REHABILITATION OF STRUCTURES USING COMPOSITE MATERIALS

By

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COMMITTEE DECISION

This Thesis/Dissertation (***Rehabilitation of Structures Using Composite Materials***) was Successfully Defended and Approved on 6th Dec 2009

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To

My Family

Sana' Rihan

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NOTATION

$A_f = nt_f w_f$, area of FRP external reinforcement, (mm^2)

A_{fv} = area of FRP shear reinforcement with spacing s, (mm^2)

A_g = gross area of section, (mm2)

A_s = area of nonprestressed steel reinforcement, (mm^2)

A_{st} = total area of longitudinal reinforcement, (mm^2)

b = width of rectangular cross section, (mm)

b_w = web width or diameter of circular section, (mm)

c = distance from extreme compression fiber to the neutral axis, (mm)

C_E = environmental-reduction factor

d = distance from extreme compression fiber to the neutral axis, (mm)

d_f = depth of FRP shear reinforcement, (mm)

E_c = modulus of elasticity of concrete, (MPa)

E_f = tensile modulus of elasticity of FRP, (MPa)

E_s = modulus of elasticity of steel, (MPa)

f_c = compressive stress in concrete,(MPa)

f'_c = specified compressive strength of concrete, (MPa)

f'_{cc} = apparent compressive strength of confined concrete, (MPa)

f_f = stress level in the FRP reinforcement, (MPa)

$f_{f,s}$ = stress level in the FRP caused by a moment within the elastic range of the member, (MPa)

f_{fe} = effective stress in the FRP; stress level attained at section failure,(MPa)

f^*_{fu} = ultimate tensile strength of the FRP material as reported by the manufacturer, (MPa)

f_{fu} = design ultimate tensile strength of FRP, (MPa)

f_l = confining pressure due to FRP jacket,(MPa)

f_s = stress in nonprestressed steel reinforcement,(MPa)

$f_{s,s}$ = stress level in nonprestressed steel reinforcement at service loads,(MPa)

f_y = specified yield strength of nonprestressed steel reinforcement,(MPa)

h = overall thickness of a member, (mm)

k = ratio of the depth of the neutral axis to the reinforcement depth measured on the same side of neutral axis

k_1 = modification factor applied to κ_v to account for the concrete strength

k_2 = modification factor applied to κ_v to account for the wrapping scheme

L_e = active bond length of FRP laminate, (mm)

M_n = nominal moment strength, (N-mm)

M_u = factored moment at section, (N-mm)

n = number of plies of FRP reinforcement

P_n = nominal axial load strength at given eccentricity, (N)

r = radius of the edges of a square or rectangular section confined with FRP, (mm)

s_f = spacing FRP shear reinforcing, (mm)

t_f = nominal thickness of one ply of the FRP reinforcement, (mm)

V_c = nominal shear strength provided by concrete with steel flexural reinforcement, (N)

V_n = nominal shear strength, (N)

V_s = nominal shear strength provided by steel stirrups, (N)

V_f = nominal shear strength provided by FRP stirrups, (N)

w_f = width of the FRP reinforcing plies, (mm)

α = angle of inclination of stirrups or spirals, degrees

ε_{bi} = strain level in the concrete substrate at the time of the FRP installation (tension is positive), (mm/mm)

ε_c = stain level in the concrete, (mm/mm)

ε_{cu} = maximum usable compressive strain of concrete, (mm/mm)

ε_f = strain level in the FRP reinforcement, (mm/ mm)

ε_{fe} = effective strain level in FRP reinforcement; strain level attained at section failure, (mm/mm)

ε_{fu} = design rupture strain of FRP reinforcement, (mm/mm)

ε^*_{fu} = ultimate rupture strain of the FRP reinforcement, (mm/mm)

ε_s = strain level in the nonprestessed steel reinforcement, (mm)

ε_{sy} = strain corresponding to the yield strength of nonprestressed steel reinforcement, (mm)

Φ = strength reduction factor

γ = multiplier on f'_c to determine the intensity of an equivalent rectangular stress distribution for concrete

κ_a = efficiency factor for FRP reinforcement (based on the section geometry)

κ_m = bond-dependent coefficient for flexure

κ_v = bond-dependent coefficient for shear

ρ_f = FRP reinforcement ratio

ρ_g = ratio of the area of longitudinal steel reinforcement to the cross-sectional area of a compression member

ψ_f = additional FRP strength-reduction factor

REHABILITATION OF STRUCTURES USING COMPOSITE MATERIALS

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ABSTRACT

The volume of existing concrete structures that need upgrading, strengthening and/or repair to resist higher design loads, or increase ductility is growing worldwide. Fiber reinforced polymer (FRP) systems for strengthening concrete structures have emerged as an alternative to traditional strengthening techniques, such as steel plate bonding, and section enlargement.

This research provides the following subjects:

- Identify the different methods of rehabilitation and the properties of materials used.
- Give an overview of the engineering properties of FRP as a strengthening material for reinforced concrete (RC) structures.
- Give guide and recommendations for the design of externally bonded FRP systems for strengthening concrete structures.

- Explain different case studies of strengthening RC structures using FRP.
- Compare between theoretical and experimental values for different configurations of FRP shear strengthening of RC beams.
- Produce an Excel sheet for shear beam strengthening design, which is base on ACI 440 equations.

Chapter 1

Introduction

1.1: General

Deterioration or damage of concrete structures is one of the major problems of the construction today.

Concrete structures deteriorate, or become unsafe due to different reasons like:

- Internal reinforcement corrosion.
- Changes in loading, changes in use, or changes in configuration.
- Poor initial design.
- Construction practices that ignore the environmental impact.
- Fire, blast loading, and earthquake.

A large number of structures constructed in the past using the older design codes in different parts of the world are structurally unsafe according to design codes today, they were solely designed for gravity loads, such buildings are not able to withstand seismic forces that would cause wide spread damages.

Older structures must be upgraded so that they meet the same requirements demanded for structures built for today and the future.

