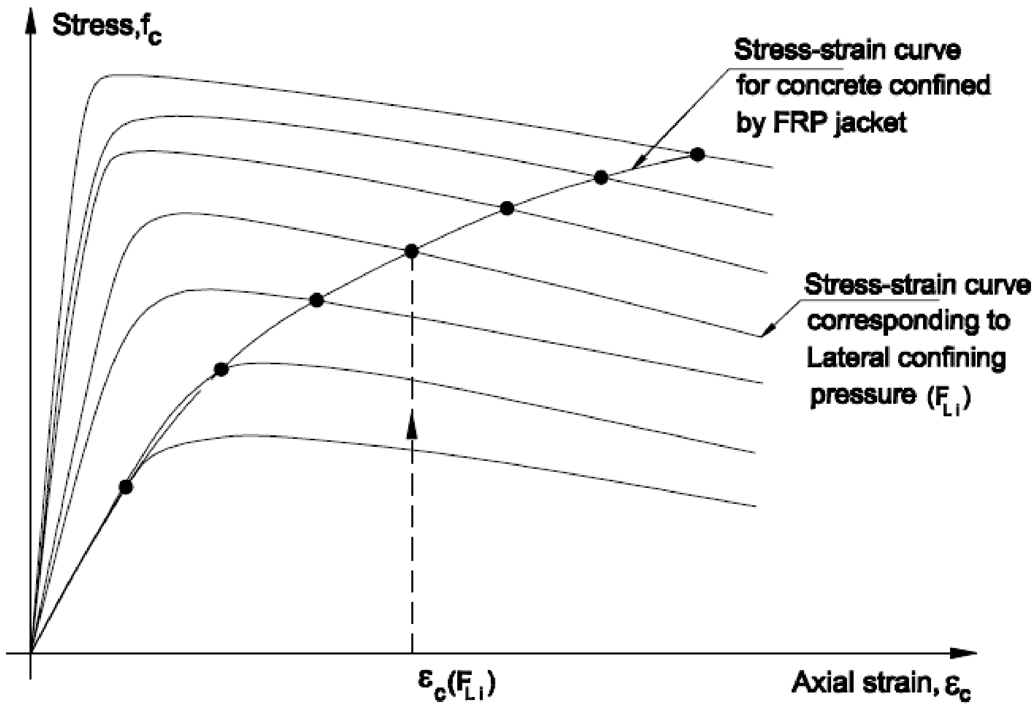


Figure 3.2 **Stress-strain increment**

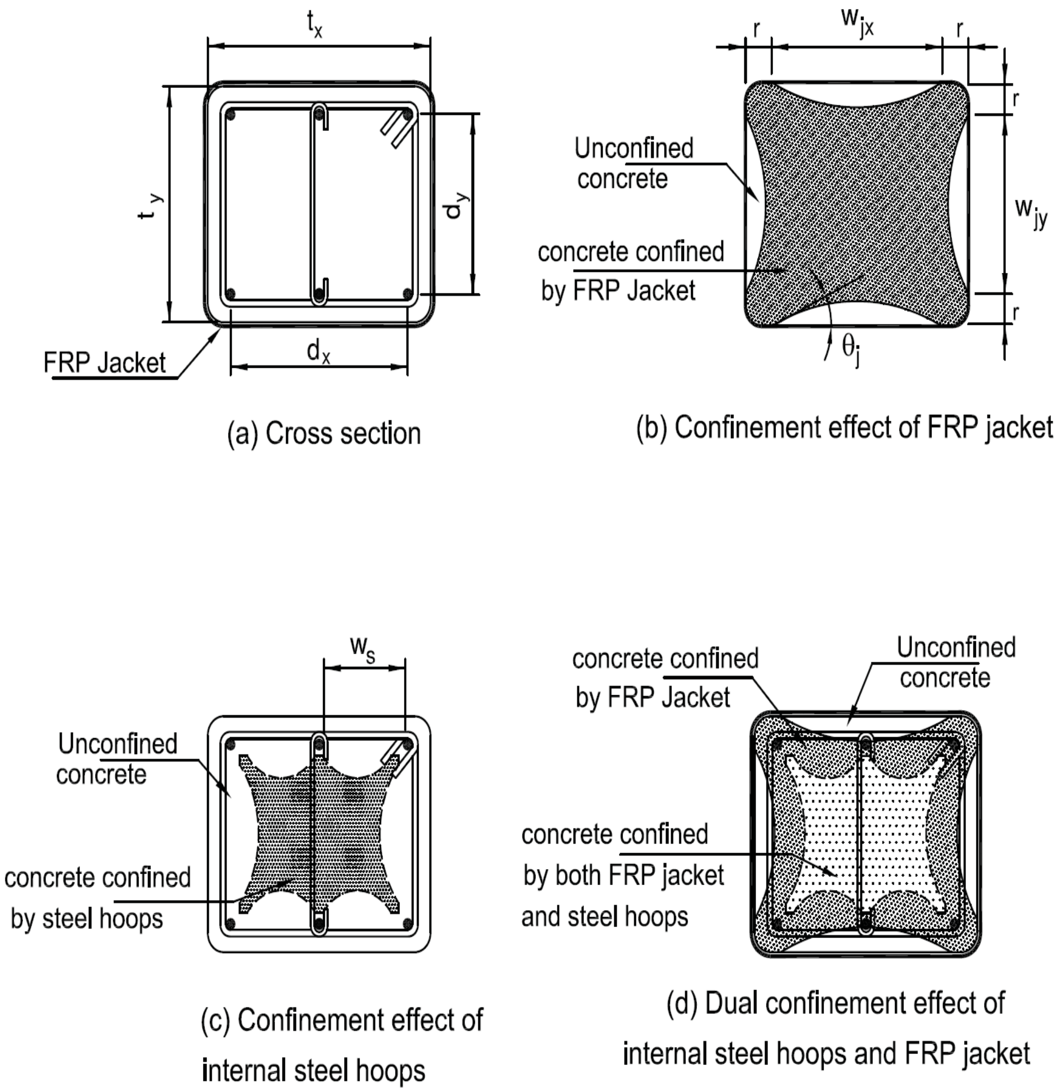


Figure 3.3 Confinement of Rectangular and square sections

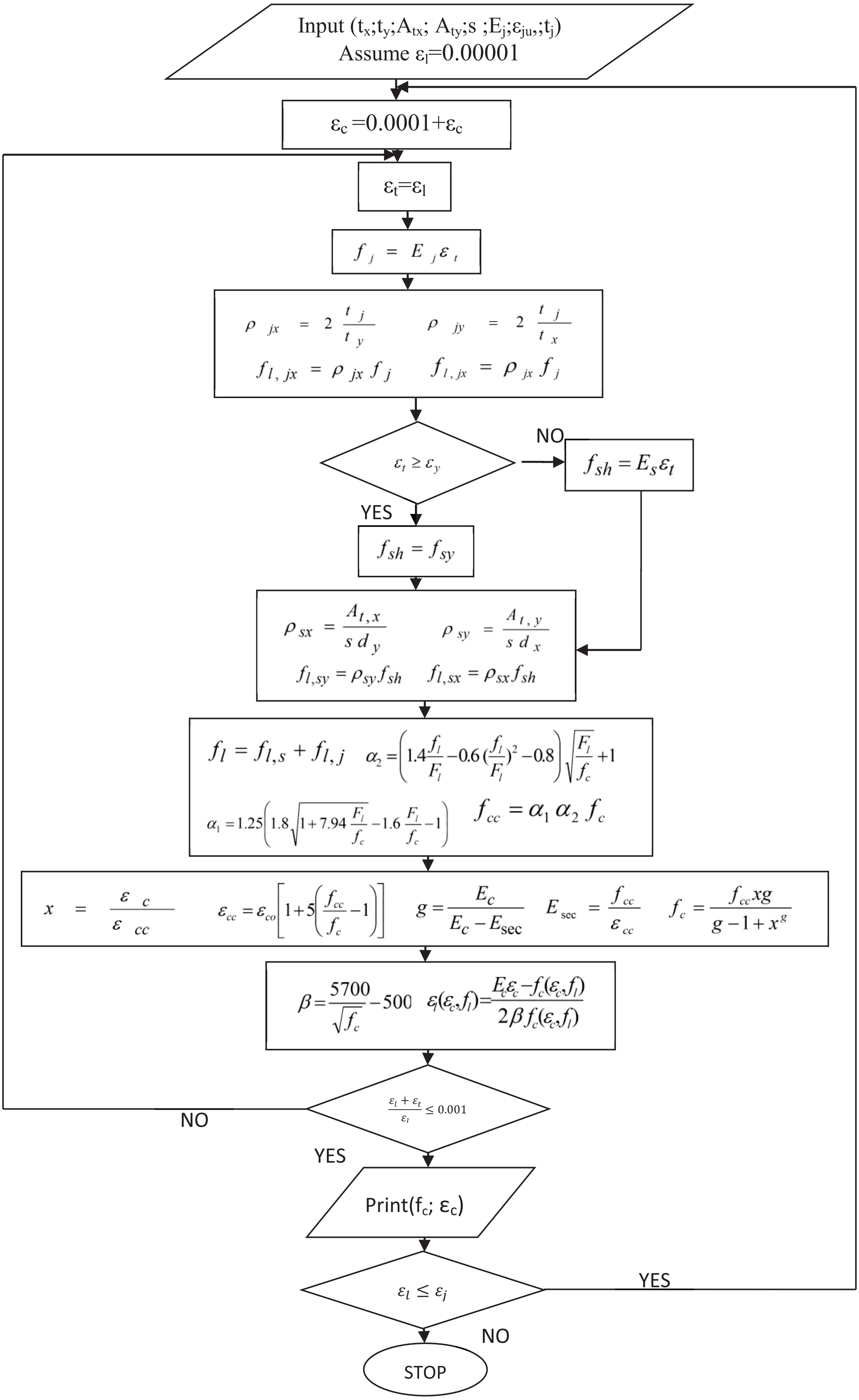


Figure 3.4Flow chart Stress-strain relationship using Mander equation

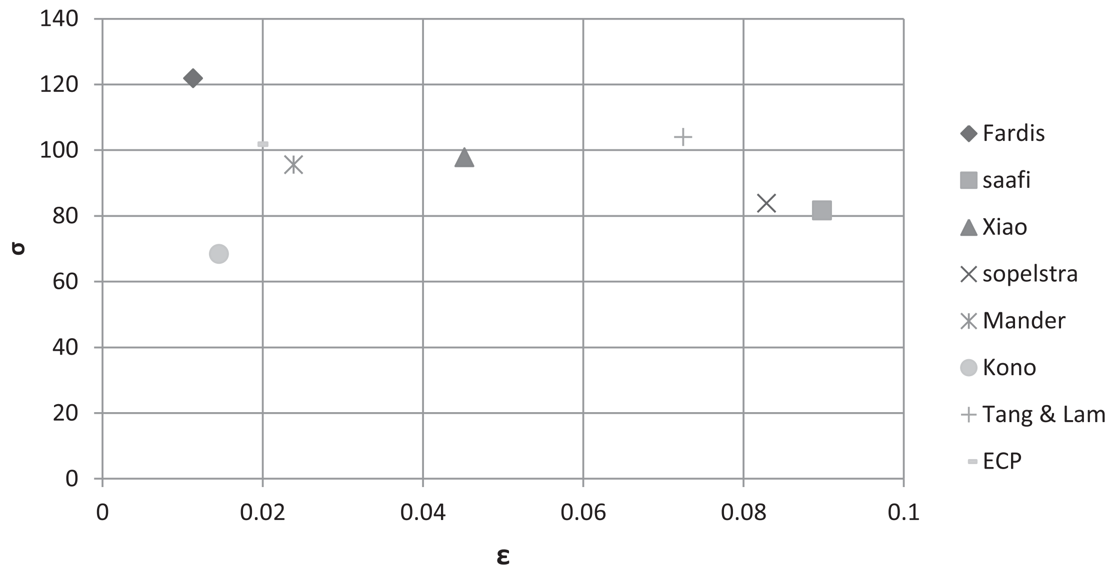


Figure 3.5 Different confining models of concrete peak stress and strain

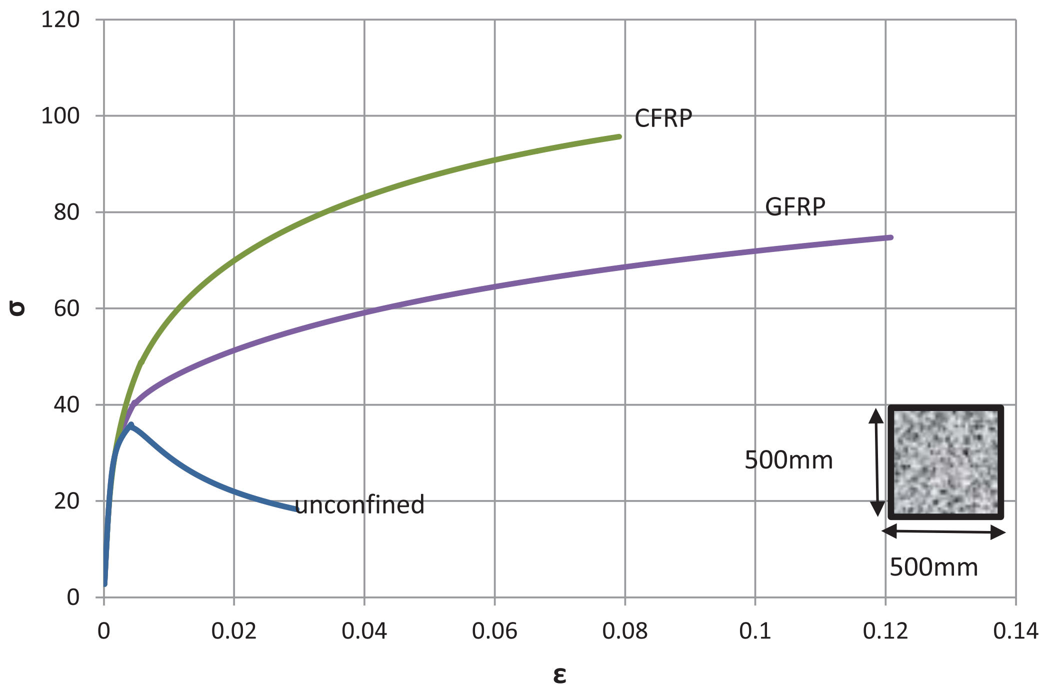


Figure 3.6 Stress strain of GFRP and CFRP

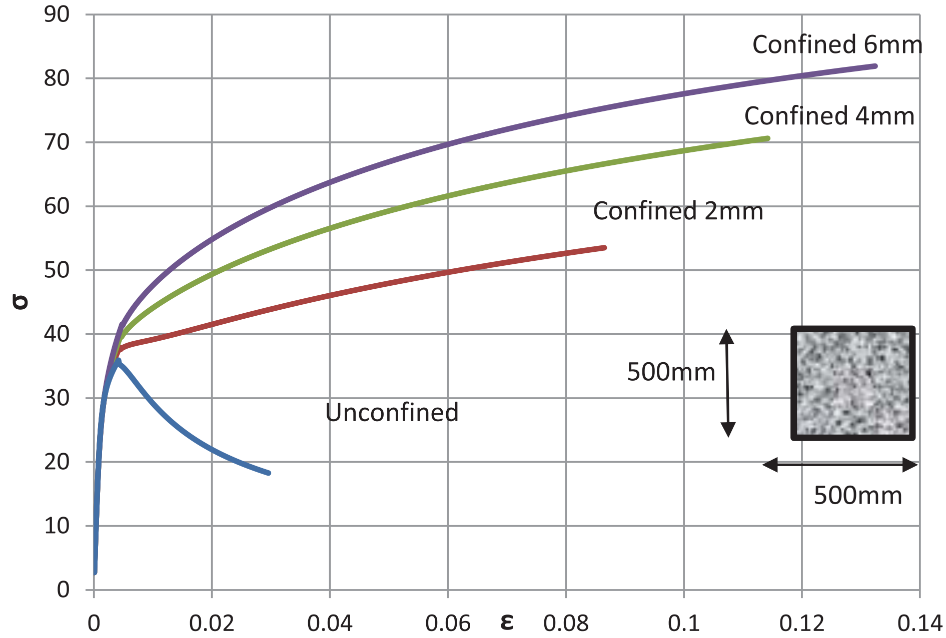


Figure 3.7 GFRP confinment of square section (f_{cu}=30MPa corner radius=0)

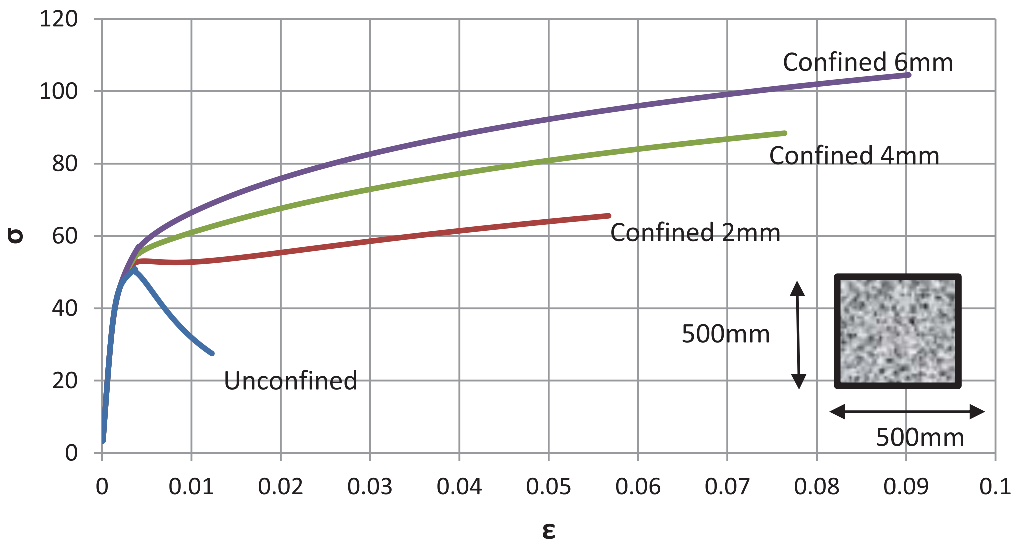


Figure 3.8 GFRP confinment of square section (f_{cu}=45MPa corner radius=0)

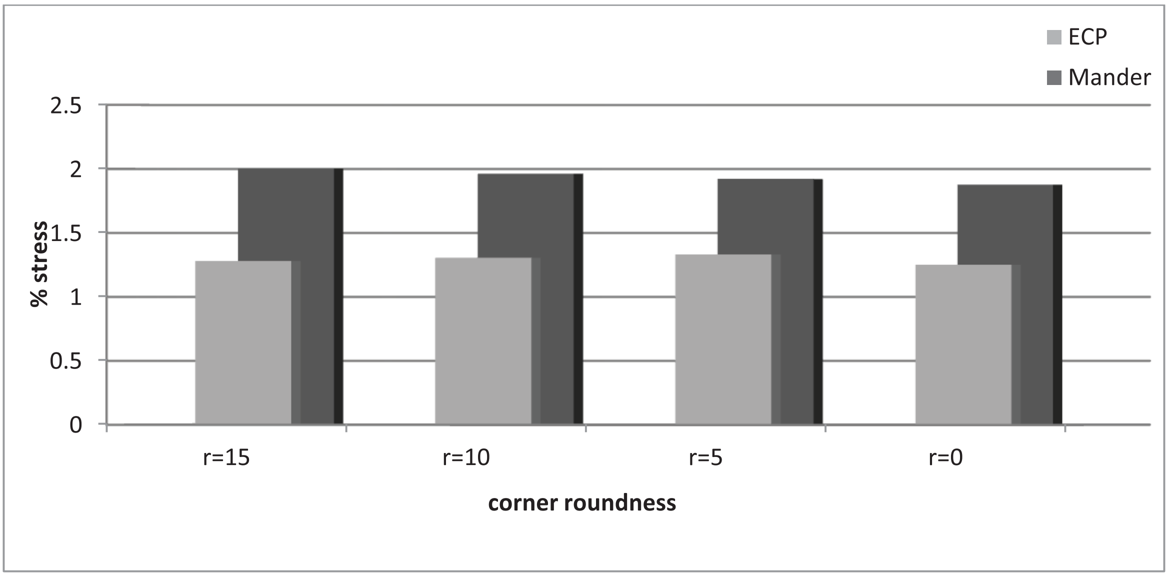


Figure 3.9 Comparison between ECP and Mander model to peak stresses for a square section using GFRP

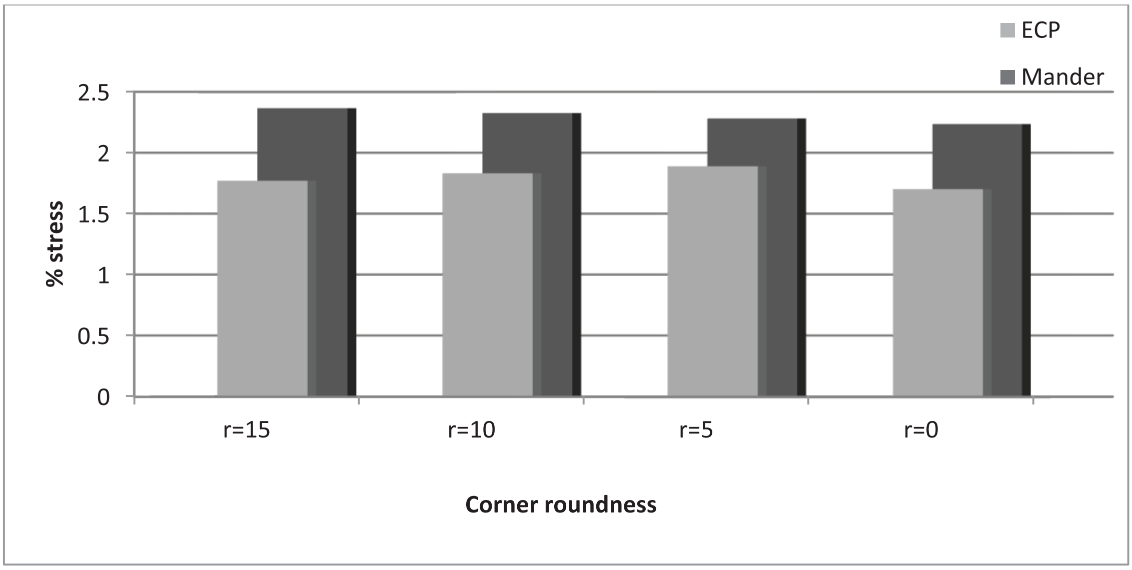


Figure 3.10 Comparison between ECP and Mander model to peak stresses for a square section using CFRP

CHAPTER 4

ULTIMATE DEFORMATIONS

AND

DUCTILITY OF MEMBERS WITH FLEXURE

4.1 Introduction

Different types of load-deflection behavior of reinforced concrete members up to and beyond ultimate load exists in literature, brittle or ductile behavior of reinforced members depends on the amount of steel used in this section. The study of load-deformation characteristics of members subjected to bending is necessary for the following reasons:

1. Brittle failure of members should be prevented in the extreme event of a structure being loaded to failure; it should be capable of undergoing large deflections at near-maximum load carrying capacity. This may save lives by giving warnings of failure and preventing total collapse.
2. The possible redistribution of bending moment, shear force and axial load that could be used in design of statically indeterminate structures depends on the ductility of the members at the critical sections. At ultimate state a new distribution of bending moments, different from initial linear elastic analysis, can be achieved when moment redistribution can take place. As ultimate load is approached, some sections reach their ultimate resistance moment before others forming plastic hinged if they are ductile enough, additional load can be carried as the moment elsewhere increase to their ultimate value. The ultimate load of the structure is reached after the formation of large numbers plastic hinges making the structure unstable and developing a collapse mechanism. Most codes allow some redistribution of moment in design, depending on the ductility of sections. Using moment redistribution can be advantageous because it may reduce the amount of reinforcement at the supports of continuous beams allowing the formation of plastic hinge at two location and enables reduction in the peak bending moments.
3. In zones subjected to earthquake, the ductility of structure when subjected to seismic loading becomes an important design consideration because the present seismic design philosophy relies on energy absorption and dissipation by post elastic deformation for survival. In major earthquakes, the structures incapable of behaving in ductility deformation must be designed for much higher seismic forces if collapse is to be avoided. Fig-

ure (4.1) shows the behavior of a typical under reinforced concrete element subjected to bending moment with different key points.