Since the number of buildings, and components of structures that have deteriorated in service and are in need for repair and maintenance is large and ever increasing, it is becoming both environmentally and economically usable to upgrade structures rather than to rebuild them, particularly if rapid, effective and simple strengthening and repair methods are available.

Since replacement of such deficient structures incurs a huge amount of money and time, strengthening and repair have become the acceptable way of improving their load carrying capacity and extending their service lives.

The choice between upgrading and rebuilding is based on factors specific to each individual case, but certain issues are considered in every case.

These are the length of time during which the structures will be out of service or providing a reduced service, relative costs upgrading and rebuilding in terms of labor, materials and plant, and disruption of other facilities.

We can summarize the rehabilitation method as an operation to bring a structure or a structural component that is deficient in design demand to the desired specific performance level.

Depending upon the state of the structure and the desired post intervention performance level, rehabilitation can be divided into two categories: repair and strengthening.

Repair is the rehabilitation of a damaged structure or a structural component with the aim of restoring the original capacity of the damaged structure.

Strengthening is the process of increasing the existing capacity of a non-damaged structure or a structural component to a higher level.

1.2: Objectives

This thesis deals generally with the subject of **rehabilitation of structures using composite materials**, specifically strengthening of main structural elements.

1.3: Literature review

Many researches, studies, and experiments in repair and strengthening of concrete structures have been done in different countries of the world.

Meier reported the use of thin carbon fiber reinforced polymer (CFRP) plates as reinforcement for flexural strengthening of concrete beams. He reported that CFRP could replace steel with overall cost savings in the order of 25%. Kaiser tested CFRP composites applied to the tension side of full-scale reinforced concrete beams and the strain compatibility over the cross-section was verified. It was suggested that inclined cracking might lead to premature failure by peeling-off of the strengthening plate.

Sato et al. studied the shear reinforcing effect of CFRP plates attached to the sides of RC beams with and without stirrups. Experimental parameters were the location and amount of CFRP and amount of stirrups. The results indicated that CFRP significantly increased the shear strength of most of the tested specimens; especially CFRP plates attached to three faces of beam (side-bottom-side) was effective. The failure mode with delamination of CFRP along a shear crack was observed in specimens without stirrup.

Experimental by Harajli et al. (2005, 2006), in which twenty-four small-scale column specimens of 300mm height were tested.

The test variables are:

1. The volumetric ratio of the FRP jackets.
2. The aspect ratio of the column section.
3. The area of longitudinal and lateral steel reinforcement.

The specimens were divided into three series depending on their aspect ratio (h/b of 1, 1.7, and 2.7, respectively). For each section aspect ratio, two groups of specimens were tested, one group corresponding to plain concrete and another group corresponding to reinforced concrete. In each group, four specimens were tested, one control specimen (without FRP) and three specimens with different areas of FRP jackets.

After studying the results, some of the important observations and conclusions have been drawn:

a. Increasing the area of FRP reinforcement increased the axial stress and axial strain that can be mobilized at failure of the column sections
b. Improvements in axial strength and strain were most significant for square columns and tended to decrease as the aspect ratio of the column section increased.
c. For the steel reinforced specimens, external confinement by FRP prevented spalling of the concrete cover and premature buckling of the longitudinal steel bars, leading to substantially improved performance.
d. The rate of increase of the measured average lateral strain with the axial strain tended to decrease as the aspect ratio of the column section and also as the area or stiffness of the FRP jacket increased.

Mirmiran et al. discussed how FRP materials significantly enhance the strength, ductility and durability of concrete columns. The longitudinal fibers serve as flexural reinforcement, while hoop fibers provided confinement and shear strength.

Analytical and experimental studies indicated higher compression and flexural strength as well as excellent pseudo-ductile characteristics. A new confinement model was proposed to quantify the gain in strength of column, which was confined with FRP materials. Test result indicated that the strength of a fiberglass/epoxy tube with 3-mm thickness filled with concrete is about triple the strength of a standard concrete cylinder.

An experimental study of the failure modes of reinforced concrete beams strengthened with prestressed carbon composite plates, by H.N.Garden, and L.C.Hollaway, 1998, tested two reinforced concrete beams of 1.0 and 4.5 m lengths in four point bending after strengthening them with externally bonded carbon fiber reinforced polymer plates. The study concluded that:

Carbon fiber reinforced polymers are high strength materials so the concrete itself may fail at the ultimate limit state in non-prestressed beams. Prestressing of the composite may shift failure away from the relatively weak tensile zone of the concrete. It has been shown that the adhesive may crack shortly before beam failure, though this does not appear to have any direct influence on structural performance. Glass fiber reinforced polymer (GFRP) blocks bonded to the composite plate are suitable for accommodating anchorage bolts and do not suffer a loss of integrity when the composite plate fractures in tension.

If plate separation initiates at the base of a shear crack due to the vertical displacement at this position in a non-prestressed beam, then a high prestress is likely to be required to enable the full flexural capacity of the section to be reached. An externally prestressed member may not be able to reach its full flexural capacity unless the section is additionally reinforced in shear.

Another experimental study by E. David, Ch. Gjelal, F.Buyle-Bodin, 1998, its divides into two parts: initially preloaded beams and repaired them with epoxy-bonded glass fiber plates, then beams were simply strengthened with several composite materials, three different composite materials have been used: Delmat glass fiber plates, Jitec glass fiber rods, and Sikadur carbon fiber sheets.

An experimental study by Ahmed Ghobarah, 2001, to develop effective selective rehabilitation schemes for reinforced concrete beam-column joints using advanced composite materials, the results have concluded that:

- GFRP jacket was capable of increasing the shear resistance of the joint.
- The proposed rehabilitation techniques for beam-column joints were successful in eliminating or delaying the shear mode of failure.
- The rehabilitation techniques were found to be effective and simple to install with minimum disruption to the function of the structure. When using FRP joint rehabilitation systems, anchoring of the edges of the FRP is important because the joint area is small, and there is need to develop the strength of the fiber.

  In addition, anchoring of the FRP is important in providing confinement to the joint.

Chapter 2

Rehabilitation Methods and Materials Used

2.1: Introduction

This chapter contains descriptions of old and new methods of rehabilitation concrete structures; and the properties of materials used in each method.

2.2: Old rehabilitation methods

The rehabilitation of structures is not new; several rehabilitation techniques were developed around the world over the past two decades and used to achieve a desired improvement.

1. Repairing a damaged concrete surface is common for concrete structures.

   The damaged concrete has to be removed until a good layer of concrete is reached and then cleaned leaving the cavity ready for suitable patching, as shown in Figure (2.1).

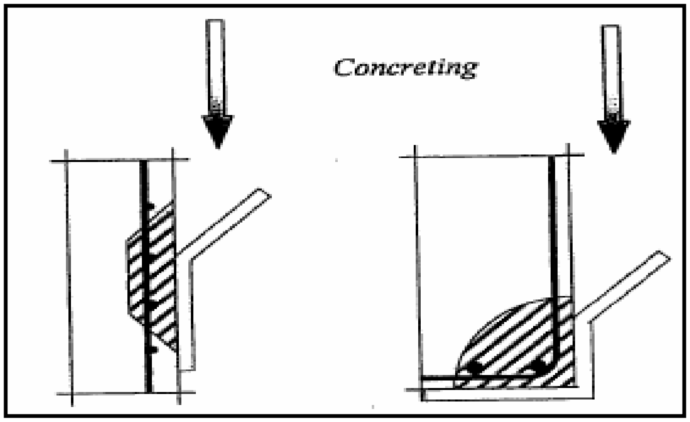

Figure (2.1) - Patch repairs to damaged concrete surfaces

(Al-Salloum, 2002)

To achieve the satisfactory structural performance, the new concrete or mortar mix should be chosen carefully.

2. Replacement of corroded reinforcement, together with the damaged concrete, when the corrosion of the reinforcement is severe.

   The replacement of steel bars is usually welded to good existing part of the old reinforcement as shown in Figure (2.2).

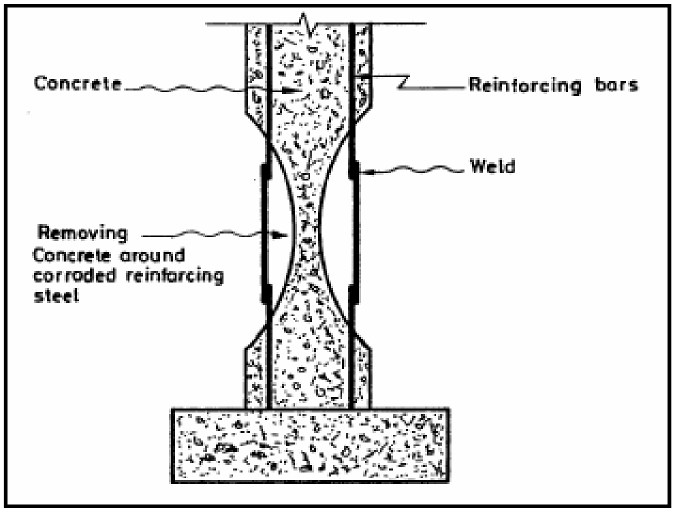

Figure (2.2) - Replacement of corroded reinforcement (Al-Salloum, 2002)

During this method, the structure may need to be temporarily supported.

3. Increasing the cross section of a concrete element as a simple strengthening method, when it is required to increase the capacity of the structural element to carry excessive loading.

   Jacketing the column with reinforced concrete jacket has proven to increase the column carrying capacity and the ductility.

   Also, to increase the cross-section of a reinforced concrete (RC) column, placing a steel tube around a column and filling the gap with concrete, this strengthens the column very effectively. See Figure (2.3).

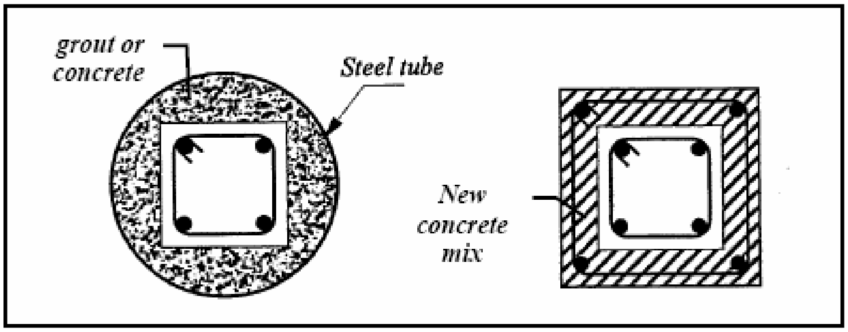

Figure (2.3) - Increasing cross-sections of columns (Al-Salloum, 2002)

4. Application of extra steel reinforcement.

   To increase the RC beam carrying capacity, placing additional reinforcement in the tension zone, protected by additional concrete cover.

   This was a very expensive process, equivalent to rebuilding a large section of the structure.

   Different examples of increasing the reinforcement of RC beams are shown in Figure (2.4).

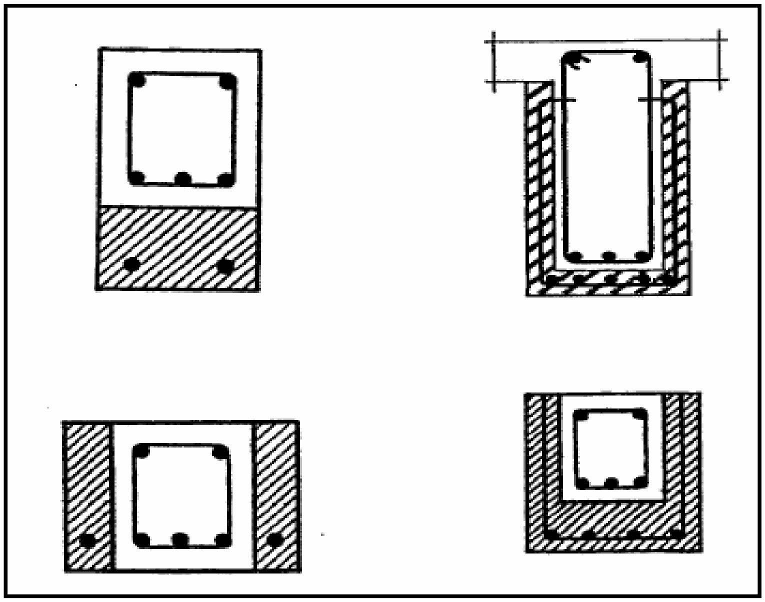

Figure (2.4) - Additional reinforcements for beams (Al-Salloum, 2002)

5. Externally bonding steel plates.

This method involves epoxy-bonded steel plates to the surfaces of members to be rehabilitated. See Figure (2.5).