Reinforced concrete structural members such as columns and beams can be confined with FRP systems to increase their axial load-carrying capacity. Axial strengthening is most suitable for circular non slender (i.e., short) columns. Combined axial and flexural strengthening of short eccentrically loaded reinforced concrete columns increases their axial and flexural capacity. Axial load-bending moment (*P–M*) strength interaction diagram can be constructed for an FRP-strengthened reinforced concrete column in a similar way to that of a non-strengthened column.

Axial strengthening is obtained by applying the FRP system oriented such that its principal fiber direction is in the circumferential (or hoop) direction of the member, perpendicular to its longitudinal axis. In addition to providing axial strengthening, hoop FRP reinforcement provides shear strengthening to the member, since it is oriented perpendicular to the member axis. As noted previously, when strengthening for only a single mode is intended, it is incumbent on the designer to determine the effects of the strengthening on the other modes and to ensure that the member has sufficient capacity in the other modes to resist the higher applied loads.

FRP axial strengthening of rectangular columns can be achieved using either continuous or intermittent strip coverage. Since the axial load is constant along the full height of the column, the FRP wrap must cover the full height of the column; however, it can be spaced intermittently, in either intermittent or spiral form. It has been shown that the confining effect is reduced when intermittent strips are used and that the confining effect depends on the spacing of the strips (or spirals) (Saadatmanesh et al., 1994; Nanni and Bradford, 1995). Equations to estimate the confinement effectiveness of intermittent hoop strips (or straps) can be found in Saadatmanesh et al. (1994) and Mander et al. (1988b). As described in the previous chapter it enhances the concrete strength in the compressive zone of the cross-section.

4.2 Load-Moment-Curvature relationship (sectional behavior)

The nonlinear behavior of a concrete member is characterized mainly by the load-moment-curvature $(P-M-\Phi)$ relationship of its cross section. An analytical model that determines the $P-M-\Phi$ diagrams including the inelastic structural response of rectangular cross section under combined uni-axial bending and axial load is presented in this section. As the intermediate step to determine its load-deflection behavior, load capacity and failure mode under a particular set of loads and boundary conditions. Extensive experimental and analytical studies on the moment-curvature and load-deflection responses of reinforced concrete beams and columns are continuously reported since the 1960s worldwide.

A typical moment-curvature curve for different levels of axial load is given by MacGregor (1997). The analytical methods developed to calculate the moment-curvature relationship are based on the application of the basic principles of equilibrium of forces and strain compatibility, assuming idealized but realistic stress-strain relations to represent the behavior of the reinforcing steel and concrete. Due to the nonlinear nature of these stress-strain relations, the use of numerical approach rather than closed-form solution has been adopted.

The proposed model uses the following assumptions:

1- Plane sections remain plane after bending. There is no twisting.
2- Strain in the reinforcement equal the strain in the concrete at the same location (i.e. no slip occurs between reinforcing steel and concrete).
3- Material laws are known and not path-dependent.
4- Axial loads are held constant while moments are increased to failure.

Discussion of the validity and adequacies of these assumptions can be found in standard textbooks such as Macgregor (1997).

The most fundamental requirement in predicting the Moment Curvature behavior of a flexural member is the knowledge of behavior of its constituents. With the increasing use of higher-grade concretes, the ductility of which is significantly less than normal concrete, it is essential to confine the concrete. In a flexure member the hoops reinforcement used to resist the shear force and confines the concrete in the compression zone. Hence, to predict the Moment Curvature behavior of a flexural member, the stress–strain behavior of confined concrete in axial compression is essential. With the development of performance-based design codes, there is an increasing need for simplified but reliable analytical tools capable of predicting the flexural behavior of reinforced concrete members beyond elastic zones. Design offices are faced more and more with the need of predicting the deformation capacity of concrete members. Amplifying the need of general approach to account for confinement of concrete and predicting the total flexural behavior of concrete member.

To define moment curvature relation, material models for concrete and steel must be defined previously. Many mathematical models for concrete are currently used in the analysis of RC structures. Among these models, modified Hognested curve for concrete stress Figure 4.2, however, the stress strain relation in the tensile region is ignored in strength calculations in this study, because concrete has low tensile strength, generally less than 20% of the compressive strength, and it makes a negligibly small contribution to the strength and energy absorption capacity of a cracked RC section.

4.2.1 Simplified moment curvature:

To ensure ductile behavior in practice, steel contents smaller than the balanced design value are always used for flexural members. Typical moment curvature relation for a lightly reinforced concrete section with one top and one bottom layer of reinforcing steel (under-reinforced concrete section) can be idealized by tri-linear relation as shown in Figure 4.3.The first stage up to cracking, the second to yield of the tension steel, and the third to the limit of useful strain in the concrete. The moment and curvature at first yield of the tensile steel should also be calculated using the defined stress strain relations for concrete (Figure 4.2) and steel (Figure 4.5). Based on normal force equilibrium, a section analysis is carried out by assuming that tension steel reaches the yielding point. (1) During section analysis, for each value of curvature, neutral axis should be updated until the difference between internal tensile force T and compressive force C is less than a given tolerance. While final steel strain is less than yielding strain, the value of curvature can be increased and iteration is repeated again. Through successive iteration, the moment and curvature are determined up to yield of steel. The connection from initial point to yield point gives the simplified basic moment curvature relation. In advance, the moment curvature relation to in post yielding stage is approximated with a straight line which conserves constant energy until ultimate point Figure 4.3.

4.2.2 Analytical moment curvature relationship M-φ

In deriving the expressions of moments and curvatures for concrete section confined with rectilinear ties, the following assumptions are made:

a. Stress-strain relationship proposed in a selected model is taken as a stress block.
b. The tensile strength of concrete is neglected.
c. The variation of strain across the section is linear up to failure.
d. Idealized stress-strain relation for the tension and compression steel is considered.
e. The steel is perfectly bonded.
f. An imaginary leg of stirrup is considered at neutral axis to simulate the tri-axial state of stress in compression concrete.

In addition to above assumptions, the three basic relationships:

(i) Equilibrium of forces
(ii) compatibility of strains and
(iii) Stress-strain relationships of materials are respected.

4.3 Moment curvature using a computer program

A computer program is developed here after to definet the moment curvature relationship of reinforced concrete cross-section subjected to bending moment and axial force the source code of this software uses C language with Borland C compiler.

In this model the cross-section is divided into small filaments (1mm each) and for each filament stresses and strains are calculated Figure 4.6. For a given strain the extreme fiber in compression, the depth of the neutral axis is found by trial. For each filament, average stresses are calculated at the centroids of the filament .By doing so the first strain at filament is determined using the compatibility requirements. This centroidials strains is later used with concrete model to calculate the stresses acting on it. Finite concrete forces in the filaments are calculated are found by multiplying the stresses by the corresponding area $C = \sum_{i=1}^{n} f_{ci} * A_i$. Stress in the reinforcement at a given level is found from the stress-strain diagram of steel with the strain value found from the compatibility requirements. Force in steel at that level is found by multiplying the stresses found with the area of reinforcement at this level $T = f_s * A_s$ as shown in Figure (4.6)

4.3.1 Numerical Procedure

The steps of solution used in the numerical approach are as follow (Figure 4.7):

1. The curvature ϕ is increases at each loop
2. Divide the concrete into strips (filaments).
3. The N.A. location is assumed.
4. Calculate the strain in concrete and steel are carried
    a. $\varepsilon_c = \phi * y_i$
    b. $\varepsilon_s = \varepsilon_c * \frac{d-\bar{y}}{\bar{y}}$
5. Calculate Stresses from Stress Strain curves of Steel and confined Concrete (chapter 3) for each strip
6. Calculate forces
    a. $C = \sum_{i=1}^{n} f_{ci} * A_i$
    b. $T = f_s * A_s$
7. Calculate the internal forces P_{int}=C+T and compare it with external
8. If equilibrium is obtained ($P_{ext}=P_{int}$)
    a. Calculate Moment
        i. $M = P_{ext.} * \left(\frac{t}{2} - cover\right) - \sum_{i=1}^{n} C_i * y_i$
    b. Increase the curvature for new increment (step 1)
9. Otherwise iterate by increasing y until equilibrium of forces is obtained (step 3).

4.4 Moment-Curvature Relationships of Jacketed Sections

Results of the numerical model developed are given in section. Different parameters have been studied to determine the effect on global strength and ductility of confined column .thickness of confining layer, type of FRP, value of external load (as percentage of ultimate capacity)have been studied for both square and rectangular columns. Figure 4.8 and 4.9 shows the moment-curvature relation for RC square column (500x500mm) with reinforcement A_s =0.01A_c steel 52 and hoops 5Ø8/m', subjected to axial load P_{ext}=0.3P_u .Three cases are compared the unconfined condition, glass fiber and carbon fiber polymers. The results shows that confinement using CFRP increase the ultimate moment of the column by 40%, while GFRP increase the maximum moment about by about 25% compared to normal column confined with hoops only. The ultimate curvature is also increased by 80% and 40% for carbon and glass respectively (Figure4.8 and 4.9)

Study the effect of FRP jacket thickness several calculations were carried using CFRP and GFRP. The moment-curvature relations for the square columns are given in Figure 4.9 and Figure 4.10 and in the unconfined condition, when wrapped with a CFRP jacket of $2.0\,\mathrm{mm}$ thickness and when wrapped with a CFRP jacket of $4.0\,\mathrm{mm}$ thickness. The same calculation is presented in Figure 4.10 for GFRP jacket. Once again, a fundamental difference is observed: the moment-curvature relations of the FRP-confined concrete column cross sections show an increasing ultimate deformation, as opposed to the steel-confined one, which, after reaching the peak strength, decays in a softening branch. In both cases, the Figures reveal that doubling the jacket thickness has resulted in a limited increase in the strength, but a significant enhancement in the ductility of the column sections is obtained.

The moment-curvature relations for the rectangular column shown in Figure 4.12 three cases are studied. The unconfined condition, when wrapped with a $4.0\,\mathrm{mm}$ thick CFRP jacket and when wrapped with a $4.0\,\mathrm{mm}$ thick GFRP jacket. The external jacketing using CFRP improved the moment capacity of the section more than the case of GFRP. On the other hand, the deformability of column sections externally confined with GFRP jackets is larger than those externally confined with CFRP jackets Figure 4.11. This is attributed to the fact that the ultimate strain of GFRP-confined concrete is higher than that of CFRP-confined concrete. The effect of confinement is less pronounced for rectangular column than square one, the effectiveness of confinement at different level of external loading is investigated for loads equal 10%, 30% and 50% of ultimate capacity Figure 4.12.

Two different issues, first the ultimate moment capacity is increased for higher external load; this can be the load that the overall compression force produced by the external load reduced the tension force in the steel reinforcement allowing it to resist the applied moment. This behavior doesn't depend on the confinement and can be distinguished separately from the increase of ultimate moment from non-confined column. Second the ultimate the ultimate moment is increased for higher external load due to the increase of compressive strength of confined concrete (in compression zone) this is more pronounced for large external loads as the applied moment(M)increase linearly with the external load(P), for the same eccentricity (e) (M=P.e),in this case the ultimate section moment capacity increase for higher external load due to the increase of strength of confined concrete for large confinement level.

4.5 Interaction diagram

The interaction diagram or "failure envelope" of a reinforced concrete cross section contains the different combination of M and P that results in the failure of the cross section as shown in Figure 4.13Thus the interaction diagram is a graphical representation of all possible combination. In order to develop the inter action diagram one has to know the concrete dimensions of the section, the longitudinal reinforcement, f_{cu} and f_y.

Concrete section may be subjected to eccentric force compression or tension. The behavior of the section under the combination of axial force and moments due to eccentricity depends on the magnitude of the moment M_u and axial force P_u .if M_u is relatively small compared to P_u,the eccentricity e will be small due to the section will be subjected to a small eccentricity. In this case, most of the section is subjected compression and column behavior will be dominated. On the other hand if the M_u is large compared to the applied axial load eccentricity e will be large so that axial load is outside the section (Figure 4.14).

The design of reinforced concrete sections requires the specification of a number of parameters such as the section width and depth, the area of steel the strengths of the concrete and steel. Where a section is subject to both axial force and bending moments about one or both axes, interaction diagrams are commonly used to determine the area of steel required to resist the moments and forces to which the section is subjected.

Developing the inter action diagram of a concrete cross section can be achieved through the application of:

1- Compatibility of strains

2- Equilibrium of forces and moments.

4.5.1 Modes of failure:

Figure 4.14 presents a series of strain distribution and the resulting points on the interaction diagram. The state of stress developed in the concrete and steel controls the type of failure. The modes of failure are explained as follows.

4.5.1.1 Compression failure

Point A corresponds to the case of pure axial loading point B represents the case of compression failure mode where the concrete reaches its ultimate strain and tension steel does not yield. This failure mode is brittle, as the column fails as soon as the concrete reach its ultimate strain without large deformation or sufficient warning.

This failure belongs to column with relatively small bending moment.

4.5.1.2 Balanced failure

At this point the concrete reaches its ultimate strain at the same time the steel reach it yield. The maximum bending moment for the section occur at this point.axial Loads larger than the balanced load cause compression failure and load smaller than the balanced load cause tension failure

4.5.2.2 Tension failure

This represents by point D on the interaction diagram. The strain in the steel is larger than the yield strain and steel will reach yielding stress. This is a ductile mode of failure since the section will crack and develop a large deformation before failure, thus giving a sufficient signs of warning before the complete collapse. This behavior is similar to that of beam rather than columns. If the axial force is equals to zero. The section will reach the case of pure bending represented by point E. both the axial loads and moment capacity increase in this zone up to balance point.

4.6 Development of interaction diagram:

4.6.1 Full range interaction diagram

The relationships needed to calculate different points on interaction diagram are driven using the principles of strain compatibility and equilibrium of forces. The following procedures are applied to evaluate a full of combination modes of failure rang interaction diagram:

- The outer most stress in the compression fiber is assumed to equal 0.003(for unconfined concrete).
- The neutral axis is assumed and strains are calculated.
- The stresses are then calculated using the strain developed.
- Increment the distance of the neutral axis until equilibrium of forces is obtained

the following equations is used to develop the interaction equation

- $P_u = C_c + C_s - T$

  Where C_c is the force in the compression zone which is the area of each strip under compression multiplied by the stress applied on it .C_s is the force of the compression steel. T is the force in the tension steel.
- $M_u = C_c \times y_c + C_s \times y_s - T \times y_t$

  Where P_u is axial load, M_u bending moment calculated at plastic centroids and y_c distance from compression force to plastic centroid ,y_s distance from compression force in compression steel to plastic centroid and distance from tension steel force to plastic centroid y_t.

4.6.2 Interaction diagram using moment curvature

An axial force has an important effect on the moment curvature relation of an RC section Figure 4.15 shows the moment curvature relations corresponding to various levels of axial force. Up to reaching the ultimate axial force P_u, the yield moment of a section increases in proportion to the axial load P and the failure curvature considerably decreases by the presence of the axial load. Same procedures as calculating the moment curvature of the section but with the variation of the axial load from 0 to P_u (max column capacity).

4.6.3 Simplified method to draw interaction diagram

Simplified interaction diagram also known as four point interaction diagrams is the simplest method to draw the interaction diagram as we calculate only four points from the full range combination (P-M) that gives us an approximate shape of the diagram as shown in Figure 4.16 .

Point A: pure axial load

$P_u = A_c \times f_{cu} + A_s' \times f_s + A_s \times f_s$ $\quad$ $M_u = 0$

Point B: Axial load with minimum eccentricity (0.05t)

$P_u = P_u$ $\quad$ $M_u = P_u \times e_{min}$

Point C: Balanced point

$P_u = C_c + C_s - T$ $\quad$ $M_u = C_c \times y_c + C_s \times y_s - T \times y_t$

Point D: Pure Bending

$P_u=0$ $M_u=C_c \times y_{ci}+ C_s \times y_s - T \times y_t$

4.7 Interaction diagram using confined concrete:

Reinforced concrete structural members such as columns and beams can be confined with FRP systems to increase their axial load-carrying capacity. Axial strengthening is most suitable for circular non slender (i.e., short) reinforced concrete columns. Combined axial and flexural strengthening of short eccentrically loaded reinforced concrete columns will increase their axial and flexural capacity. An axial load-bending moment (*P–M*) strength interaction diagram can be constructed for an FRP-strengthened reinforced concrete column in a fashion similar to that of a non-strengthened column.

Axial strengthening is obtained by applying the FRP system oriented such that its principal fiber direction is in the circumferential (or hoop) direction of the member, perpendicular to its longitudinal axis. In addition to providing axial strengthening, hoop FRP reinforcement provides shear strengthening to the member, since it is oriented perpendicular to the member axis. As noted previously, when strengthening for only a single mode is intended, it is incumbent on the designer to determine the effects of the strengthening on the other modes and to ensure that the member has sufficient capacity in the other modes to resist the higher applied loads.

Confinement provided by FRP increases the strength and strain capacity of the concrete. When the column is loaded with concentric load, it can be safely assumed that FRP will provide a guaranteed increase in the strength capacity of the column. When the columns are subjected to eccentric load or a contribution of axial force and moment, the strains at the opposite sides of the column along the axis of the moment are not the same. The applied moment induces compression on one side and tension on the other side. As the magnitude of moment increases, the strain in the tension side keeps decreasing. In a typical reinforced concrete column, the increase in eccentricity is assumed to occur until the beam-column becomes a beam. In other words, the column could act as a beam loaded in the lateral direction. In the case of columns confined with FRP, allowing excessive tensile strain may not be advisable because excessive cracking on the tension side might reduce the confinement effect.

A computer based program used to develop the full range of the confined concrete interaction diagram using the stress strain curve of confined concrete as illustrated in chapter 3 using Mander confining model and the simplified steel stress diagram for and rectangular and square section uses any kind of FRPs.

The ultimate compressive force and the ultimate bending moment for a concrete column cross section are related by the interaction diagram or the failure surface. The moment-curvature curves can be used to develop the axial load-bending moment interaction envelopes for rectangular columns confined with FRP and steel hoops. Such failure envelopes are essential tools for judging the capacity of a retrofitted short column under the action of eccentric loads. Figures show the load-moment interaction diagrams for the square column cross sections shown in Figure 4.17, Figure 4.18and Figure 4.19,shows the behavior in case of rectangular section, in the unconfined conditions as well as after being wrapped using a CFRP jacket of 4.0mm thick. It can be seen that, on the section level, a tremendous improvement in the capacity of the sections is obtained for load levels above the balanced ones. This would be expected since large axial loads improve in the concrete strength by increasing its to be noted that the enhancement of FRP confinement is not huge in case of under reinforced concrete section, as overall section capacity in bending is governed by the steel failure rather than the concrete failure confinement level. It can also be observed that the improvement in the capacity of the square column is more significant than that of the rectangular one due to the fact that efficiency of external jacketing decreases with the increase in the degree of rectangularity of the cross section.