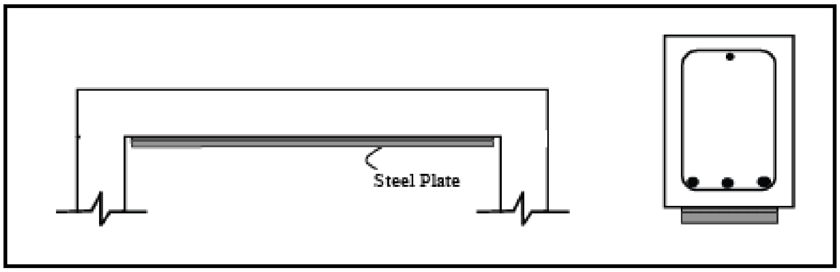

Figure (2.5) - Rehabilitation of beams with steel plates (Al-Salloum, 2002)

The first study of epoxy-bonded steel plates for strengthening and repairing of RC members is carried out by L'Hermite and Bresson in 1967.

This method has been applied to increase the carrying capacity of existing structures and bridges in many countries around the world.

The advantages of this technique are the relative simplicity of the implementation, the speed of application, and the relatively small consequential change in structural size.

Extensive experimental study investigations on the factors influencing the structural performance of plated concrete beams have been reported by Swamy et al 1987.

The main advantage of using this method in rehabilitation bridges is that it does not need closing down of the traffic during the repair.

From the various studies, it was observed that the bonding of steel plates to the tension faces of concrete beams can lead to a significant improvement in structural performance, under both service and ultimate loading conditions, and delays the usual cracks.

The disadvantages of this technique are:

a. Difficulty in handling of heavy steel plates for long span beams.
b. Relative difficulty in shaping the plate to suit the structural element to be repaired.
c. Forming clean butt joints in steel plates at small intervals.
d. Steel plates are not suitable to be wrapped or jacked around tapered members.
e. The possibility of high corrosion at the steel epoxy interface.
f. Limitation in available plate lengths (which are required in case of flexural strengthening of long girders).

An effective way of eliminating the corrosion problem and other disadvantages listed above is to replace steel plates with corrosion resistance synthetic materials such as fiber reinforced polymer (FRP) composites.

In addition to their higher corrosion resistance, many polymer composites have tensile and fatigue strengths that exceed those of steel.

2.3: New rehabilitation methods

The difficulties encountered in using steel plates technique and the ever-increasing demands for rehabilitation of structures have prompted engineers and researchers to search for better and more reliable and innovative solutions.

FRP composites exhibit several attractive properties, such as low weight to strength ratio, non-corrosive, high fatigue resistance, and high tensile strength.

In addition, some types of FRP composites are very flexible, it can be formed almost to any desired shape, light enough to be handled on the job site with no need for heavy equipment, and they occupy negligible space as compared to the existing structural members.

Only a few years ago, FRP composites are being considered for application to the rehabilitation of concrete structures.

However, they have been successfully used in a variety of industries such as aircraft, automobile, naval and marine boats.

The properties of FRP composites and their versatility have resulted in significant saving in construction costs and reduction in shut down time of facilities as compared to the old strengthening methods.

Research has shown that FRP can be used very efficiently in strengthening the concrete structural elements such as columns, beams, slabs, and walls.

Among many other applications, FRP composites may be used to strengthen concrete and masonry walls to better resist lateral loads as well as circular structures such as tanks and pipelines to resist internal pressure and reduce corrosion.

The method of strengthening structural element using composite materials, and properties of composite materials will be discussed in the following sections.

2.4: Composite materials

Polymer composites are defined as a matrix of polymeric material reinforced by fibers or other reinforcement with a discernible aspect ratio of length to thickness.

The major factors affecting the physical and mechanical performance of the FRP matrix composite are the orientation, length, shape and composition of the fibers, the mechanical properties of the matrix resin and the adhesion of the bond between the fibers and the matrix.

The matrix protects fibers from abrasion and it forms a protective barrier between the fibers and the environment, thus preventing attack from moisture, chemicals and oxidation, and its important in providing shear, transverse tensile and compression properties.

FRP composites can be used in the concrete structures in the following forms:

- Plates: at a face to improve the tension capacity.
- Bars: as reinforcement in beams and slabs replacing the steel bars.
- Cables: as tendons and post tension members in suspension and bridge girders.
- Wraps: around concrete members to confine concrete and improve the compressive strength.

FRP generally consisting of high strength carbon, aramid, or glass fibers in a polymeric matrix.

Carbon fibers:

Carbon fibers are the predominate reinforcement used to achieve high stiffness and high strength. The density of carbon fibers is $1900 kgm^{-3}$.

Aramid fibers:

Aramid fibers are very tough organic synthetic fibers; it's having high strengths up to 3000MPa, and a very low density around $1400 kgm^{-3}$.

Aramid fibers are fire resistant and perform well at high temperatures.

They are insulators of both electricity and heat, and are resistant to organic solvents, fuels and lubricants. It is generally the higher modulus material, which finds use in composites.

Glass fibers:

Glass fibers can be categorized into two sets:

One with elastic modulus around 70GPa and with strengths after processing in the range 1000-2000 MPa, another with a modulus around 85 GPa with strengths after processing in the range 2000-3000 MPa.

The density of glass fiber is about $2500 kgm^{-3}$. Glass fibers are the most commonly used reinforcing fibers, because they have good properties in an absolute sense and relative to weight, they also have very good processing characteristics and they are inexpensive. The processing characteristics of particular types of glass fiber have been modified and optimized over many years to achieve the required performance, such as choppability, low static buildup and conformance to complex shape, and resin compatibility requirements such as fast wet out, good fiber/matrix adhesion.

Typical properties of various types of fiber material are provided in Table (2.1).

Table (2.1) - Typical properties of fibers (Feldman 1989, Kim 1995).

Material		Elastic Modulus (GPa)	Tensile Strength (MPa)	Ultimate Tensile Strain (%)
Carbon	High strength	215-235 ≅ steel	3500-4800	1.4-2.0
	Ultra high strength	215-235 ≅ steel	3500-6000	1.5-2.3
	High modulus	350-500 (1.75-2.5)Es	2500-3100	0.5-0.9
	Ultra high modulus	500-700 (2.5-3.5)Es	2100-2400	0.2-0.4
Glass	E	70 0.35Es	1900-3000	3.0-4.5
	S	85-90 (0.43-0.45)Es	3500-4800	4.5-5.5
Aramid	Low modulus	70-80 (0.35-0.40)Es	3500-4100	4.3-5.0
	High modulus	115-130 (0.58-0.65)Es	3500-4000	2.5-3.5

The mechanical properties of FRP vary significantly from one product to another. Factors such as volume fraction (ratio of volume of fibers to the volume of matrix material) and type of fiber and resin, fiber orientation, dimension and quality control during manufacturing have a major role in establishing the properties of the product.

Reinforcements are available in a variety of configurations of which there are main categories:

- Unidirectional: in which all the fibers lie in one direction.
- Bidirectional: in which the fibers lie at 90o to one another.
- Random: in which the fibers are randomly distributed and are in plane.

Composite materials are not homogeneous; their properties are dependent on many factors, such as the type of fiber, quantity of fiber, and the configuration of the reinforcement.

They are generally completely elastic up to failure and exhibit neither a yield point nor a region of plasticity. They tend to have low strain to failure (less than 3%). The resulting area under the stress/strain curve, which represents the work done to failure, is relatively small when compared to many metals. The properties of composites are dependent on the properties of the fiber and the matrix, the proportion of each, and the configuration of the fibers.

If all fibers are aligned in one direction then the composite is relatively stiff and strong in that direction, but in the transverse direction it has low modulus and low strength.

FRP exhibit several attractive properties such as low weight to strength ratios, non-corrosiveness, ease of application, but the most important properties of FRP materials is their high tensile strength.

The strength is about twice that of prestressing steel strands, and fairly high compared with ordinary steel as shown in Figure (2.6).

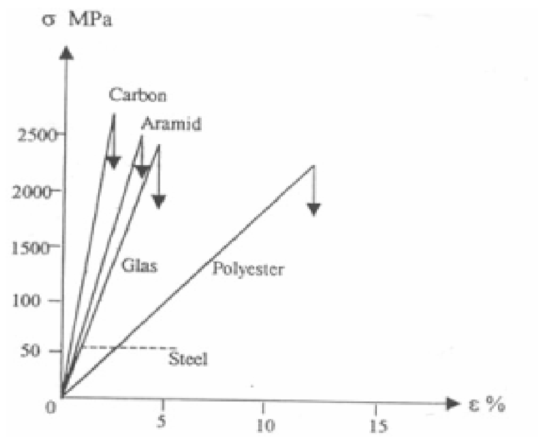

Figure (2.6) - Materials for strengthening of concrete (Mukherjee, 2001)

The tensile strength of FRP composite system is affected by the tensile strength of the fibers, volume fraction, resin system, and the bond performance of fibers and matrix. FRP composites reach their ultimate tensile strength without exhibiting any yielding of the material (linear up to failure).

The main advantages of FRP composite in form of plate are as follows:

- Strength of plates: FRP composite plates may be designed with components to meet a particular purpose and may comprise varying proportions of different fibers. The ultimate strength of the plates is at least three times the ultimate strength of steel for the same cross-sectional area.
- Weight of plates: the density of FRP composite plates is only 20% of the density of steel. Composite plates do not require extensive jacking and support systems to move and hold in place.
- Versatile design of systems: composite plates are of unlimited length, may be fixed in layers to suit strengthening requirements, and are so thin that

fixing in two directions may be accommodated by varying the adhesive thickness.

- Durability of strengthening system: composite plates non-corrosive.
- Improved fire resistance: composite plates are a low conductor of heat, thus reducing the effect of fire or heat on the underlying adhesives.
- Maintenance of strengthening system: composite plates will not require maintenance painting, which reduces the whole life cost of this system.

2.5: Rehabilitation models

Various materials and methods that is available for rehabilitation of concrete structures.

To achieve durable rehabilitation it is necessary to consider the factors affecting the design and selection of rehabilitation systems as parts of a composite system.

Selection of a rehabilitation material is one of the many interrelated steps; equally important are surface preparation, the method of application, construction practices, and inspection.

The critical factors affecting the durability of concrete structures rehabilitation in practice are shown in Figure (2.7).