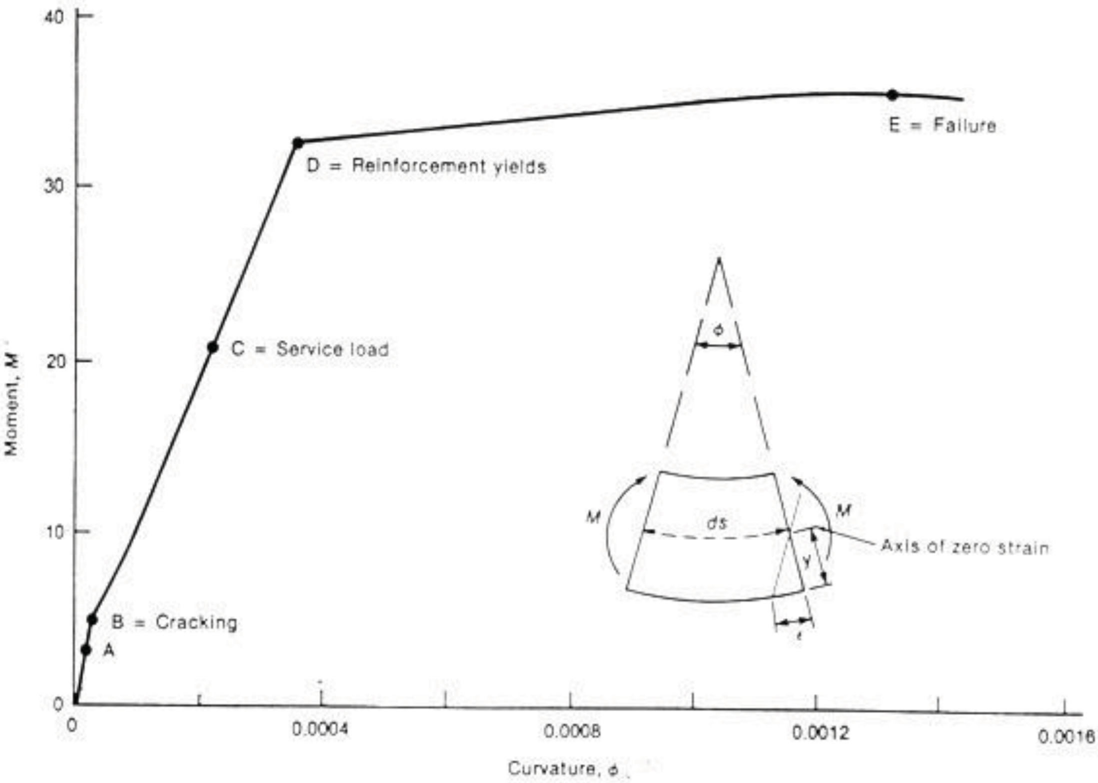


Figure 4.1 Moment curvature for reinforced concrete member

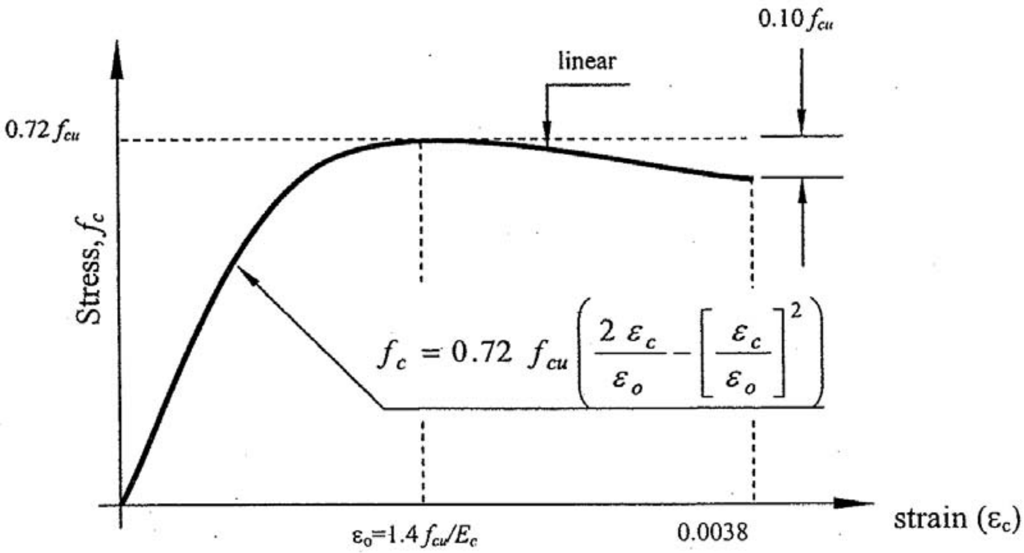


Figure 4.2 Modified Hognested stress strain curve for concrete

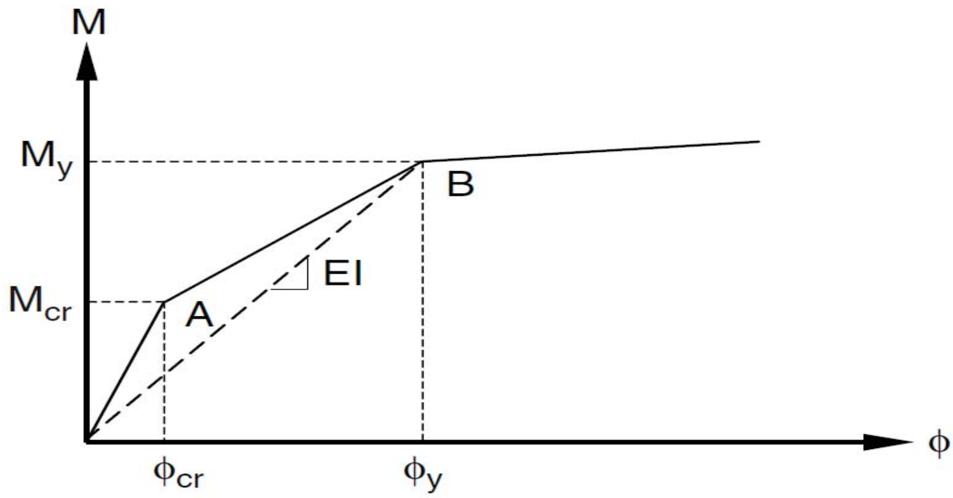


Figure 4.3 Simplified moment curvature for reinforced concrete member

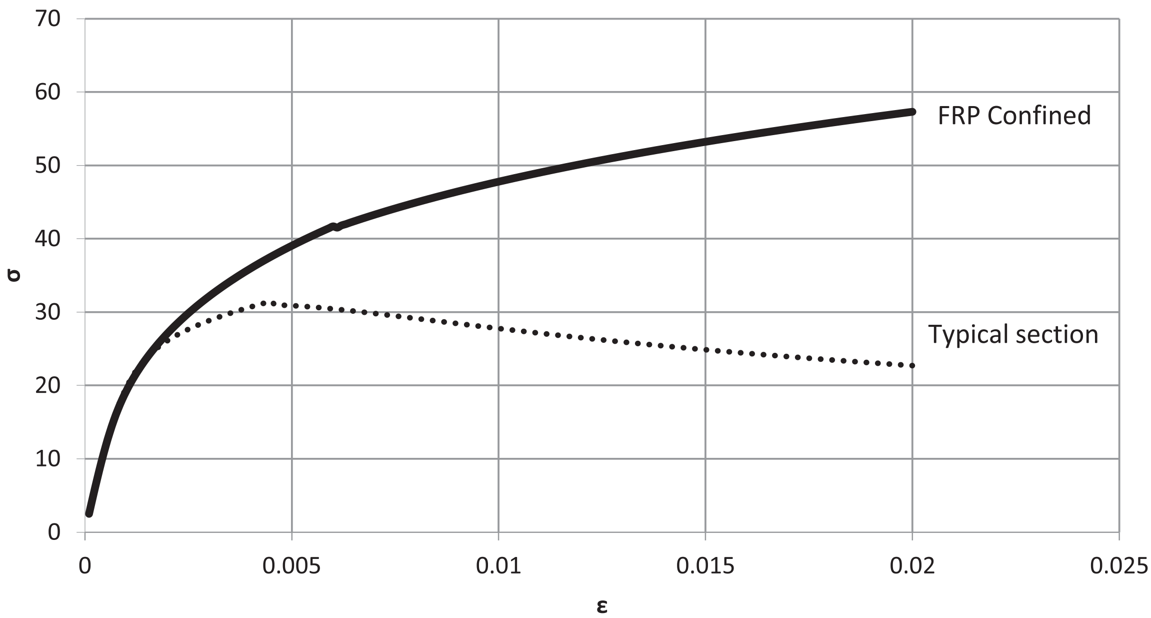


Figure 4.4 Stress-strain diagram of FRP confined and RC concrete

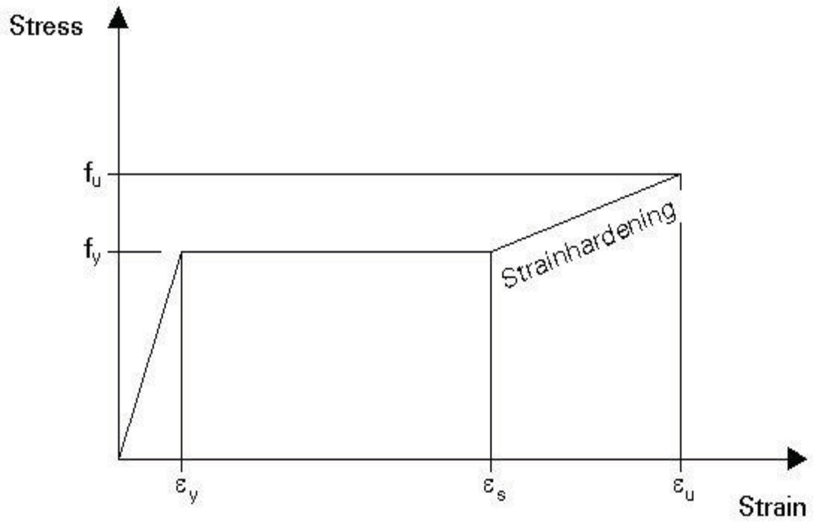


Figure 4.5 Tri-axial stress-strain diagram of steel

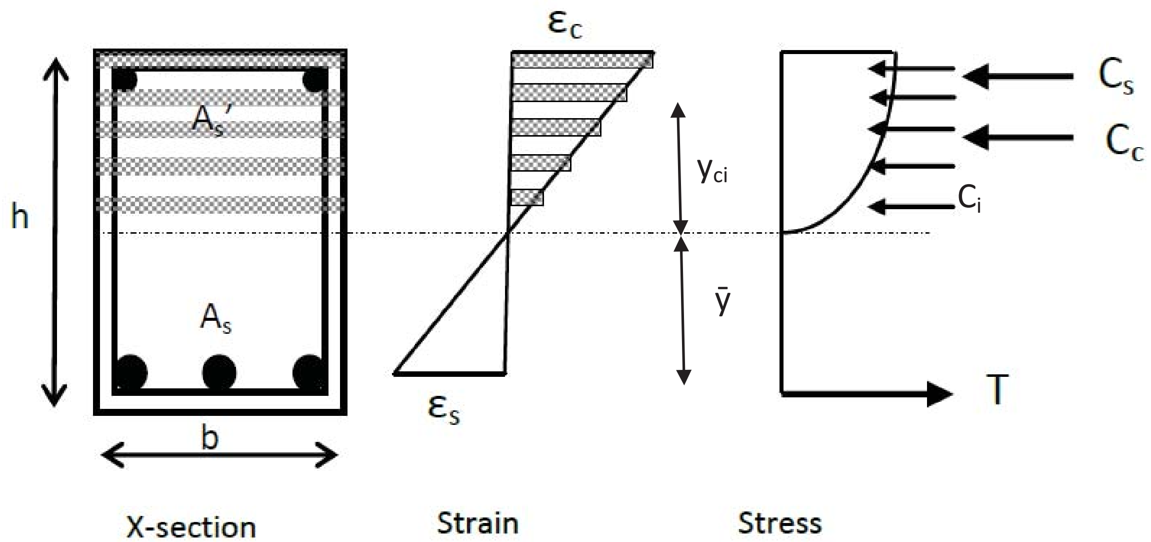


Figure 4.6 Strains and forced for a concrete member (filament approach)

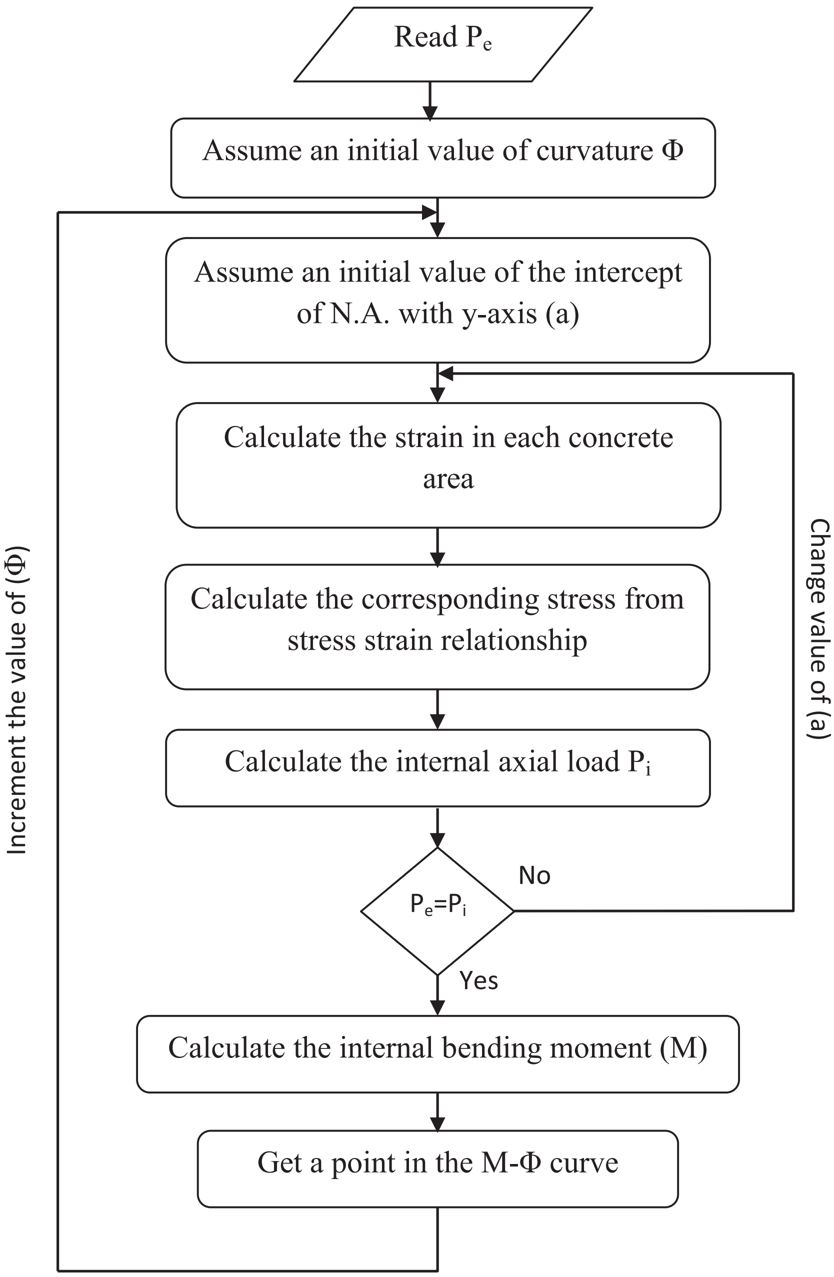


Figure 4.7 Flow chart of constructing a M- φ diagram

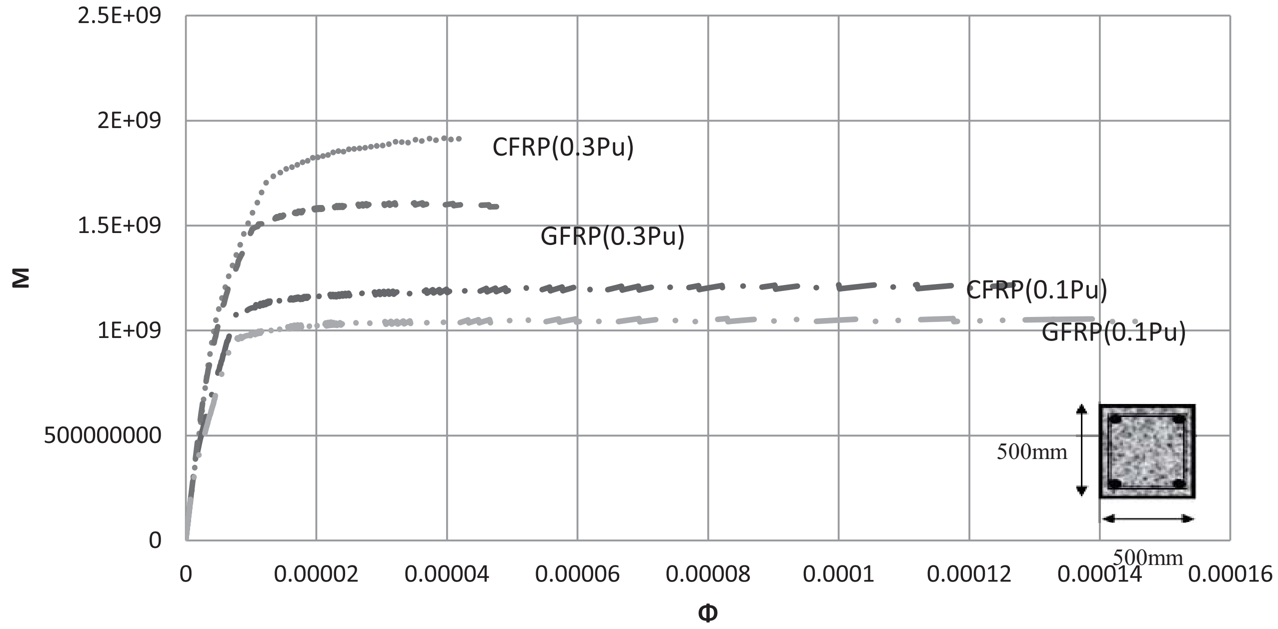


Figure 4.8 Moment curvature of concrete confined with CFRP and GFRP square section tj=4mm

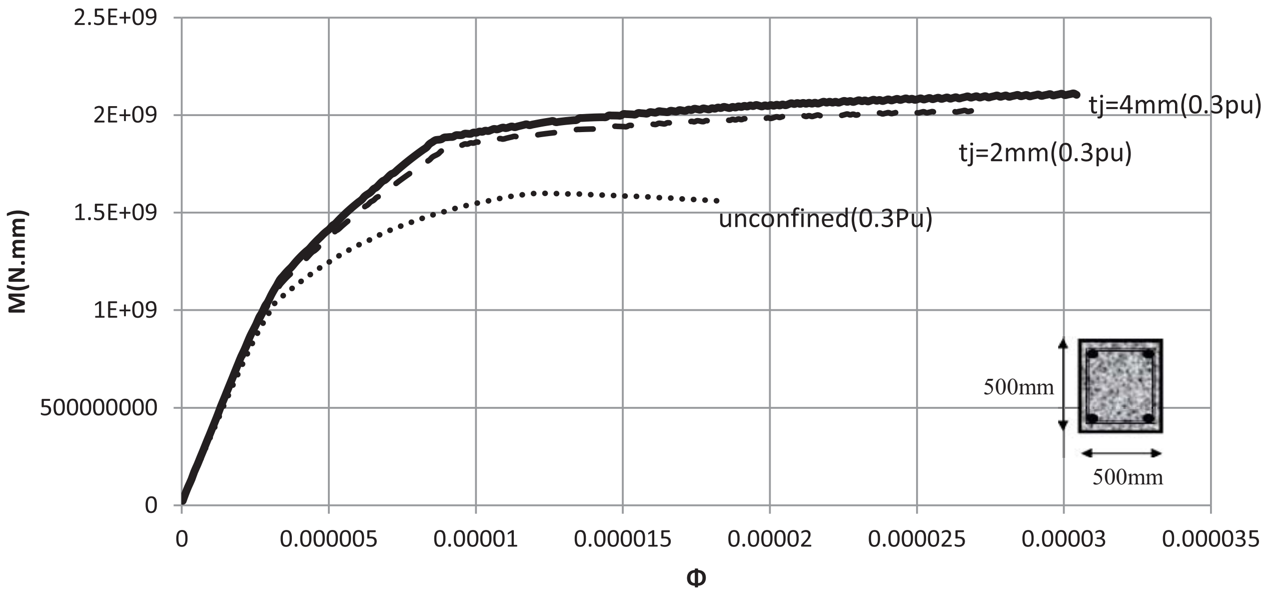


Figure 4.9 Moment curvature of concrete confined with CFRP square section

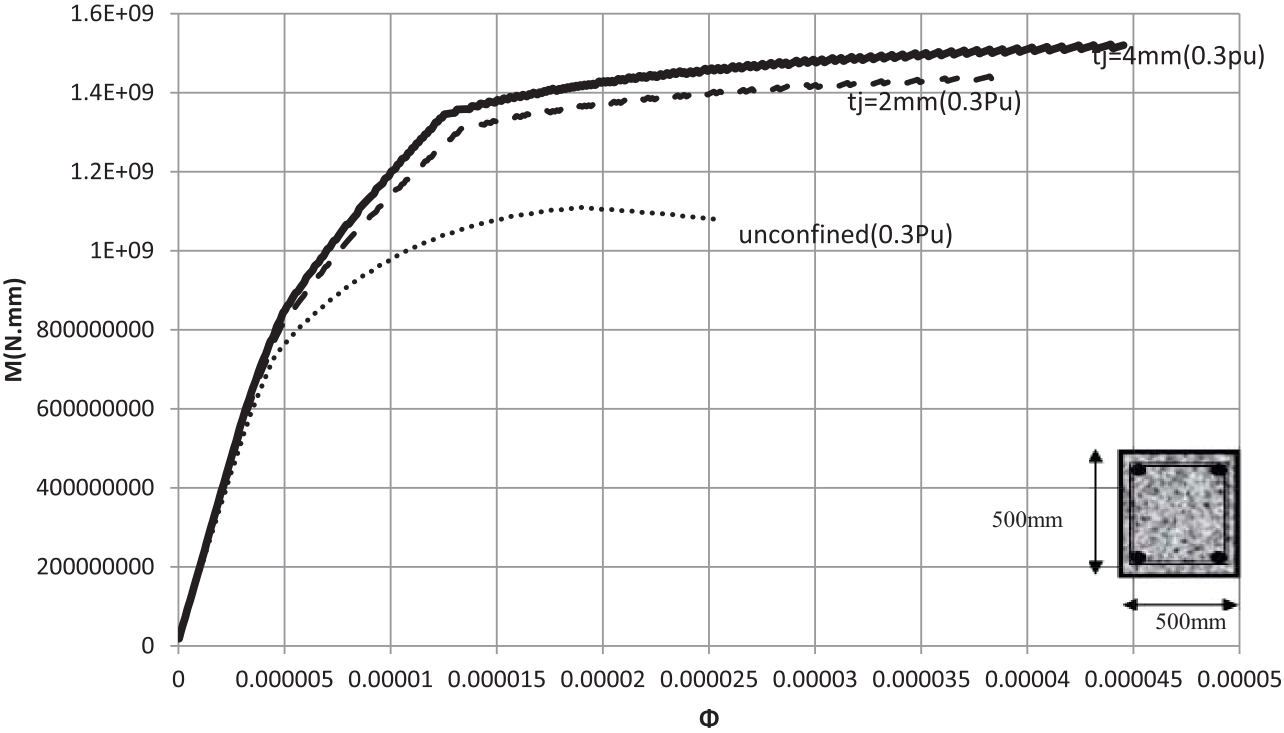


Figure 4.10 Moment curvature of concrete confined with GFRP square section

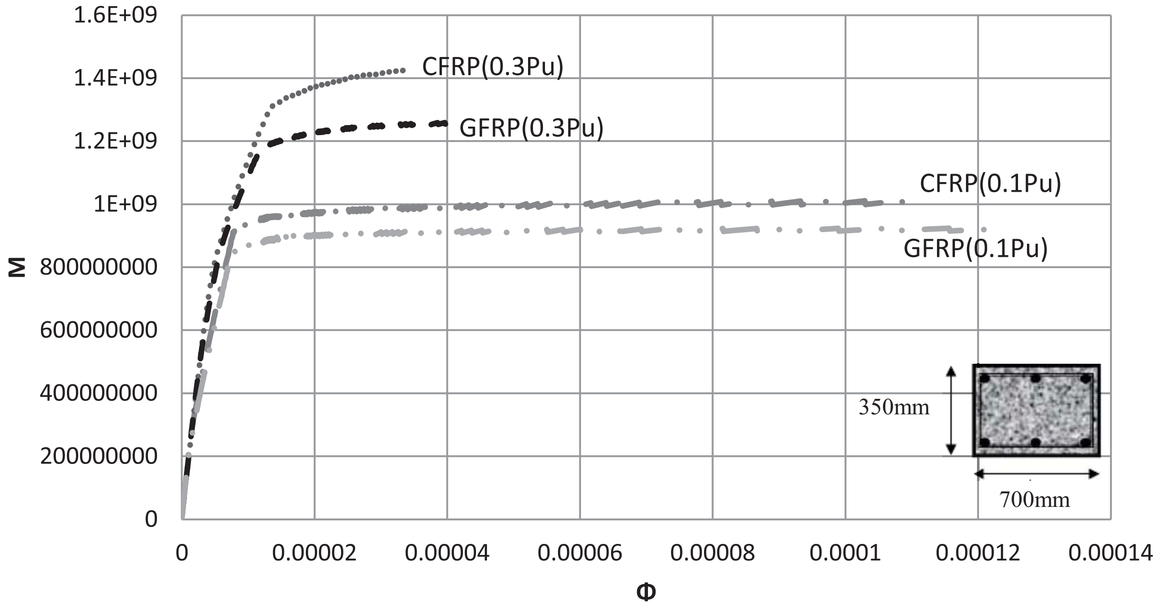


Figure 4.11 Moment curvature of concrete confined with CFRP and GFRP rectangular section tj=4mm