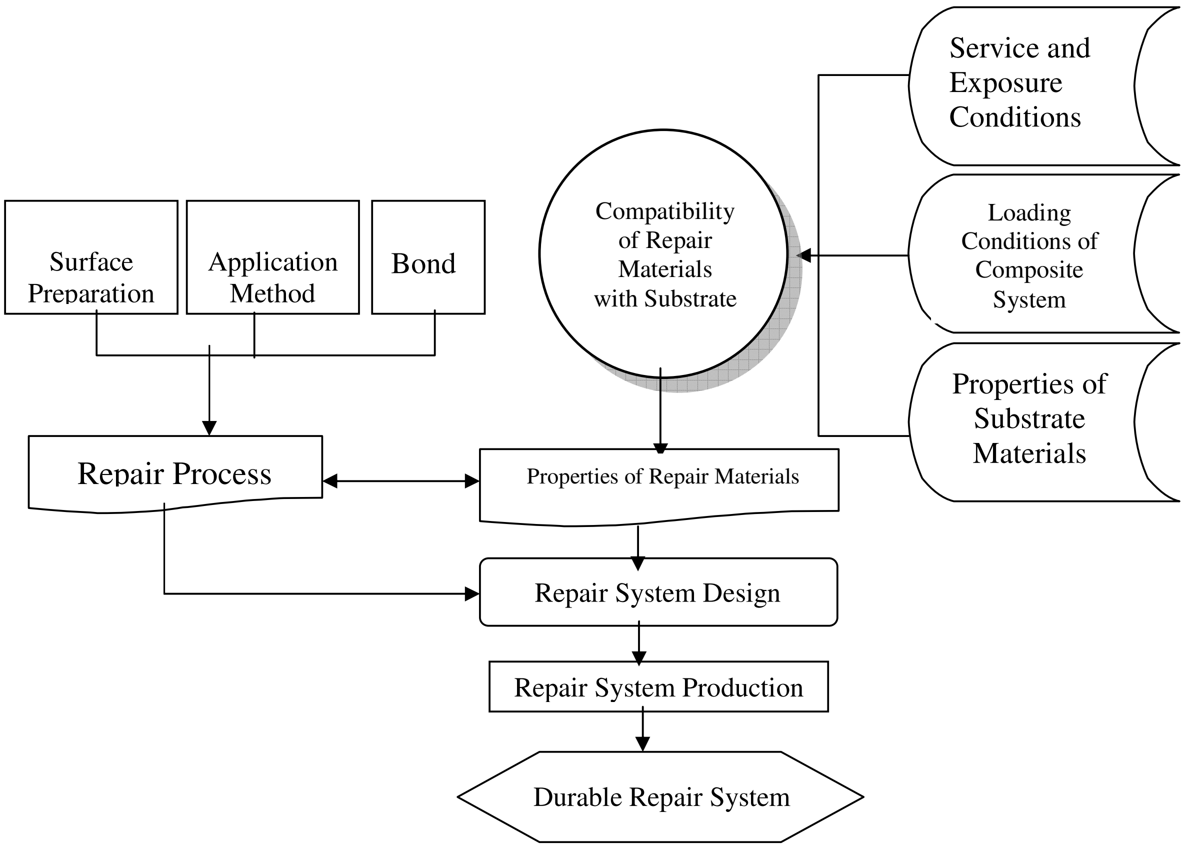

Figure (2.7) -Factors affecting the durability of concrete repair systems

(Emmons and Vaysburd 1995)

Economical solutions depend upon an understanding of the materials and experience of what they can safely achieve.

One or more rehabilitation methods may be selected, depending on the nature of the damage.

One of strengthening techniques is with FRP composite plate bonding.

These composite plates may use different reinforcement materials, in different proportions, and within different matrix materials.

It's also being of different lengths, and multiple layers may be used. These may be fixed at any required geometry on the surface of the structure.

The plates may be stressed or unstressed, and the ends mechanically anchored or bonded by adhesive only.

The range of structural needs and deteriorate for which FRP composite plate bonding already offers an appropriate solution is very wide, as illustrated in Table (2.2).

Table (2.2) - Applications for composite plate bonding (Hollaway,L.C.,1999)

Structural need/deficiency	FRP composite plate bonding solution	Comments
Corrosion of reinforcement in reinforced concrete	Replacement of lost reinforcement by plates of equivalent effect	Damaged concrete must be replaced without impairing behavior of plates
Inadequate flexural capacity of reinforced concrete	Design FRP composite plate bonding solution to add tensile elements	Extent of strengthening limited by capacity of concrete in compression. Plates anchored by bond or mechanically at their ends
Inadequate stiffness or serviceability of cracked reinforced concrete structure	Add external prestress by means of a stressed composite plate	
Avoidance of sudden failure by cracking of cast iron	Addition of FRP composite plate bonding, either stressed or unstressed, to tensile face	
Increase in structural capacity of timber structures	Increase in stiffness and ultimate capacity by plate bonding	Particularly appropriate with historic structures
Enhancement of shear capacity	Enhance by external bonding of stressed plates, or by web reinforcement	Web reinforcement techniques little researched

2.6: Comparison between CFRP plates and steel plates

Table (2.3) - Comparison CFRP plates vs. steel plates (Sika CarboDur, 2003)

Properties	CFRP Plates	Steel Plates
Tensile strength	Very high	Low vs. CFRP
E-modulus	Three different modulus	Given
Fatigue resistance	Excellent	Good
Corrosion	Non corrosive	Corrosive
Weight	Lightweight	Heavy
Fire precaution	None	Must be anchored
Length of the plate	No limitation	Limitation by weight
Handling	Easy-Flexible plates	Difficult-Stiff plates
Crossing	Easy-Thin plates	Difficult, expensive factory made
Application cost	Low	Medium to high

Chapter 3

Strengthening of Main Structural

Elements Using FRP

3.1: Introduction

Deterioration of reinforced concrete structures is one of the major problems of the construction today.

A large number of structures constructed in the past using the older design codes are structurally unsafe according to today's design codes. And some structures have to carry higher loads, and other need to be rehabilitated due to errors made during the design or construction stages.

Since replacement of that structure incurs a huge amount of money and time, strengthening of existing reinforced concrete structures has become one of the most important activities in civil engineering, and has become the acceptable way of improving their load carrying capacity and extending their service lives.

Various methods are available to strengthen those structures, strengthening of building by bonding steel plates have been applied in construction for over twenty years.

The main disadvantages of using steel plates are steel corrosion in the adhesion zone and heavy weight of single plates.

The use of composite materials as alternatives to the steel plate for strengthening has been established as an effective method applicable to many types of concrete structures such as beams, columns, and walls.

Strengthening of RC beams and columns using FRP will be discussed in this chapter according to ACI 440.

3.2: General Design Considerations

FRP strengthening systems should be designed in accordance with ACI 318-99 strength and serviceability requirements, using the load factors stated in ACI 318-99. The strength reduction factors required by ACI 318-99 should also be used.

While FRP systems are effective in strengthening members for flexure and shear and providing additional confinement, other modes of failure, such as punching shear and bearing capacity of footings, may be unaffected by FRP systems. It is important to ensure that all members of a structure are capable of withstanding the anticipated increase in loads associated with the strengthened members. (8.2.2 ACI 440)

3.2.1: Environmental Considerations

Environmental conditions uniquely affect resins and fibers of various FRP systems. The mechanical properties (for example, tensile strength, strain, and elastic modulus) of some FRP systems degrade under exposure to certain environments, such as alkalinity, salt water, chemicals, ultraviolet light, high temperatures, high humidity, and freezing and thawing cycles.

The engineer should select an FRP system based on the known behavior of that system in the anticipated service conditions. if the FRP system is located in a relatively benign environment, such as indoors, the reduction factor is closer to unity. If the FRP system is located in an aggressive environment where prolonged exposure to high humidity, freeze-thaw cycles, salt water, or alkalinity is expected, a lower reduction factor should be used.

See Table (3.1)

Table (3.1) - Environmental-reduction factor for various FRP systems and exposure conditions (ACI 440.2R-02, Table 8.1)

Exposure conditions	Fiber and resin type	Environmental reduction factor C_E
Interior exposure	Carbon/epoxy	0.95
	Glass/epoxy	0.75
	Aramid/epoxy	0.85
Exterior exposure (bridges, piers, and unenclosed parking garages)	Carbon/epoxy	0.85
	Glass/epoxy	0.65
	Aramid/epoxy	0.75
Aggressive environment (chemical plants and waste water treatment plants)	Carbon/epoxy	0.85
	Glass/epoxy	0.50
	Aramid/epoxy	0.70

3.2.2: Design materials properties

The material properties used in design equations should be reduced based on the environmental exposure condition.

The design ultimate tensile strength should be determined using the environmental reduction factor given in Table (3.1) for the appropriate fiber type and exposure condition.

$$f_{fu} = C_E \, f^*_{fu} \qquad \text{Equation (3.1)}$$

Where:

f_{fu} = design ultimate tensile strength of FRP, (MPa)

f^{*}_{fu} = ultimate tensile strength of the FRP material as reported by the manufacturer, (MPa)

Similarly, the design rupture strain should also be reduced for environmental-exposure conditions.

$$\varepsilon_{fu} = C_E \varepsilon^{*}_{fu} \qquad \text{Equation (3.2)}$$

Where:

ε_{fu} = design rupture strain of FRP reinforcement, (mm/mm)

ε^{*}_{fu} = ultimate rupture strain of the FRP reinforcement, (mm/mm)

Because FRP materials are linearly elastic until failure, the design modulus of elasticity can then be determined from Hooke's law. The expression for the modulus of elasticity (E_f), given in Equation (3.3)

$$E_f = \frac{f_{fu}}{\varepsilon_{fu}} \qquad \text{Equation (3.3)}$$

3.3: Strengthening of RC elements using FRP

In general, RC beams fail in two types:

- Flexural failure.
- Shear failure.

RC beams must be designed to develop its full flexural capacity to assure a ductile flexural failure mode under extreme loading.

The shear failure of RC beam is sudden and brittle in nature, it gives no advance warning prior to failure, and it's more dangerous than the flexural failure.

However, many of RC structures had encountered shear problems due to various reasons, such as error in design calculations and improper detailing of shear reinforcement, poor construction practices, changing the function of a structure from a lower service load to a higher service load, and reduction in shear reinforcement steel area causing corrosion in service environments.

Among different types of FRP materials, CFRP appears to be the most applicable in strengthening techniques, because of high strength, durability and fatigue characteristics, best resistance to chemical corrosion compared to other FRP, ease in handling at construction sites, due to the light weight, which reduce labor costs.

Before beginning to strengthen a concrete structure the designer should determine whether it is suitable and more economic to strengthen the existing structure or to replace it. If the internal reinforcement has started to corrode or if the concrete is attacked by chemicals, the corroded bars or the attacked concrete or both of them should be removed and replaced before strengthening.

The composite plate bonding generates significant improvements in ultimate capacity of reinforced concrete members and that the stiffness of a member is increased, causing reductions in the maximum overall deflection and strains throughout the cross-section of the beam.

Today there are several types of FRP strengthening systems, which are summarized below:

- Wet lay-up systems (hand lay-up).

- Systems based on prefabricated elements.
- Special systems, e.g. automated wrapping, prestressing etc.

The three types shown in Figure (3.1).

Figure (3.1) - (a) Hand lay-up of CFRP sheets or fabrics. (b) Application of prefabricated strips. (c) Automated RC column wrapping.

The basic FRP strengthening technique, which is most widely applied, involves the manual application of either wet lay-up (so-called hand lay-up) or prefabricated systems by means of cold cured adhesive bonding.

On site the concrete surface has to be sandblasted, to remove the laitance, and after sandblasting the surface must be cleaned with compressed air or water. The surface of the CFRP plate must be clean so it is best to keep any peel ply on until just before applying adhesive.

3.3.1: Flexural strengthening

Reinforced concrete elements, such as beams and columns, may be strengthened in flexure through the use of FRP composites epoxy-bonded to the tension face of the beam, with the direction of fibers parallel to that of high tensile stresses (member axis). The concept is illustrated in Figure (3.2).