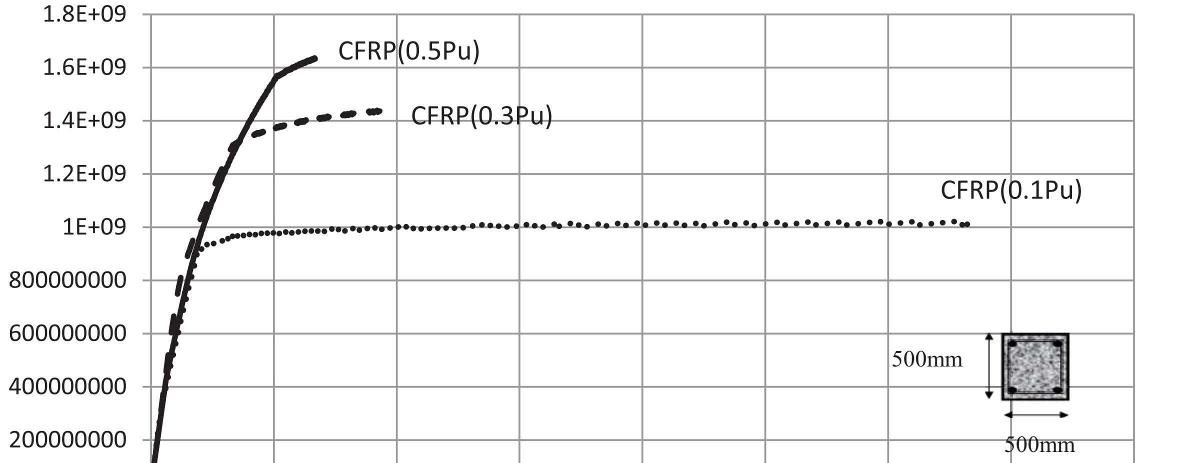


Figure 4.12 Moment curvature of concrete confined with CFRP square section under different axial load

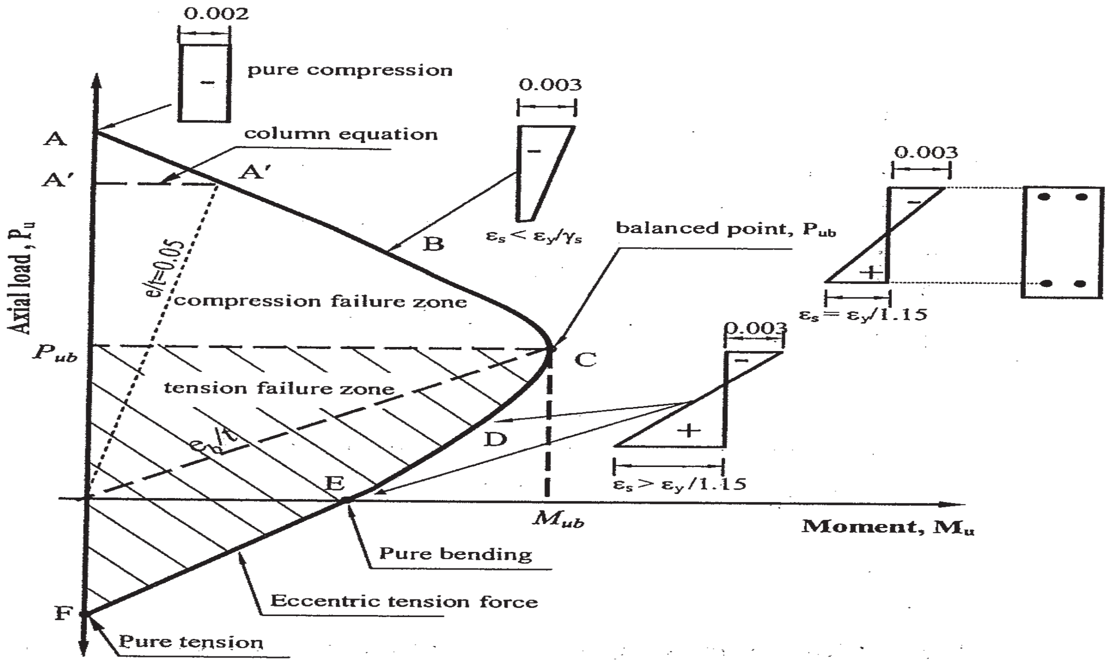


Figure 4.13 Full range load moment interaction Diagram

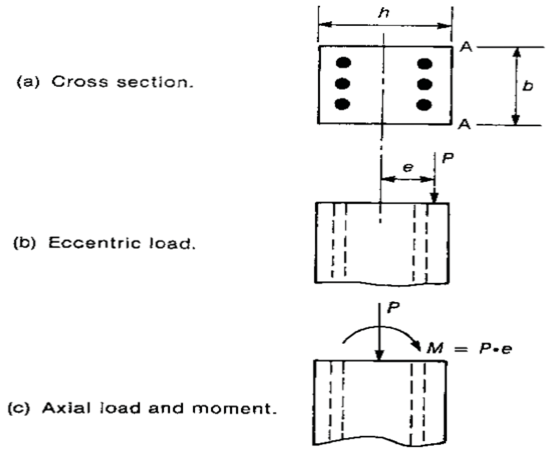


Figure 4.14 Load and moment on column

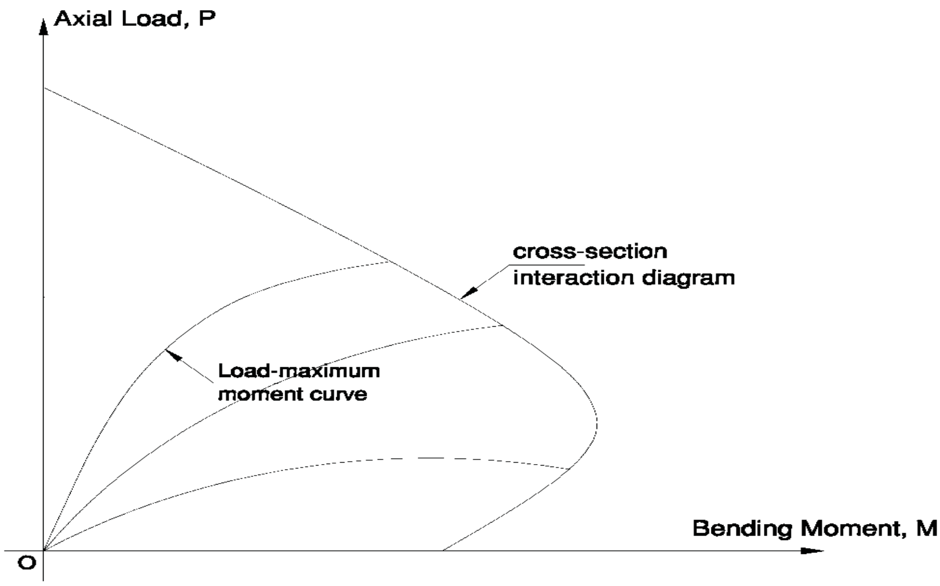


Figure 4.15 Interaction diagram using moment curvature

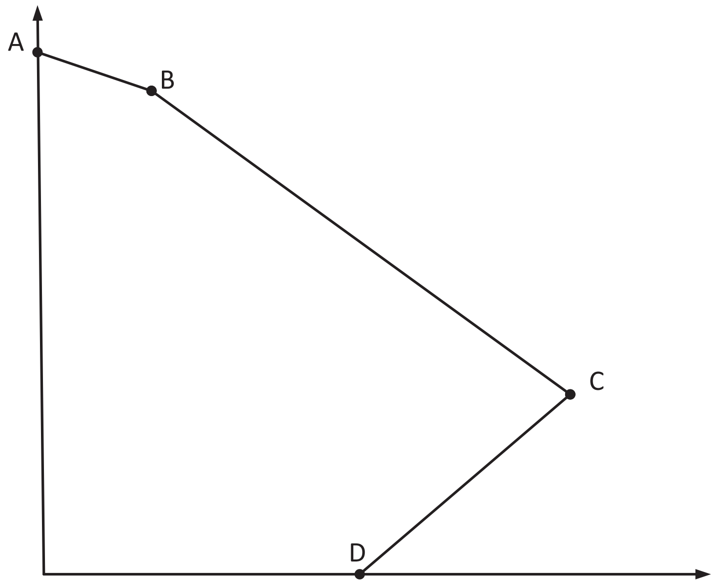


Figure 4.16 Simplified interaction diagram

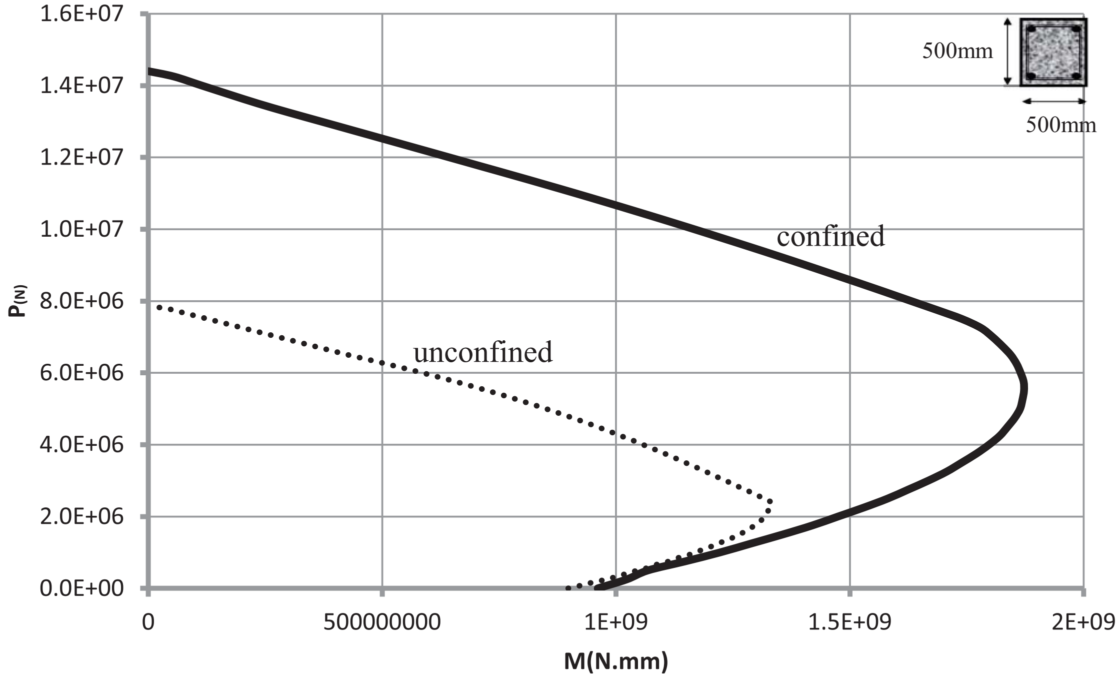


Figure 4.17 Moment load interaction diagram for confined and unconfined square column

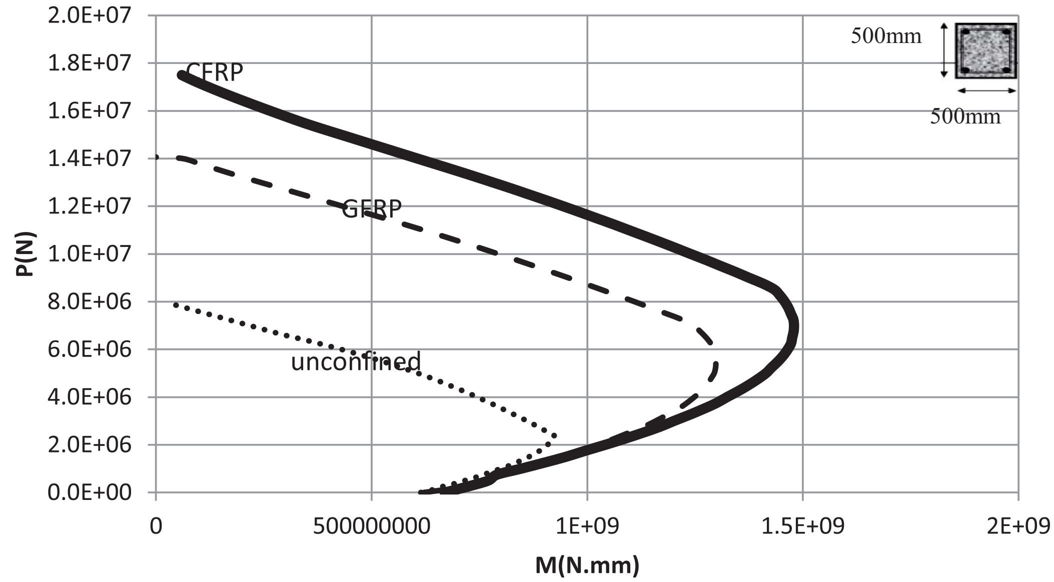


Figure 4.18 Moment load interaction diagram for CFRP and GFRP confined square column

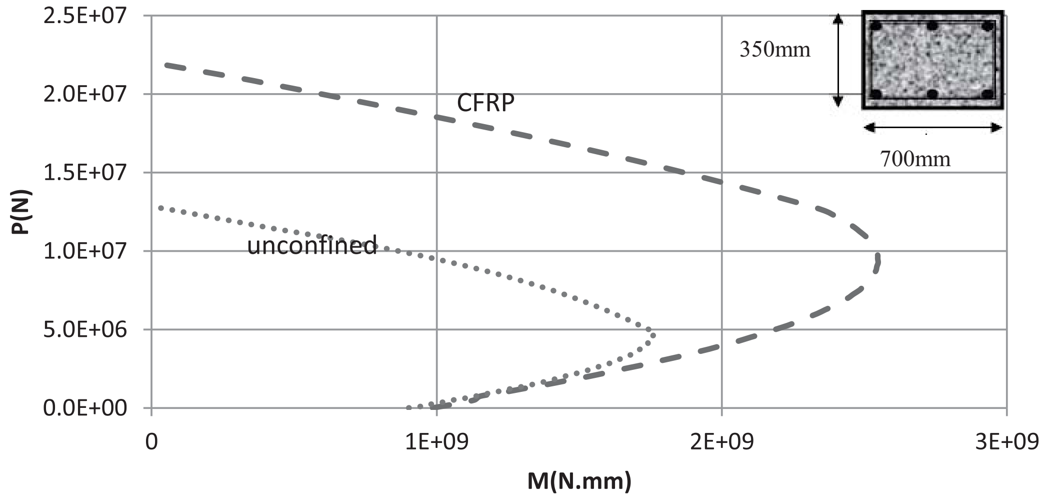


Figure 4.19 Moment load interaction diagram for CFRP confined rectangular column

CHAPTER 5

SEISMIC RETROFIT AND DUCTILITY ENHANCEMENT
FOR
RC COLUMNS

5.1 Introduction

Concrete columns are commonly poor in flexure, shear strength and flexure strength. A large number of retrofit techniques and schemes have been developed and tested. they include steel jacketing, active confining by wire pre-stressing, use of composite materials jacketing involving glass fiber, carbon fiber, or other fibers in epoxy matrix, as well jacketing with reinforced concrete.

In this chapter, the term ‘rehabilitation’ is used as a comprehensive term to include all types of repair, retrofitting and strengthening that lead to reduced earthquake effect on structure. The term ‘repair’ is defined as reinstatement of the original characteristics of a damaged section or element and is confined to dealing with the as-built system. The term ‘strengthening’ is defined as intervention that lead to enhancement of one or more seismic response parameters (stiffness, strength, ductility, etc.), depending on the desired performance.

The performance objectives are set depending on the structural type, the importance of the building; they can be specified as limits on one or more response parameter such as stresses, strains and displacements.

The selection of the rehabilitation scheme and the level of intervention is a rather complex procedure, because many factors of different environment come into play. A decision has to be taken on the level of involvement personal. Some common strategies are the restraint or change of use of the building, partial demolition and/or weight reduction, addition of new lateral load resistance system, member replacement, transformation of non-structural into structural components and local or global modification (stiffness, strength and ductility) of elements and system. In addition, methods such as base isolation, provision of supplemental damping and incorporation of passive and active vibration control devices may apply. The alternatives of ‘no intervention’ or ‘demolition’ are more likely the outcomes of the evaluation if the seismic retrofit of buildings is quite expensive and disruptive.

5.2 Column retrofit techniques

5.2.1 Steel jacketing

The technique was originally developed for circular columns. Two half shells of steel plates are positioned over the area of column to be retrofitted and are welded on site to form a continuous tube around the column with a small gap around the column. The gap is then grouted with a pure cement grout. While in rectangular columns it's recommended to use an elliptical jacket that provide a full continues confining action similar to the circular column

The steel jacketing option involves the total encasement of the column with thin steel plates placed at a small distance from the column surface, with the ensuing gap filled with non-shrink grout. An alternative to a complete jacket is the steel cage alternative. Steel angles are placed at the corners of the existing cross-section and either transversal straps or continuous steel plates are welded on them. In practice, the straps are often laterally stressed either by special wrenches or by preheating to temperatures of about 200–400C, prior to welding. Any spaces between the steel cage and the existing concrete are usually filled with non-shrink grout. When corrosion or fire protection is required, a grout concrete or shotcrete cover may be provided. The corrugated steel jacketing technique can be applied for the rehabilitation of columns and beam column joints.

The steel jacket is effective in passive confinement. The lateral stress is produced by the flexibility restrain as the concrete attempts to expand laterally in the compression zone from the effect of the high axial compression strain, or in tension zone due to the expansion of lap splice. Level of confinement depends on the stiffness of jacket.

5.2.2 Concrete jacketing

RC jacketing is one of the most commonly applied methods for the rehabilitation of concrete members. The longitudinal reinforcement placed in the jacket passes through holes drilled in the slab and new concrete is placed in the beam–column joint. The main advantage of the RC jacketing technique is the fact that the lateral load capacity is uniformly distributed throughout the structure of the building thereby avoiding concentrations of lateral load resistance, which occur when only a few shear walls are added. A disadvantage of the method is the presence of beams which may require most of the new longitudinal bars in the jacket to be bundled into the corners of the jacket. Because of the presence of the existing column, it is difficult to provide cross ties for the new longitudinal bars, which are not at the corners of the jacket.

The addition of thick layer of reinforced concrete around the column in the form of a jacket enhances flexural strength, ductility and shear strength of column. It has been used more frequently in building rather than bridges. It has been used in some japans bridges retrofit. By doweling the longitudinal reinforcement of the jacket in the footing with sufficient development length the column flexure strength can be enhanced, although this must be accompanied with footing retrofit measures to ensure plastic hinge development in the column.

5.2.3 Composite material jackets

The ease of application of FRP composites renders them attractive for use in structural applications; especially in cases where dead weight, space or time restrictions exist. Although FRP composites can have strength levels significantly higher than those of steel and can be formed of constituents such as carbon (CFRP), glass (GFRP), and aramid (AFRP) fibers, it is important to note that its use is often dictated by strain limitations. They are very sensitive to transverse actions (i.e. corner or discontinuity effects) and unable to transfer local shear (i.e. interfacial failure). Clearly, they carry no compressive forces. Choosing the type of fibers, their orientation, their thickness and the number of plies, results in a great flexibility in selecting the appropriate retrofit scheme that allows to target the strength hierarchy at both local (i.e. upgrade of single elements) and global (i.e. achievement of a desired global mechanism) levels. In general, FRP composites behave in a linear elastic fashion to failure without any significant yielding or plastic deformation. Additionally, it should be noted that unlike reinforcing steel, some fibers (such as carbon fibers) are anisotropic. This anisotropy is also reflected in the coefficient of thermal expansion in the longitudinal and transverse directions. The large differences in strength (transverse strength < longitudinal strength) and coefficients of thermal expansion can result in bond deterioration and splitting of concrete. Moreover, these can cause lateral stresses and low cycle fatigue under repeated thermal cycling.

The effectiveness of strengthening depends on the bond conditions, the available anchorage length and/ or the type of attachment at the FRP ends, the thickness of the laminates, among other less important factors. According to experimental data, failure of the FRP reinforcement may occur either by peeling off (de-bonding) through the concrete near the concrete–FRP interface or by tensile fracture at a stress which may be lower than the tensile strength of the composite material, because of strength concentrations (e.g. at rounded corners or at de-bonded areas). In many cases, the actual failure mechanism is a combination of FRP de-bonding at certain areas and fracture at others. The choice of constituents and details of the process used to fabricate the composite significantly affect environmental durability. Exposure to a variety of environmental conditions can dramatically change failure modes of the composites, even in cases where performance levels remain unchanged. In other cases, exposures can result in the weakening of the interface between FRP composites and concrete, causing a change in failure mechanism and sometimes a dramatic change in performance.

In the case of columns, shear failure, confinement failure of the flexural plastic hinge region and lap splice de-bonding can be accommodated by the use of FRPs At this juncture it is important to stress that none of these failure modes and associated retrofits should be viewed separately, since retrofitting for one deficiency may only shift the problem to another location and/or failure mode without necessarily improving the overall performance. For example, a shear-critical column, strengthened over the column center region with carbon wraps, is expected to develop flexural plastic hinges at column ends which, in turn, need to be retrofitted for the desired confinement levels. Furthermore, lap splice regions need not only to be checked for the required clamping force to develop the capacity of the longitudinal column reinforcement, but also for confinement and ductility of flexural plastic hinge. Shear and flexural strengthening of beams can be achieved by the application of either epoxy-bonded laminates or fabrics extending in the compression zone or epoxy bonded FRP fabric wrapped around the beam. In the case of beam–column joints, the jacket is designed to replace missing transverse reinforcement in the beam–column joint. The FRP technique can be also used for strengthening walls.

5.3 Columns retrofit design using FRP

5.3.1 Assessment of member inelastic rotation and ductility capacity

The plastic curvature $Ø_p$ is the difference between ultimate curvature $Ø_u$ corresponding to the limit compression strain ε_{cu} and the yield curvature $Ø_y$ thus

$$\text{Ø}_p = \text{Ø}_u - \text{Ø}_y \quad 5.1$$

The plastic curvature is assumed to be constant over the plastic hinge length L_p which is calibrated to give the same plastic rotation as occur in the real structure

According to Prestlye et al. 1996 a reasonable estimate for the plastic hinge formed against a supporting member is given by

$$L_p=0.8H+0.022f_{ye}d_{bl}\geq 0.044\ f_{ye}d_{bl} \quad 5.2$$

In equation 5.2 H is the distance from the critical section of the plastic hinge to the point of contraflexure and d_{bl} is the diameter of the longitudinal reinforcement

The plastic rotation is

$$\theta_p = L_p\,\text{Ø}_p = L_p\,(\text{Ø}_u - \text{Ø}_y) \quad 5.3$$

the limit curvatures Ø_u and Ø_y can be found through using a full moment curvature analysis . for simplicity the yield curvature can be approximated to

$$\text{Ø}_y=\frac{M_y}{E_c\,I_e} \quad 5.4$$

In which M_y is the yield moment, E_c is the modulus of elasticity of concrete and I_e is the effective inertia of cracked section.

From section analysis in the critical section the ultimate extreme fiber compression strain ε_{cu} is

$$\text{Ø}_u= \frac{\varepsilon_{cu}}{\bar{y}} \quad 5.5$$

Where $\bar{y}$ is the neutral axis depth

5.3.2 Retrofit for flexure ductility enhancement

With poorly confined columns that are expected to sustain large inelastic rotation in plastic hinge, a prime concern will be retrofit design to enhance the ductility capacity. Test on circular columns retrofitted with FRP jackets to improve ductility indicated that its confinement effectiveness is more efficient than with steel jackets Priestly et al.(1996) . It is thought that this is a result of the elastic nature of the jacket material. With a steel jacket, yield under hoop tension may occur early in seismic response. On unloading, residual plastic strain remain in the jacket reducing the effectiveness of the next cycle response. With GRRP and CFRP which have essentially linear stress-strain characteristics up to failure, there is no cumulative damage and successive cycles to the same displacement result in constant rather than increasing hoops strain.

Retrofit design according to Priestly et al (1996) presented a design procedure to enhance the ductility capacity. The retrofit design approach is based on displacement rather force consideration. In this approach the retrofit design attempts to provide the appropriate jacket thickness and height to achieve a specified displacement at the center of seismic force under the design seismic input.

The method is most simply explained by reference to a simple multi span bridge under transverse response. It relates the volumetric ratio of confinement to the required plastic rotation $\theta_{p.}$ The method with the theoretical background is summarized below (Figure 5.1).

1. The designer has to determine the ductility capacity μ_Δ of the retrofitted column

$$\mu_\Delta = \frac{\Delta p}{\Delta y} \qquad 5.6$$

Where Δ_y is the yield displacement and Δ_p is the plastic displacement corresponding to the column rotational capacity of the column hinge θ_p

2. The required plastic rotation θ_p of the plastic hinge is found from the expression

$$\theta_p = \frac{\Delta p}{H} \qquad 5.7$$

Where H is the distance from the critical section of plastic hinge to the point of contra flexure

3. The plastic curvature is found from the expression

$$\varnothing_p = \frac{\theta p}{Lp} \qquad 5.8$$

Where the plastic hinge length is calculated by

$$L_p = g + 0.044 f_{ye} d_b \qquad 5.9$$

Where g is the gap between the jacket and the supporting member equation (5.9) was proposed by Priestly et. al.(1996) to take into account the concentration of plasticity at the gap due to the existence of the jacket.

4. The maximum required curvature is

$$\text{Ø}_m = \text{Ø}_y + \text{Ø}_p \qquad 5.10$$

The yield curvature can be approximately calculated by

$$\text{Ø}_y = \frac{M_y}{E_c\, I_e} \qquad 5.11$$

In which M_y is the yield moment, E_c is the modulus of elasticity of concrete and I_e is the effective inertia of cracked section.

5. The maximum compression strain is given by

$$\varepsilon_{cu} = \text{Ø}_u\, c \qquad 5.12$$

Where c is the neutral axis depth that can be obtained from flexure strength calculation.

6. The volumetric ratio of confinement required ρ_j is given by

$$\rho_j = f_j\ \varepsilon_{cm} \qquad 5.13$$

where f_j is a material dependant relationship between ultimate compression strain and volumetric ratio of jacket confinement.

The following equation has been experimentally derived for circular columns retrofitted with steel jackets. In order to take into account the fact that FRP-confined concrete can reach ultimate compression strain value more than those reached by steel confined concrete, Priestly et. al.1996 suggested that in case of FRP-confined concrete the ultimate crushing strain can be given by

$$\varepsilon_{cu} = 0.004 + \frac{2.5\, \rho_{ju} \varepsilon_{ju}}{f_{cc}} \qquad 5.14$$

Where f_{uj} and ε_{uj} are the ultimate stress and strain of the jacket material and f_{cc} is the confined concrete given by equation (3.21) and ρ_j is the volumetric ratio of confinement for circular column diameter D is expressed by

$$\rho_j = \frac{2\,t_j}{D} \qquad 5.15$$

where t_j is the jacket thickness

Equation (5.14) and (5.15) can be combined together and solved for the required jacket thickness as

$$t_j = \frac{0.1\,(\varepsilon_{cu}-0.004)Df_{cc}}{f_{ju}\varepsilon_{ju}} \qquad 5.16$$

Very few tests were done on rectangular columns retrofitted with rectangular FRP jacket wrapped directly onto the existing column Priestly et. al. (1996) postulated that for rectangular columns the jacket effectiveness could be assumed 50% of that of circular ones and the equation will be modified to

$$\varepsilon_{cu} = 0.004 + \frac{2.5\,\rho_{ju}\varepsilon_{ju}}{f_{cc}} \qquad 5.17$$

Where for a rectangular jacket the volumetric confinement ρ_j is expressed as

$$\rho_j = \frac{(\varepsilon_{cu}-0.004)f_{cc}}{1.25\varepsilon_{ju}\,f_{ju}} \qquad 5.18$$

Equations 5.16 and 5.17 can be combined together and solved for the required jacket thickness for a rectangular column to achieve a target ultimate compression strain

$$t_j = \frac{(\varepsilon_{cu}-0.004)f_{cc}}{2.5f_{ju}\varepsilon_{ju}}\left[\frac{b\,h}{b+h}\right] \qquad 5.19$$

It should be mentioned that’s in order to obtain the value of maximum required compression strain using equation (5.13) one have to perform moment curvature analysis . A perquisite for such an analysis is a uni-axial model for concrete confined with FRP in their approach .However Priestly et. al. (1996) allowed obtaining neutral axis from flexure strength calculations.

The previously described procedure was used to predict the ultimate displacement for columns tested in the investigation. The calculations of each column started with ε_{cu}calculated according to equation (5.14) for circular column and equation (5.17) for rectangular column used the provided jackets thickness t_j (Figure 5.2).

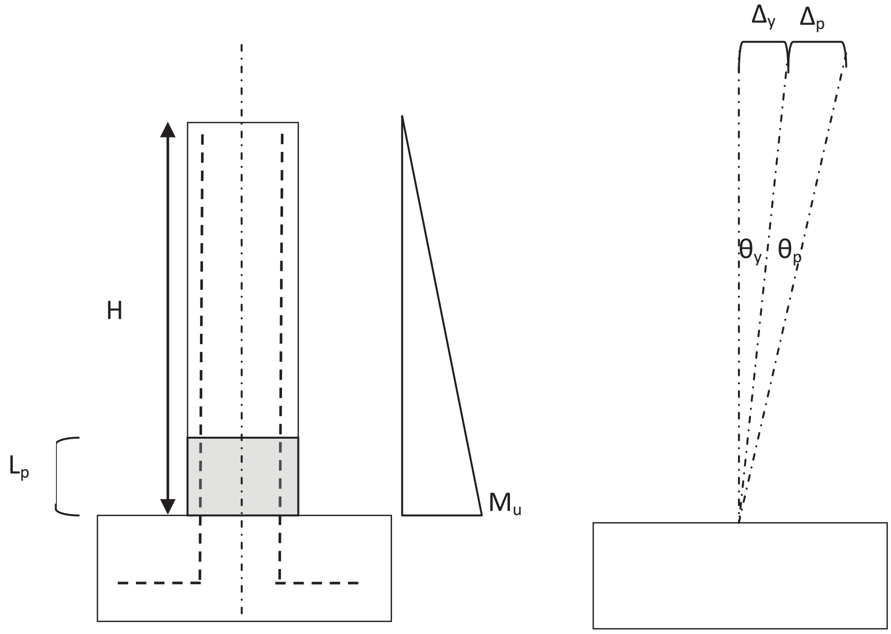


Figure 5.1 Inelastic deformation of model column

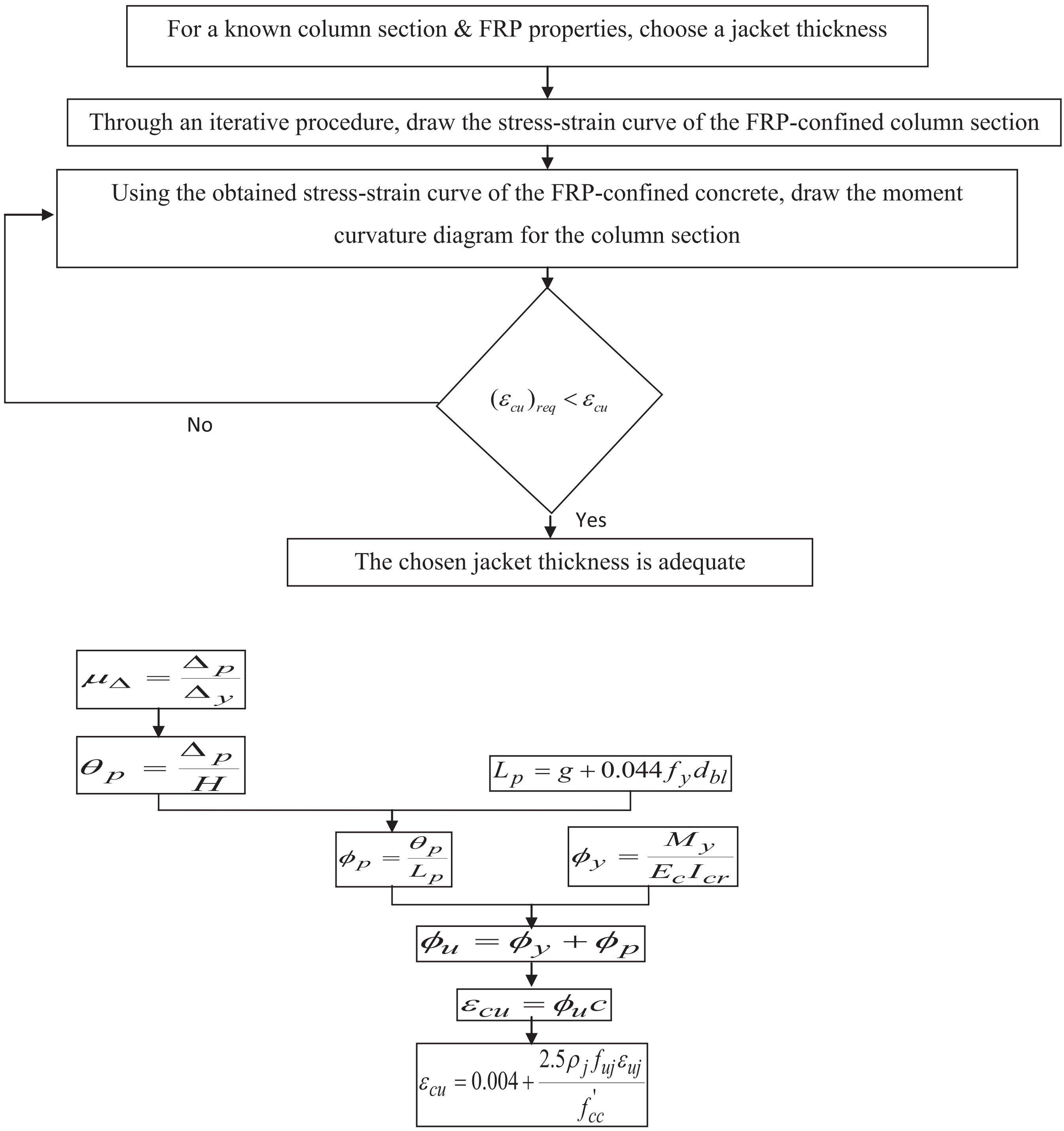


Figure 5.2 Flow chart diagrams shows the steps of design of FRP jacket

CHAPTER 6

PARAMETRIC STUDY

6.1 Introduction

For concentrically loaded RC members with circular cross-section, the lateral confinement of concrete results in a substantial increase in load capacity and ductility. FRP-confinement is less effective for RC compression members with square and rectangular cross-sections than for members with circular cross-sections due to stress concentration at the corners and lack of confinement on the flat sides.

The gain in load capacity of loaded RC members due to FRP-confinement in non-circular cross-sections is influenced by the aspect ratio of the cross-section, the FRP-material used, thickness of the raping and the compressive strength of concrete in the member. In the following chapter a parametric study will be discussed on variable circular and non-circular sections as shown in table 6.1.

6.2 Effect of cross section type on confinement

FRP materials have been used successfully to improve the strength and deformability of circular concrete columns with a geometrical configuration that allows the fibers to be uniformly stressed, thus providing a highly effective confinement for concrete across the entire cross-section as in Figure 6.1. For square and rectangular columns, the confinement effectiveness is much reduced due to the non-uniformity of the confinement pressure across the cross-section.

The aspect ratio of the cross-section has a significant effect on the gain in load capacity at low levels of FRP-confinement. The load carrying capacity decreases, with the increase of the aspect ratio of the cross-section. To increase the effectiveness of FRP-confinement in non-circular cross-sections, researchers recommended rounding the corners. hence a significant improvement in load capacity because it eliminate cross-section corners and flat sides, therefore improving the axial load capacity of compression members of square and rectangular cross-sections is affected by shape-modified to become close to a circular section. The more the rectangularity ratio increase the less the effective is their of the FRP confinement the area of unconfined concrete increase so the load capacity of the section remain unchanged see Figure 6.2. While in circular sections the effect of confinement is on the full section. As shown in Figure 6.3 the enhancement of stress-strain curves on a rectangular section due to confinement and in Figure 6.4 on a circular section.

6.3 FRP material used

Fiber-reinforced polymers (FRPs) are currently used as confining materials in structures. It has high strength-to-weight ratio, high confinement strength, easy to install and maintain, fatigue resistant, non-magnetic, non-metallic and durable. With respect to rehabilitation, strengthening and retrofitting of existing and deteriorated structures, FRP sheets have now become very popular devices in enhancing the performance of existing RC columns. Columns that are confined by both steel reinforcement and FRP, which are sometimes referred to as “hybrid RC columns”, have now become common in existing buildings and bridges.

Three types of FRP are commonly used. These are aramid fibers, glass fibers, and carbon fibers (CFRPs). We consider carbon fiber reinforced polymers (CFRPs) and glass fibers (GFRPs) due to its ideal properties compared to aramid fibers. FRPs in general produce small increase in weight and area which lead to a more economical project cost. Unlike steel jacket as confinement, it does not suffer from negative aspects like field welding and corrosion.

The CFRP are more effective on the confining of columns compared to the GFRP. There is about 50 % more of load carrying capacity when using CFRP than GFRP as shown in Figure 6.5 an interaction diagram shows the difference between E-GFRP and S-GFRP ,the S-GFRP have a higher strain-stress capacity that’s why it gives a more load caring capacity for the confined column (Figure 6.6)

6.4 Effect of concrete ultimate stress

For a fixed cross-section, columns with high concrete strength need more confinement pressure to get the same enhance their load carrying capacity (Figure 6.7).

Figure 6.8 shows the effect of the concrete strength on the behavior of the confined columns. Two columns wrapped with the same amount of the CFRP (2 mm) with cube concrete compressive strengths of 30 MPa ,45 MPa and 60 MPa are compared. The corresponding unconfined columns have axial strengths in pure compression of 10129 kN and11984 kN, and pure bending moment capacity of 575kN·m and 588 kN·m respectively. It is noted from Figure 6.8 that at a loads level higher than the axial strength of the corresponding unconfined column, an increase in the unconfined concrete strength has a negative effect on the normalized moment capacity, but at a load level below the axial strength of the corresponding unconfined column, the effect is inversed. On the other hand, the column with a higher unconfined concrete strength is seen to be less ductile (Figure 6.7). This is because the ductility of the confined column is closely related to the ultimate strain of confined concrete, and also the ultimate strain of the confined concrete is highly dependent on the confinement ratio. An increase in the unconfined concrete strength without increasing the amount of the FRP results in a reduced confinement ratio which negatively affects the ultimate strain of the confined concrete. This observation suggests that although an improvement in the axial strength of columns can be achieved by using higher strength concrete, there will be no corresponding improvement in the ductility (Figure 6.9).

6.5 Thickness of FRP effect

The results show that the confinement of columns with GFRP wrap increases the load-carrying capacity of reinforced concrete columns (Figure 6.10). The greater the number of GFRP layers, the greater the gain in axial load-carrying capacity with respect to unconfined columns. Figure 6.11 shows the interaction diagram of confined column versus the number of layers. It is evident that confinement with GFRP wrap improves the column ductility (Figure 6.12). This increased ductility allows for a higher level of axial strain failure corresponding to the rupture of GFRP wrapping (Figure 6.13). In most cases, failure is initiated at or near the corner, because of the high stress concentrations in these zones. The columns ductility increased as the number of layers of wrapping increase. Figures 6.14,15,16,17,18,19,20,21,22,23,24,25 shows the effect of the amount of the FRP on the predicted behavior of RC columns stress-strain relation with different compressive strengths and corner radii, where the reference column is compared with an unconfined column and a column wrapped with 2,4 and 6 mm of GFRP and CFRP, all having the same unconfined concrete strength f_{cu} 30MPa and 45 MPa. The unconfined column possesses Pu of 8274 kN and Mu of 558 kN·m. It can be seen from Figure 6.11 that an increase in the amount of FRP results in substantial increases in the strength and ductility of the columns. Compared to the unconfined column, an increase of 66% in axial strength is found for the column with the 4 mm GFRP jacket under pure compression, and this increase is nearly double (131%) for the 6mm GFRP jacket (Figure.6.12). Much higher increases in curvature and ductility are found from the FRP confinement (Figure6.12). In the case of pure bending (i.e. when Pu=0), the curvature ductility factor is only about 3 for the unconfined column, but considerably increases to approximately 20 and 40 for the columns confined by the 4 mm and 6 mm FRP respectively .Note that if the axial load is near zero, the FRP confinement results in little enhancement in the moment capacity. This indicates that in the case of pure bending, FRP jackets with fibers oriented only in the hoop direction will provide minimal enhancement to the moment capacity. If the moment capacity is to be enhanced, the recourse to FRP jackets with fibers orientated in the longitudinal direction of the column will have to be made.

6.6 Sharp edge effects on FRP confinement

The behavior of uniformly confined concrete circular sections has been extensively studied, leading to many models for both the compressive strength and the stress strain behavior. However, much less is known about the behavior of concrete in FRP-confined square columns, in which the confining pressure provided by the FRP varies over the cross-section and only part of the concrete is effectively confined (Park, Paulay 1975). As a result, the effectiveness of confinement is much reduced (Mirmiran *et al.* 1998) and rounding the right angle corners is generally recommended both to enhance confinement effectiveness and to reduce the detrimental effect of a sharp corner on the tensile rupture strength of FRP. A square column with rounded corners is shown in Figures 6.16,17 (square columns are considered as a special case of rectangular columns with “width = depth”). To improve the effectiveness of FRP confinement, corner rounding is generally recommended. Due to the presence of internal steel reinforcement, the corner radius is generally limited to small values. The efficiency of the CFRP confinement is higher for circular than for square sections, as expected. The radius of the corners of square sections is of great importance in relation to the level of confinement: The increase of ultimate strength of sharp edged sections is low, although there is a certain gain of load capacity and of ductility; Corner radii of 0 mm and 20 mm show a moderate gain of load capacity, with effectiveness increasing with rounding radius. The increase of axial deformation is very high for all the considered geometries. Figure 6.26 shows the effect of the corner radius on a rectangular column confined with GRFP the load carrying capacity is enhanced due to the corner radius the load capacity increase as the corner radius increases this is due to the increase in the stress-strain relation as shown in Figure 6.27.

6.7 FRP strain capacity

Columns confined with CFRP and GFRP are compared, both have the same unconfined column and a glass FRP (GFRP) confined column with the same unconfined concrete strength. The elastic modulus and hoop rupture strength of the GFRP are 200000 MPa and 70000 MPa respectively, and the total thickness of the GFRP wrap is 4 mm. Such a GFRP wrap supplies the same maximum confining pressure to the column as the 4 CFRP wrap, but has a higher strain capacity. The strain of the GFRP at hoop rupture is 0.04, while that of the CFRP is 0.02. Figures 6.14,15,16,17 the ultimate axial strain of the FRP-confined concrete column wrapped with the GFRP is 18.2% higher than that of the reference column wrapped with CFRP. While this increase in the ultimate strain of the confined concrete has no effect on the axial strength and moment capacity of the columns (Figure 6.9), it results in considerable increases in the curvature and ductility factor (Figure6.12) At an axial load of 5000 kN, the ultimate curvature and curvature ductility factor of the column are 19% and 18% higher respectively for the column confined by the GFRP than those for the one confined by the CFRP. This observation confirms that the ductility of the column is closely related to the strain capacity of the confining FRP.

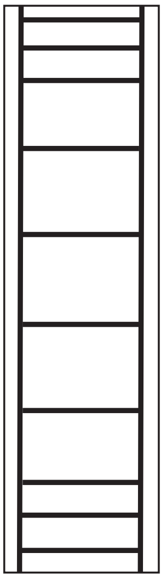
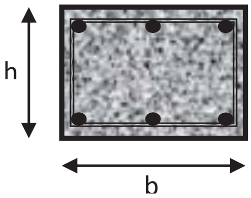
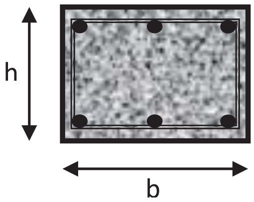
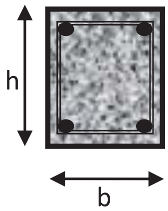
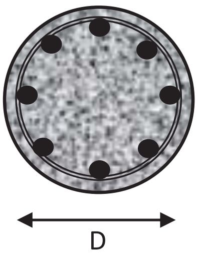
Elevation	X-sec Shape	b (mm)	h (mm)	d (mm)	f_{cu} (MPa)			E_s (GPa)	E_j (GPa)		ε_j	
									GF	CF	GF	CF
		700	350	-	30	45	60	200	70	230	0.04	0.02
		750	500	-	30	45	60	200	70	230	0.04	0.02
		500	500	-	30	45	60	200	70	230	0.04	0.02
		-	-	500	30	45	60	200	70	230	0.04	0.02

Table 6.1 Geometry of column sections

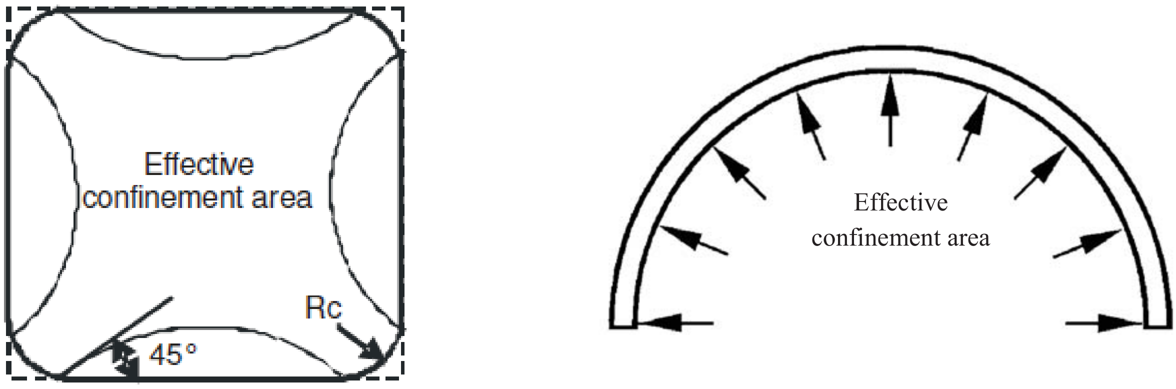


Figure 6.1 Effective confined concrete section

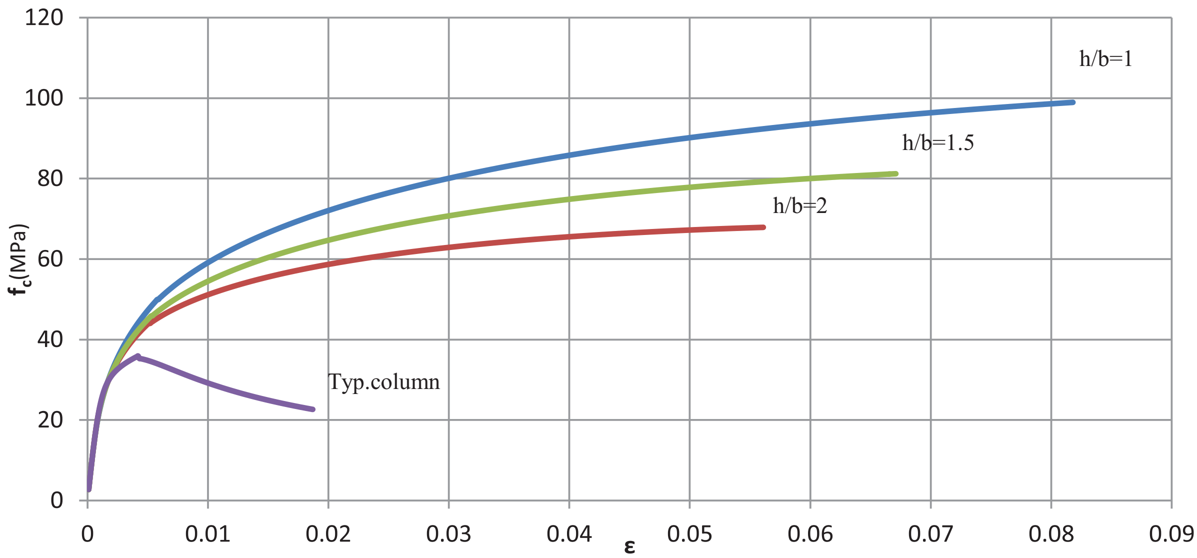


Figure 6.2 The effect of aspect ratio on confinement

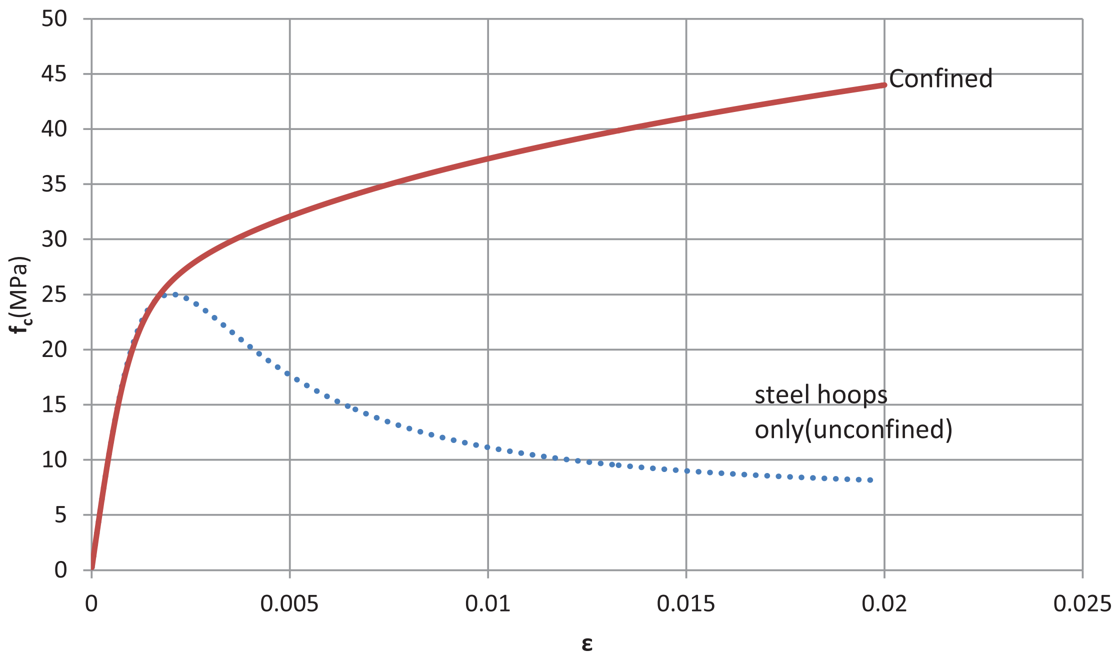


Figure 6.3 The effect of FRP confinement on a rectangular section

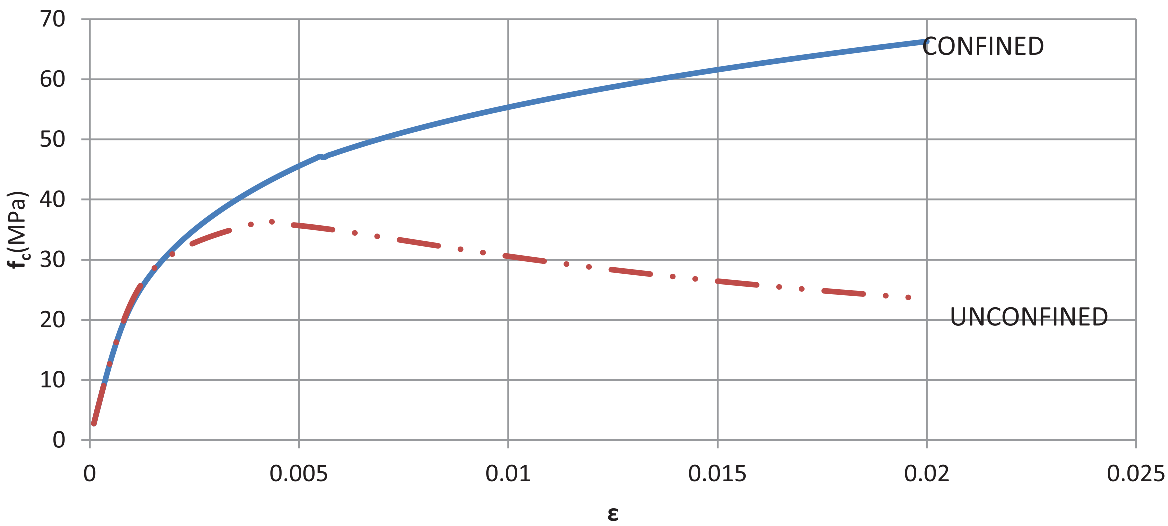


Figure 6.4 Circular section confined stress-strain diagram

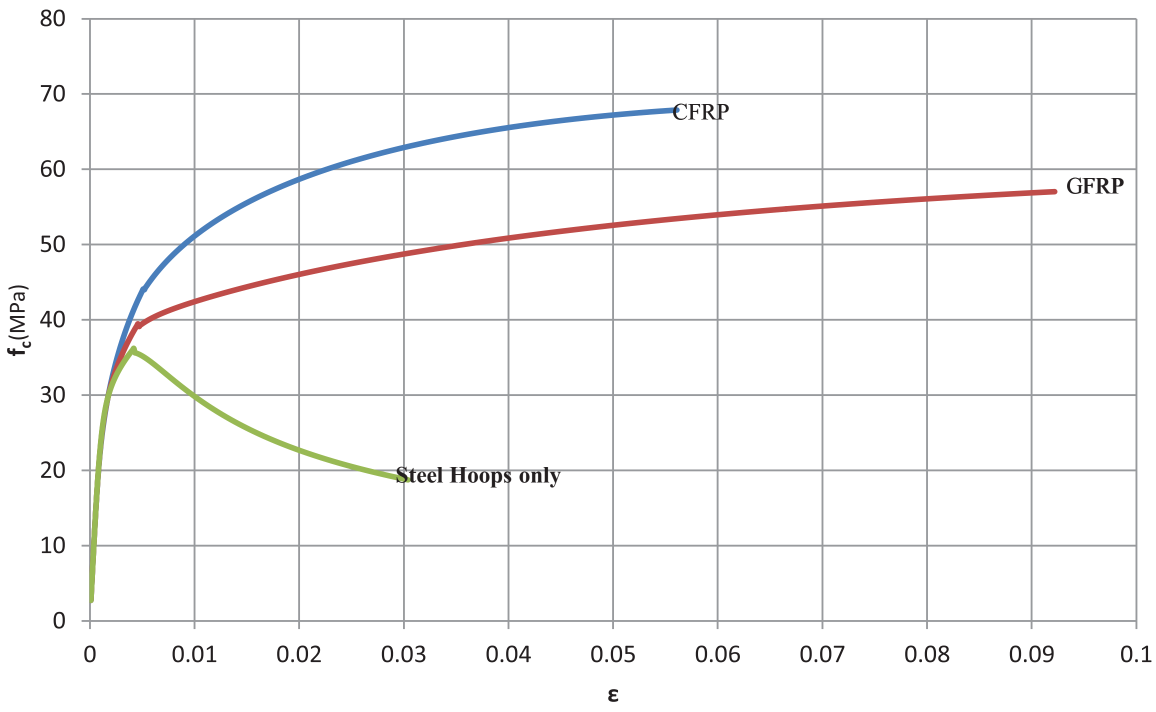


Figure 6.5 Different between GFRP and CFRP for rectangular section

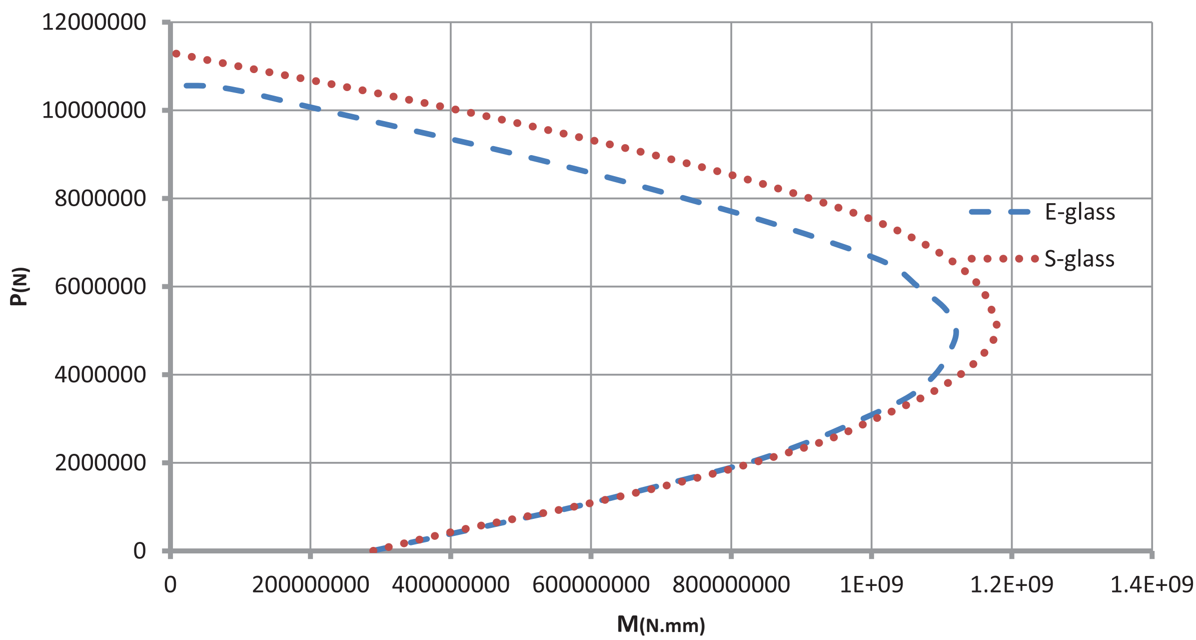


Figure 6.6 Difference between Glass fibers

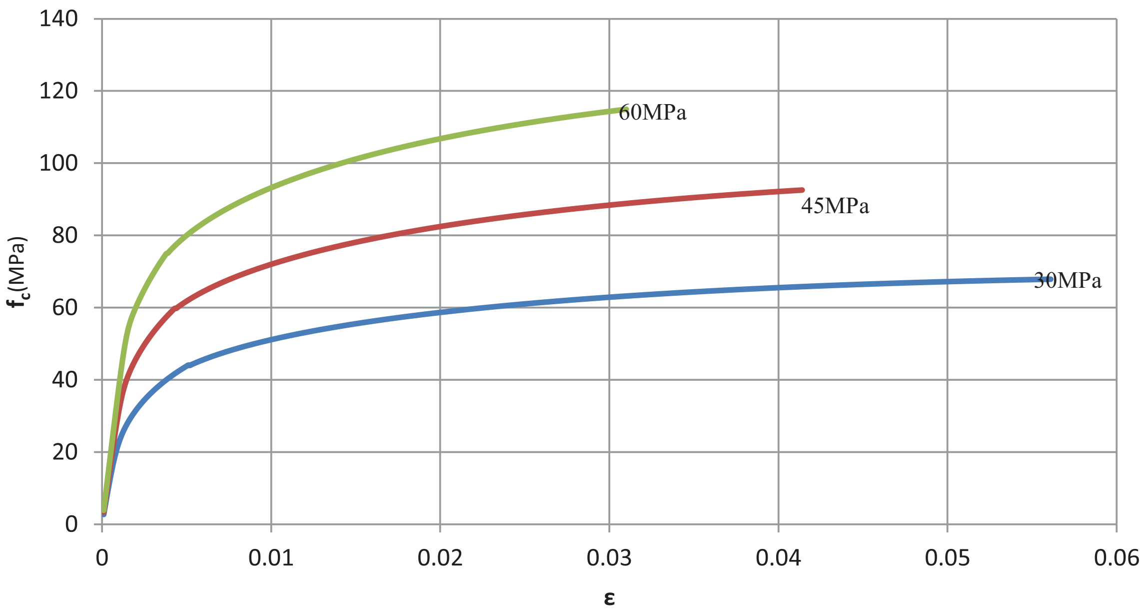


Figure 6.7 Effect of CFRP Confinement for rectangular section with different Concrete Strength

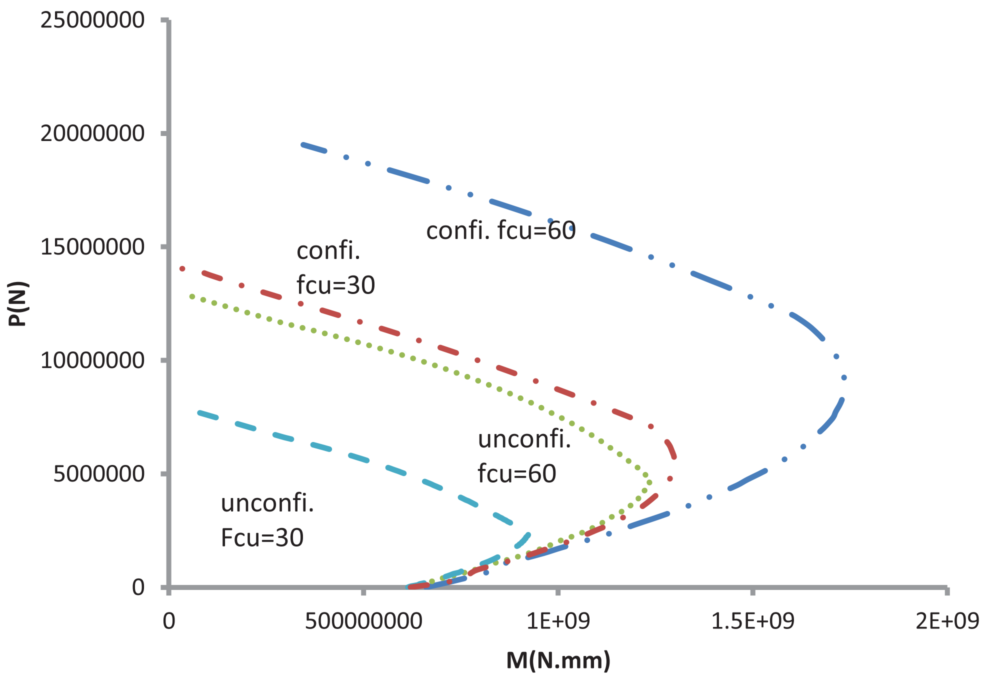


Figure 6.8 Interaction diagram of variable f_{cu}

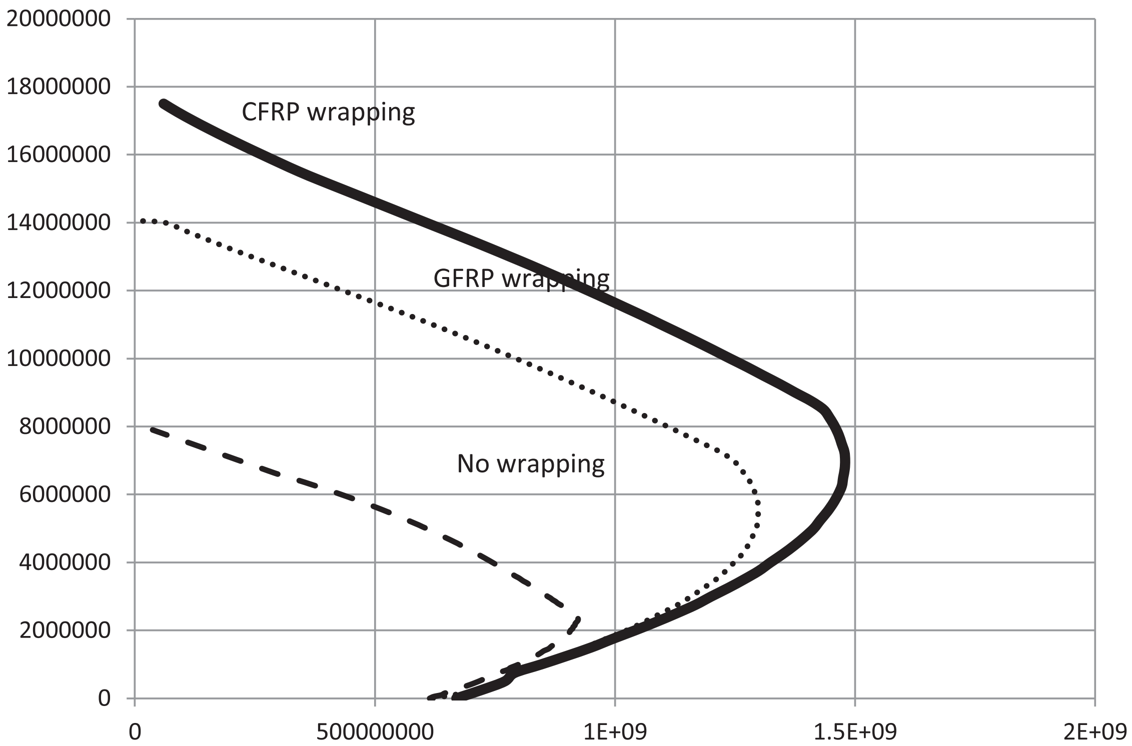


Figure 6.9 Different FRP interaction diagram

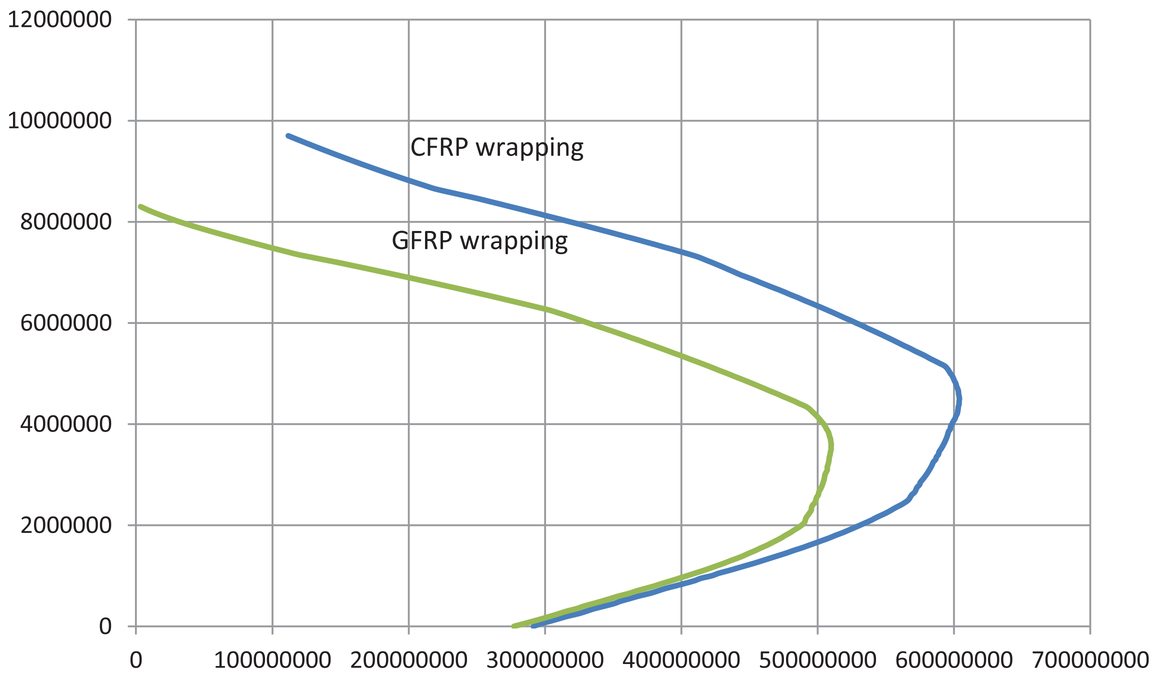


Figure 6.10 Circular interaction

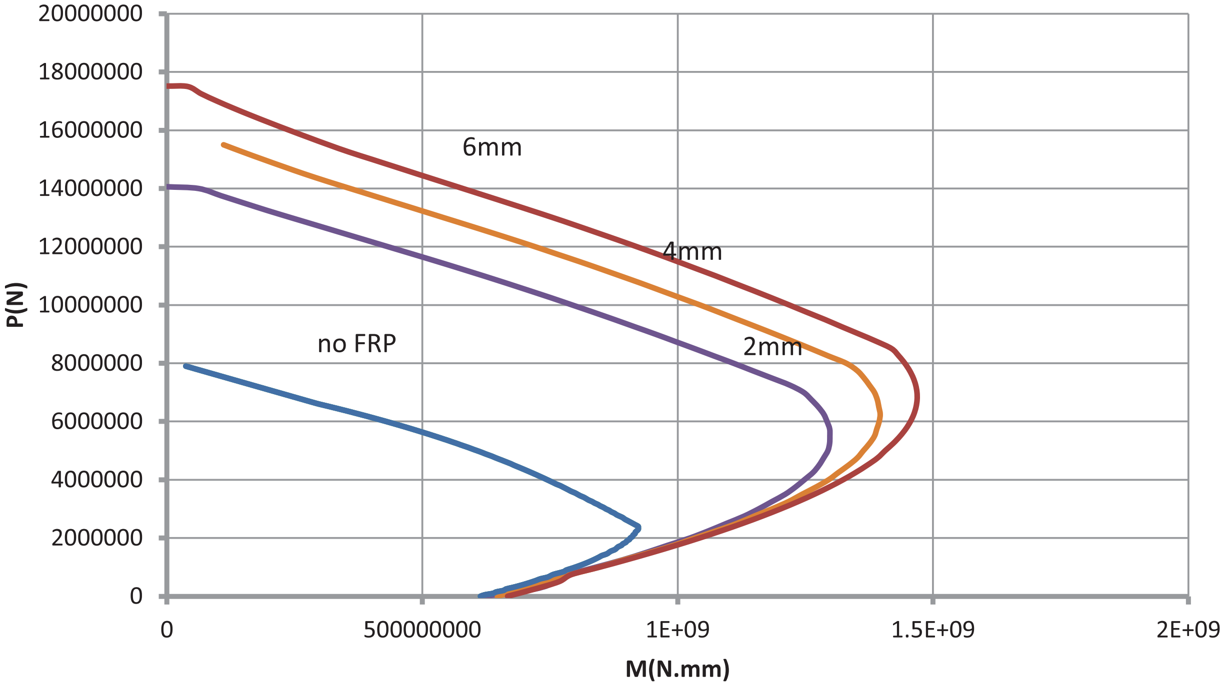


Figure 6.11 Confinement of rectangular column with different thickness

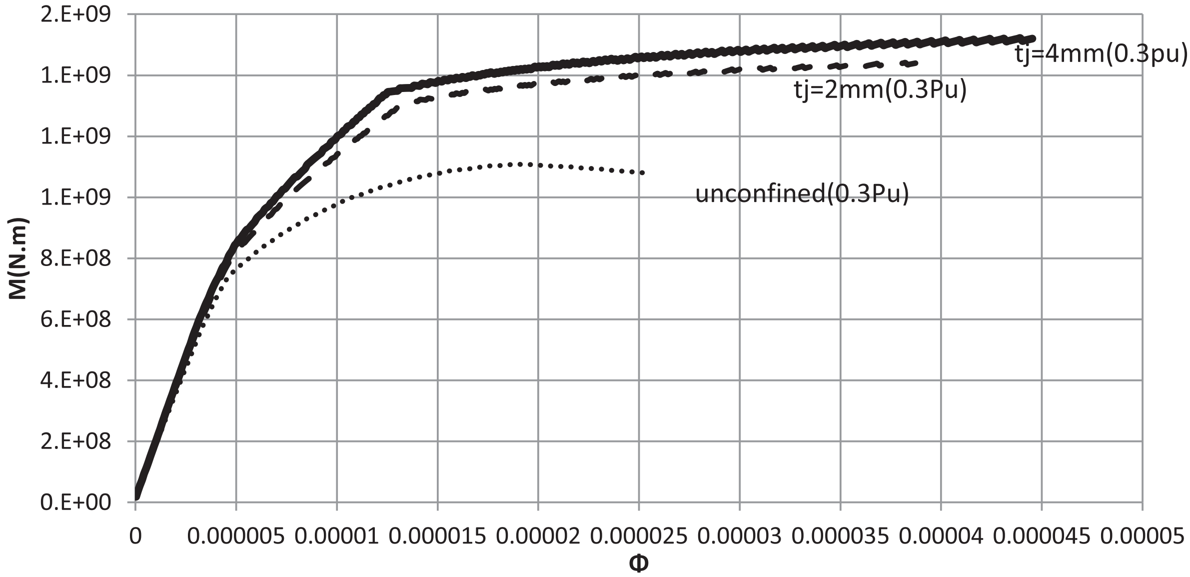


Figure 6.12 Moment curvature for confinment of squae column

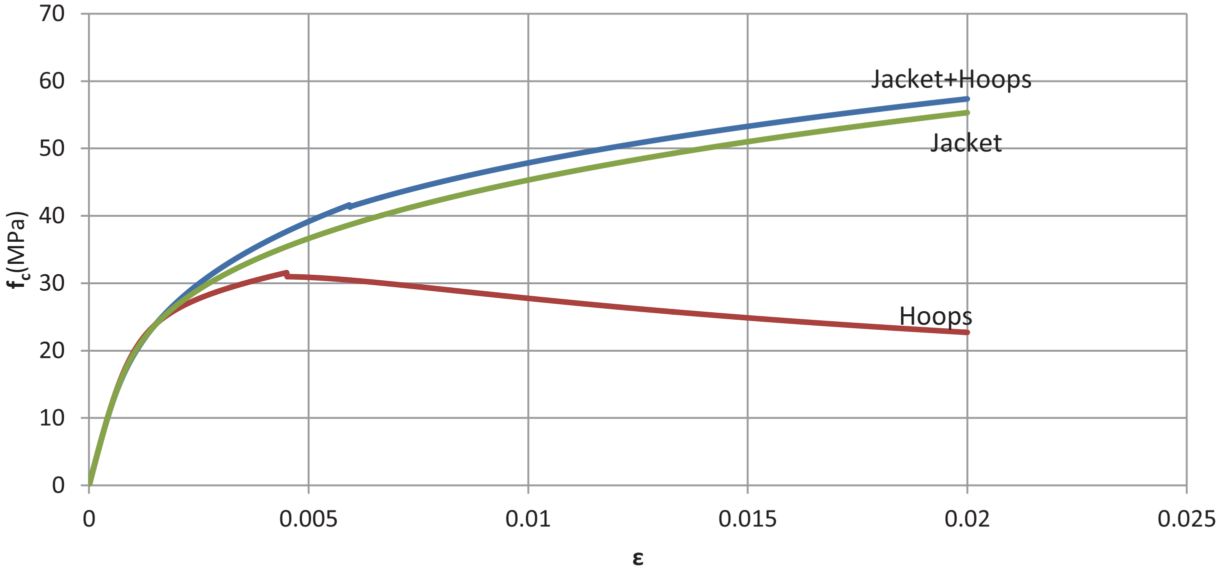


Figure 6.13 Confinment of square column different between jackets and no jackets

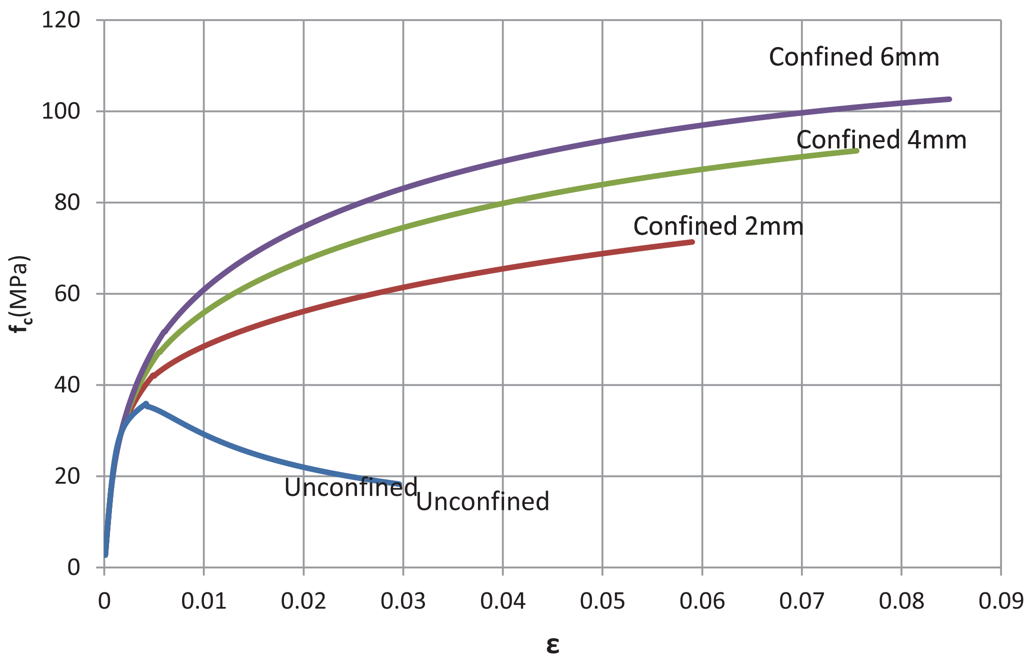


Figure 6.14 CFRP confinment of square section (f_{cu}=30MPa corner radius=0)

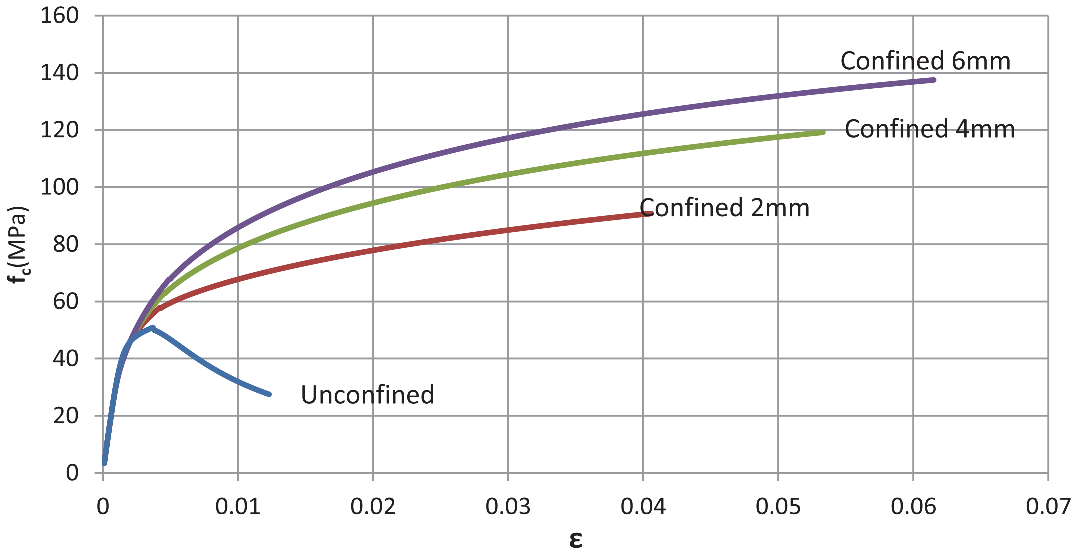


Figure 6.15 CFRP confinement of square section (f_{cu}=45MPa corner radius=0)

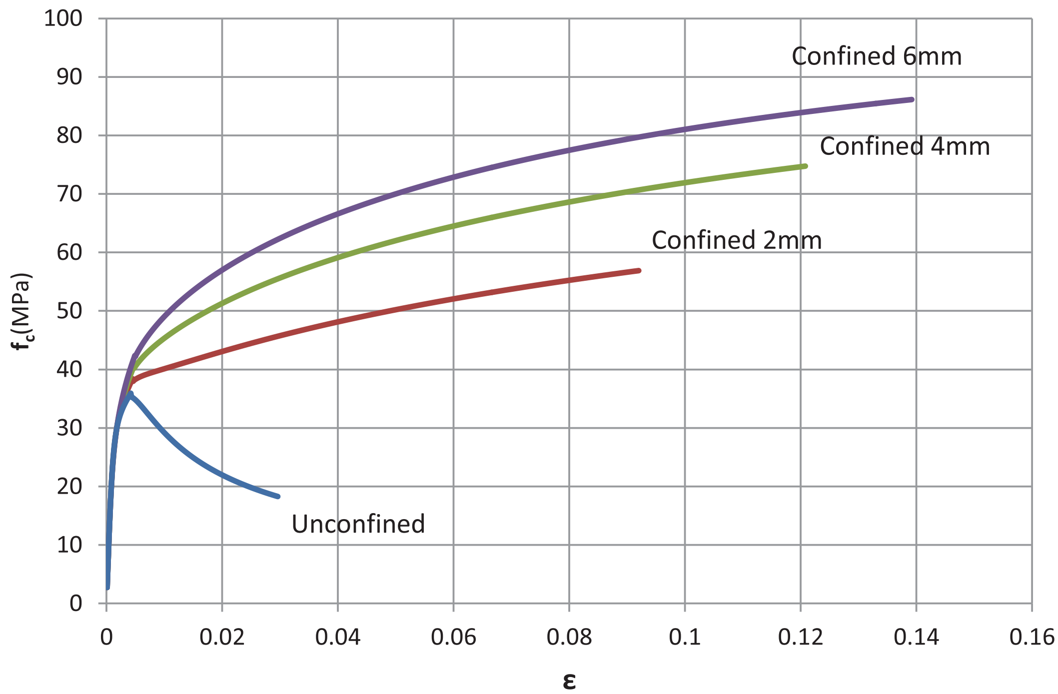


Figure 6.16 GFRP confinement of square section(f_{cu}=30MPa corner radius=10)

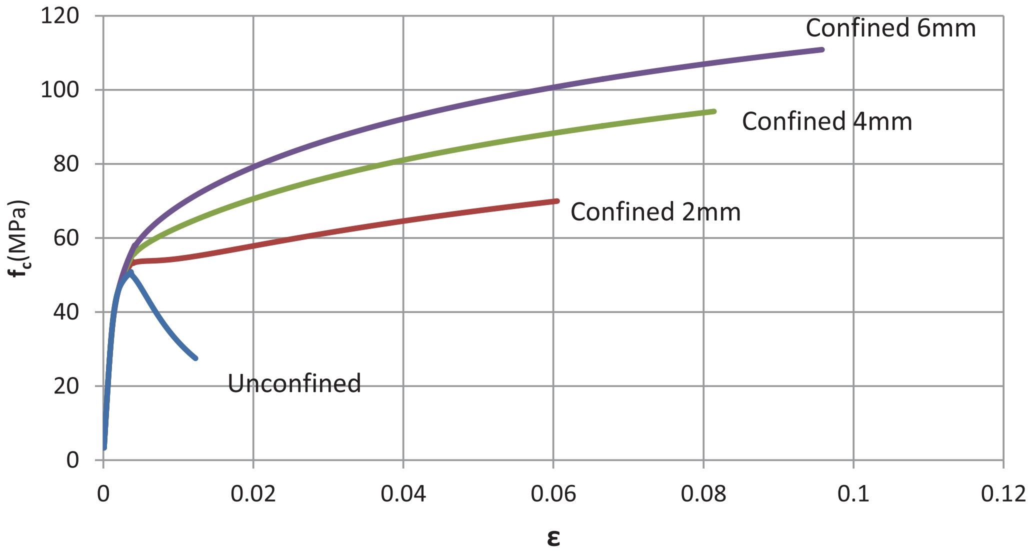


Figure 6.17 GFRP confinment of square section(f_{cu}=45MPa corner radius=10)

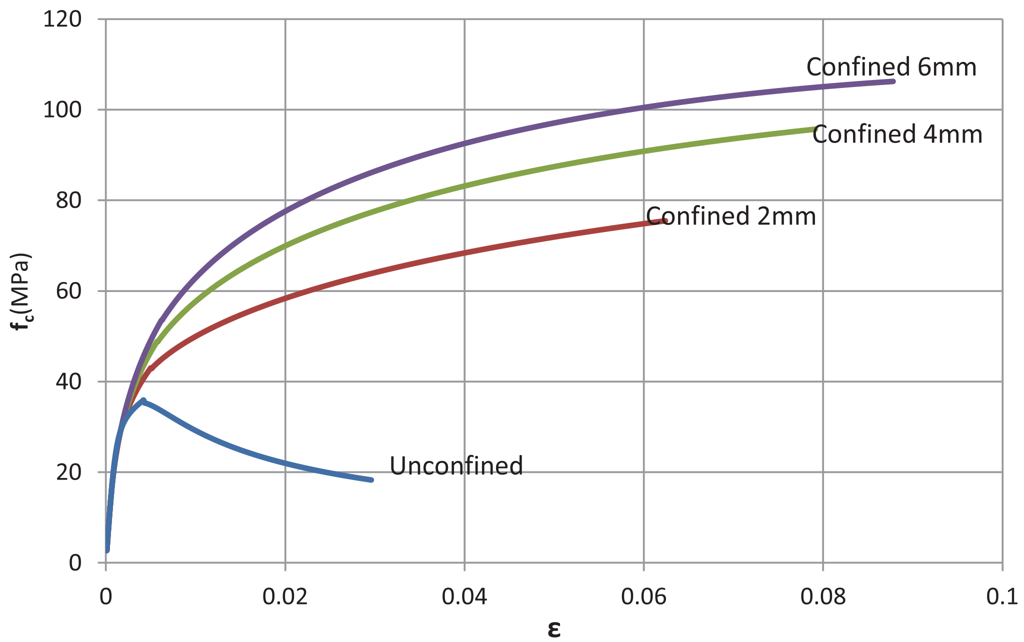


Figure 6.18 GFRP confinement of rectangular section (f_{cu}=30MPa corner radius=0)

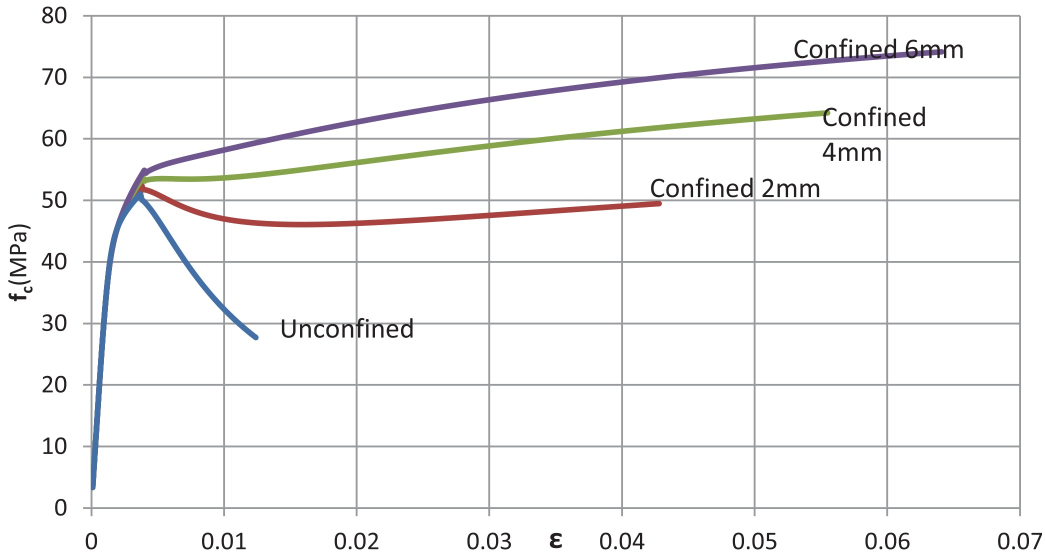


Figure 6.19 GFRP confinement of rectangular section (f_{cu}=45MPa corner radius=0)

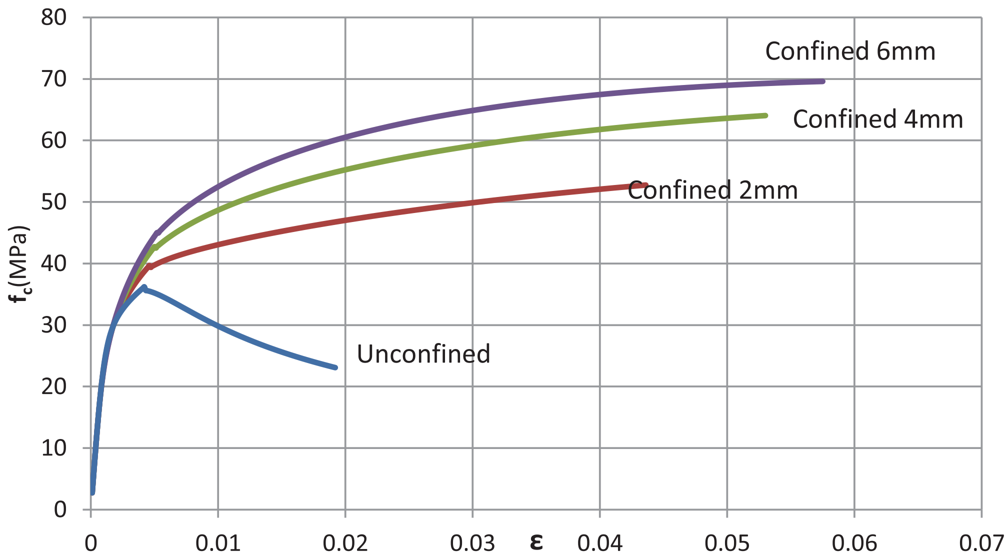


Figure 6.20 CFRP confinement of rectangular section (f_{cu}=30MPa corner radius=0)

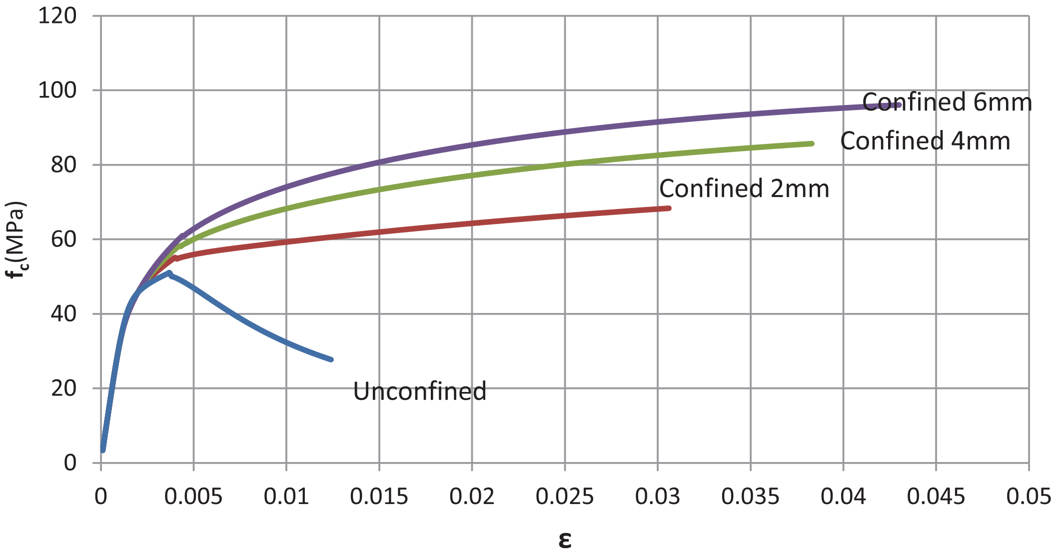


Figure 6.21 CFRP confinement of rectangular section (f_{cu}=45MPa corner radius=0)

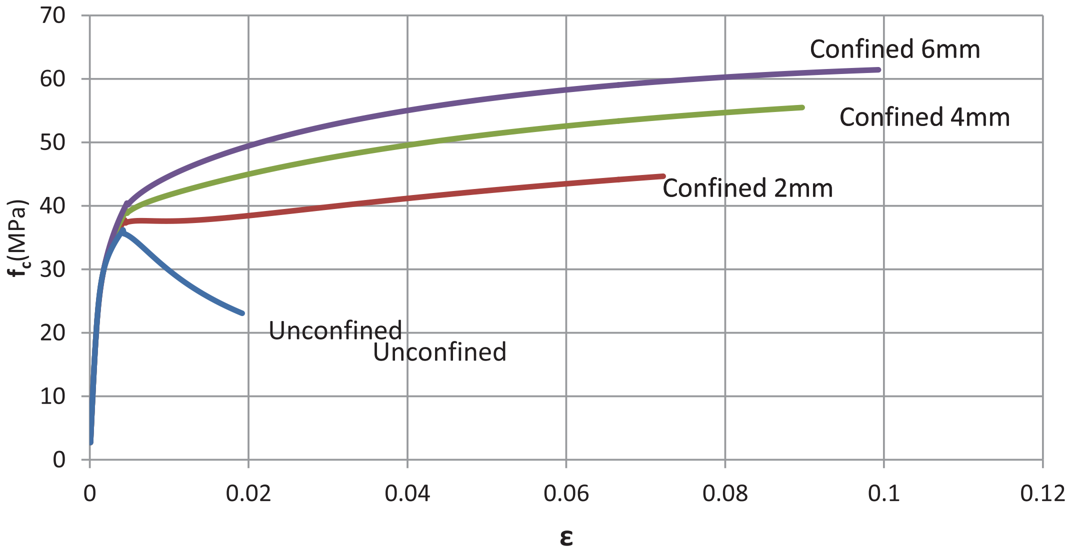


Figure 6.22 GFRP confinement of rectangular section (f_{cu}=30MPa corner radius=10)

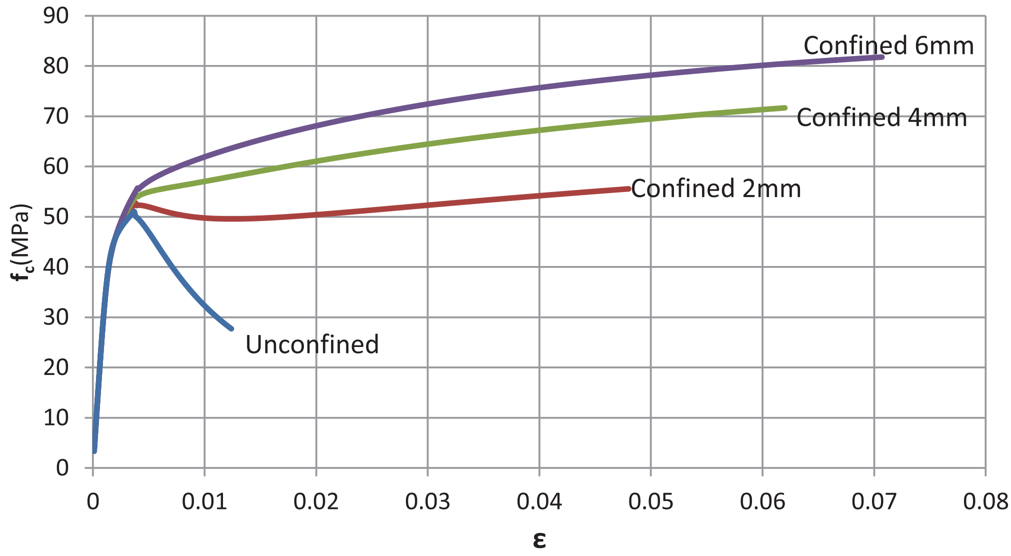


Figure 6.23 GFRP confinement of rectangular section (f_{cu}=45MPa corner radius=10)

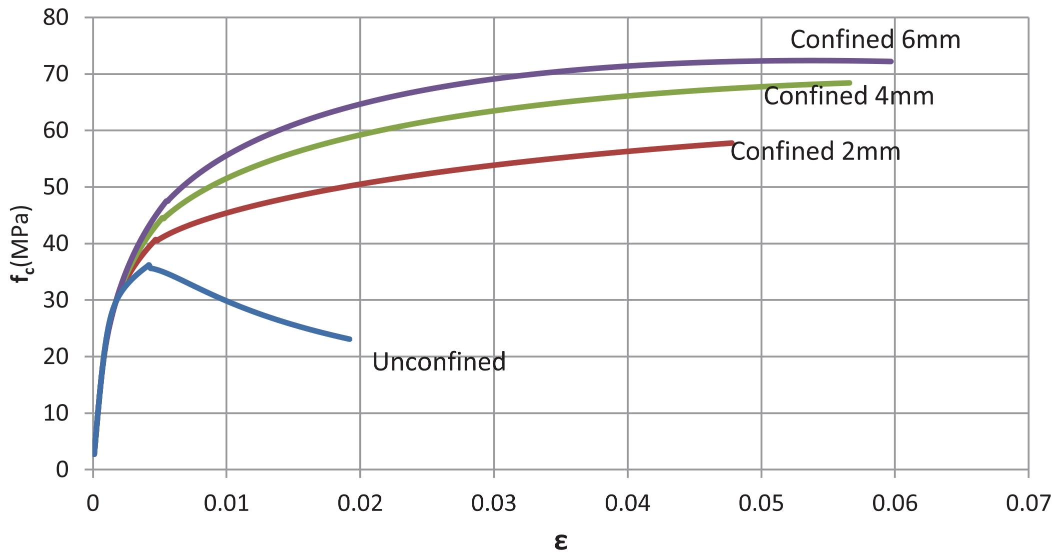


Figure 6.24 CFRP confinement of rectangular section (f_{cu}=30MPa corner radius=10)

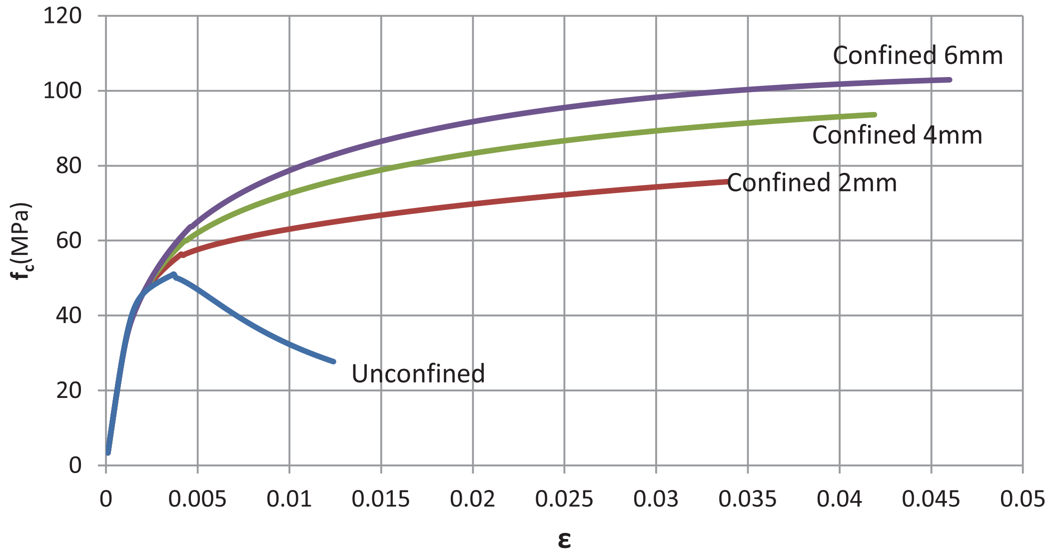


Figure 6.25 CFRP confinement of rectangular section (f_{cu}=45MPa corner radius=10)

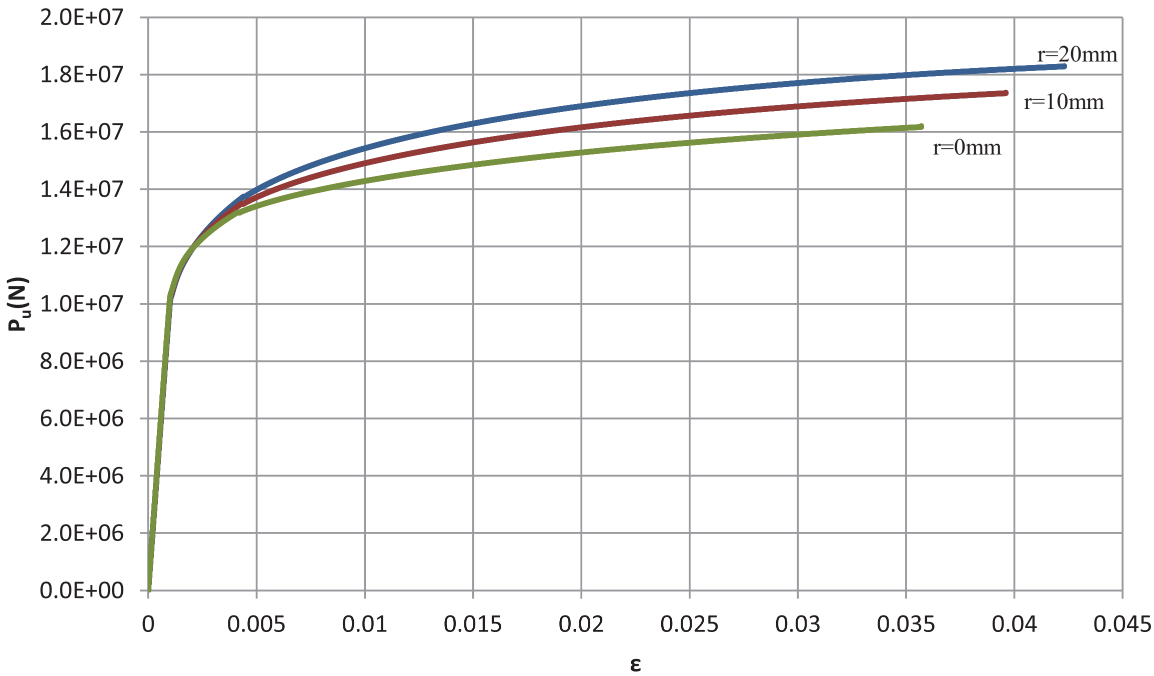


Figure 6.26 Load strain behavior of confined rectangular column with CFRP jacket

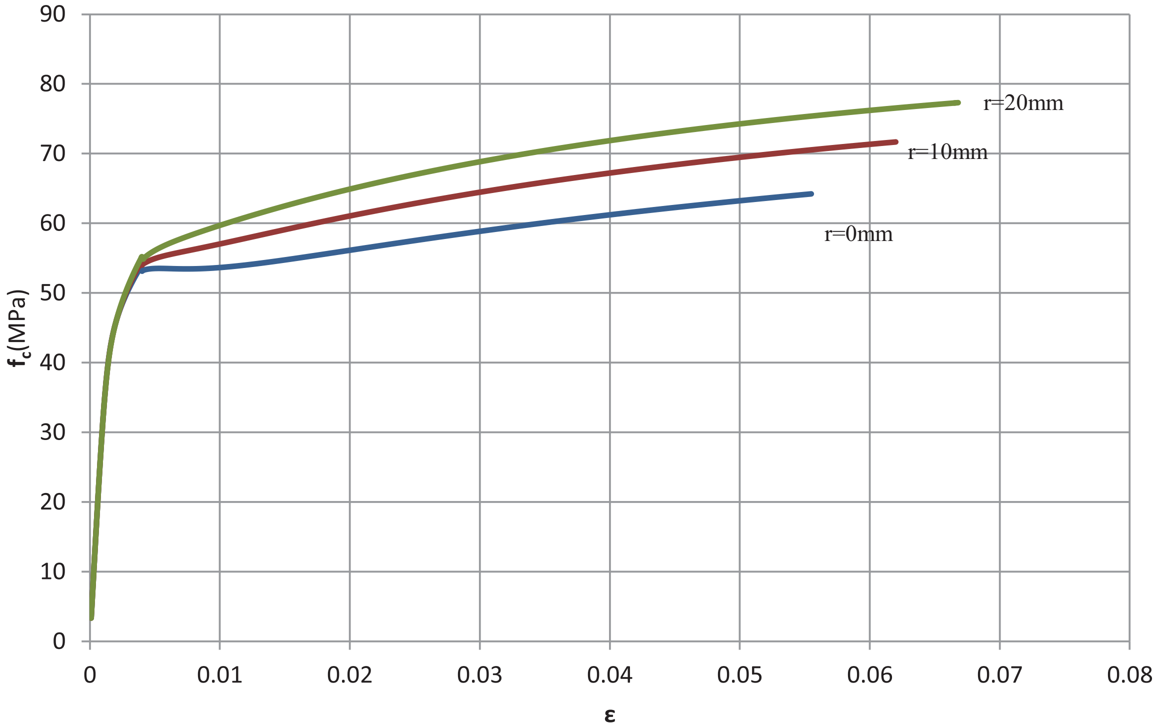


Figure 6. 27 Stress strain behavior of confined rectangular column with GFRP jacket

CHAPTER 7

CONCLUSIONS

7.1 Summary

This research presented the effect of fiber-reinforced polymer jacket retrofitting reinforced concrete columns. It discusses the enhancement of concrete ductility and strength. Stress-strain curves, moment curvature and load moment curves are plotted to show the effect of the FRP jacket.

The wrapping of composite sheets around concrete columns is a promising method for structural strengthening and repair. This rehabilitation technique is of practical interest, as the lay-up of the sheets is rather easy; it does not require specialized tools, and the epoxy resins employed cure at ambient temperatures. The columns were confined by means of carbon-epoxy or glass fiber sheets and loaded concentrically in axial compression. The effects of various parameters on the structural behavior of the confined concrete columns are investigated. These parameters included the concrete strength, thickness of FRP layer, corner radius and FRP material used. The results showed that composite confinement considerably enhance the structural performance of concrete columns, especially with regard to ductility. The potential to restore the full strength of severely damaged columns or enhance the performance of column, as retrofitted columns exhibit axial load carrying capacities equal or superior to those of undamaged columns, along with significant increases in ductility. For such cases tests on plain concrete cylinders are sufficient for further investigations of this retrofit method, as the key parameters which really affect strength and ductility are the concrete strength, composite fiber type and sheet thickness.

Step-by-step design procedures are proposed for the axial load capacity enhancement of circular and rectangular reinforced concrete columns confined with fiber reinforced polymer wraps. The design methods are intended for practicing engineers in that they are relatively simple to apply and are made readily available in a flow chart format. Comments are provided to explain the design philosophy and rationale leading to the various design equations.

7.2Conclusions

Based on the parametric study, the following conclusions are drawn:

Confinement by FRP jackets enhances the performance of rectangular concrete columns. CFRP wrapping is more effective for square sections than for rectangular sections. As the aspect ratio increase from one to two.

Carbon fiber reinforced polymers (CFRPs) are recommended for retrofitting due to thier ideal properties compared to glass fibers and aramid fibers. From studies, glass fibers are weak in aging and should be protected from chemical attack. CFRPs have been proven to be more efficient than aramid and glass fibers when applied to concrete columns as external reinforcement.

The axial strength and curvature ductility of RC column can be considerably enhanced by using the FRP confinement, and a higher amount of FRP produces a higher degree of enhancement. In the case of pure bending, FRP jackets with fibers oriented only in the hoop direction, a has little effect on the moment capacity of the columns results. In this case, the use of longitudinal FRP has to be considered in order to increase the bending moment capacity.

The ultimate axial strain of confined concrete can be higher in a column wrapped with GFRP than in a column wrapped with CFRP; this is because the GFRP has a higher strain capacity than the CFRP. While this increased ultimate strain has no effect on the axial strength and moment capacity of the confined column, it yields a considerable improvement in the curvature ductility of the column.

An increase in the unconfined concrete strength has different effects on the moment capacity of confined column for load levels above and below the axial strength of column. An increased unconfined concrete strength reduces the curvature ductility of the column because the ultimate axial strain of confined concrete is reduced without increasing the amount of FRP.

The FRP confinement is much less effective for rectangular columns but an increase in the corner radius is beneficial to both strength and ductility. An increase in the aspect ratio has a negative effect on the axial strength of the FRP-confined column, but may have small beneficial effects on the moment capacity and ductility.

Uni-axial compression results of RC columns confined with CFRP jackets have shown that the increase of ultimate strength is influenced and increases with the radius of the corners of square sections. On the other hand, the increase of axial deformation capacity is up to 8 times that of unconfined concrete, even for the sharp edged sections.

The number of layers of FRP materials is major parameters, having a significant influence on the behavior of specimens. The test results from preliminary testing proved that the benefit of confinement could be enhanced by increasing the stiffness of external confinement by applying multiple layers and a good corner radius for square shape.

The corner radius in square columns affects the behavior. It determines stress concentration at corner zone. A larger radius can expand the strong constraint zone and diminish the stress concentration. So the reduced confining pressure in a square section due to the concentration of stresses at the corners is solved by using a square section with circular corners

CFRP materials produce a better lateral confinement pressure to column specimens than the other FRPs it can be used for strengthening.

The higher the concrete strength more FRP confining pressure is needed to make a significant effect on the column confining pressure. Both compressive strength and ductility enhancement are more pronounced in low strength concrete rather than high strength concrete.

The ultimate strain of the confined concrete increases. Consequently, the tensile reinforcement steels undergo strain hardening range resulting in increment of the bending moment carrying capacity and the ductility. Increment of the ductility provides higher reliability of the confined column.

Although glass fiber has a larger elongation at failure than carbon fiber, carbon fiber has a larger energy-absorbing capacity, indicated by its larger area under the stress-strain curve. Based on an energy balance approach, this results in an increase in ultimate axial load and ductility for strengthening with carbon fiber than for strengthening with glass, if the volumes of straps are equal.

The rate of increase in the ultimate axial load, ductility and maximum moment capacity decreases for increasing concrete compressive strength.

7.3 Recommendations for Further Research

Based on the study presents in this thesis it is recommended for further studies to:

Make a lab work to identify the difference between confinement models and result from the lab work and to modify and predict a new model that's suits each shape of the column with different aspect ratios.

Make a finite element model to predict the stress concentration at corners and the bond between the FRP and the concrete column and their effect on the stress-strain relation of confined concrete column.

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ال ّمخص:

حص ًى ا. عَلقخشسينُت انّذع ّتبب نبُف ىلعن ُبّست أونفو لارحسخذ فٍ رنك ال ٍ
الحصش الجنب ٍنمعيىد و معضخصش ملكبُشرقىحجبف ٍراضغطال َحس ً انّذعى. فينِاسرنىاث
الأخُشقبشص بلرخخذاوسخشاث ين الأبُف ىلعنُبسَتنصحشالخجس ٍنخشينت وقذأخش َ ج
دساسيث عذّذة ييخشانيضع نبّرج سيضَ ُتدقُقت للاجهبد والافعلن حسبةنالقىةناشييت
للأع ّذة. خقىرن نبّرج الَسخخيت انّفخعُن، نبّرج حص ّ ُت و أحشي حجُهُت. جخأش حهك
رانبّر جس ًك الأبُف وقىة ضغط خشسينتونسيت الإسخطيت ونص فطقشصاوتقطبع. سىف
نذسط فٍ هزةالشسيت يذي جتُش هزة العايم ع ٍه احخلالنعقطباع خشسب ِن ولفتُُتحمحن ُ و
حقىاتقطبع لإسخ ُعبة الاحبّ لنازئذة.

الأكاديمية العربية للعلوم والتكنولوجيا والنقل البحرى

كلية الهندسة والتكنولوجيا

قسم هندسة التشييد والبناء

إعادة تأهيل الأعمدة الخرسانية باستخدام سترات مركبة

اعداد

حسام شريف الألفي

رسالة مقدمة للأكاديمية العربية للعلوم والتكنولوجيا والنقل البحرى لاستكمال متطلبات نيل درجة

الماجستير

فى

الهندسة الإنشائية

تحت إشراف

ا.د. مشهور غنيم

الهندسة الإنشائية

جامعة القاهرة

ا.م.د. عبد المنعم سند

الهندسة المدنية

الاكاديمية العربية للعلوم والتكنولوجيا والنقل البحرى

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