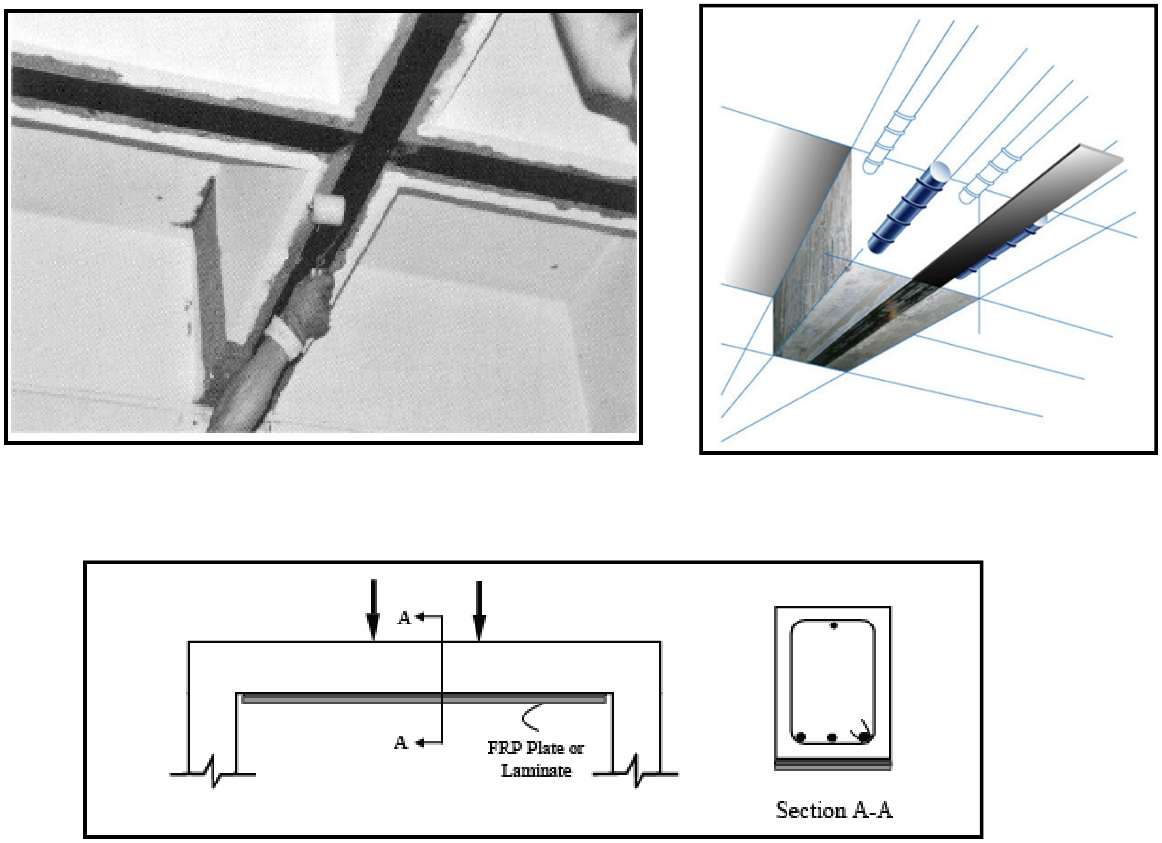

Figure (3.2) - Flexural strengthening of RC beams with CFRP

Bonding FRP reinforcement to the tension face of a concrete flexural member with fibers oriented along the length of the member will provide an increase in flexural strength. Increases in overall flexural strength from 10 to 160% have been documented (Meier and Kaiser 1991; Ritchie et al. 1991; Sharif et al. 1994).

3.3.1.1: Assumptions

The following assumptions are made in calculating the flexural resistance of a section strengthened with an externally applied FRP system:

- Design calculations are based on the actual dimensions, internal reinforcing steel arrangement, and material properties of the existing member being strengthened;
- The strains in the reinforcement and concrete are directly proportional to the distance from the neutral axis, that is, a plane section before loading remains plane after loading;
- There is no relative slip between external FRP reinforcement and the concrete;
- The shear deformation within the adhesive layer is neglected since the adhesive layer is very thin with slight variations in its thickness;
- The maximum usable compressive strain in the concrete is 0.003;
- The tensile strength of concrete is neglected;
- The FRP reinforcement has a linear elastic stress-strain relationship to failure.

3.3.1.2: Analysis of design

These design recommendations are based on limit states design principles.

The strength-design approach requires that the design flexural strength of a member exceed its required moment strength as indicated by Equation (3.4). Design flexural strength ΦM_n refers to the nominal strength of the member multiplied by a strength-reduction factor, and the required moment strength M_u refers to the load effects calculated from factored loads.

$$\Phi M_n \geq M_u \qquad \text{Equation (3.4)}$$

A strength reduction factor given by Equation (3.5) should be used, where ε_s is the strain in the steel at the ultimate limit state.

$$\phi = \begin{cases} 0.90 & for \ \ \varepsilon_s \geq 0.005 \\ 0.7 + \dfrac{0.20(\varepsilon_{sy} - \varepsilon_s)}{0.005 - \varepsilon_{sy}} & for \ \ \varepsilon_{sy} \angle \ \varepsilon_s \angle 0.005 \\ 0.70 & for \ \ \varepsilon_s \leq \varepsilon_{sy} \end{cases} \qquad \text{Equation (3.5)}$$

(ACI 440.2R-02, Eq. 9-5)

Where:

ε_{sy}: strain corresponding to the yield strength of nonprestressed steel reinforcement

This equation sets the reduction factor at 0.90 for ductile sections and 0.70 for brittle sections where the steel does not yield, and provides a linear transition for the reduction factor between these two extremes.

Adequate ductility is achieved if the strain in the steel at the point of concrete crushing or failure of the FRP, including delamination or debonding, is at least 0.005.

3.3.1.3: Failure modes

The flexural strength of a section depends on the controlling failure mode. The following flexural failure modes should be investigated for an FRP strengthened section (GangaRao and Vijay, 1998):

- Crushing of the concrete in compression before yielding of the reinforcing steel;
- Yielding of the steel in tension followed by rupture of the FRP laminate;
- Yielding of the steel in tension followed by concrete crushing (is the most desirable);
- Shear/tension delamination of the concrete cover (cover delamination);
- Debonding of the FRP from the concrete substrate (FRP debonding).

Concrete crushing is assumed to occur if the compressive strain in the concrete reaches its maximum usable strain ($\varepsilon_c = \varepsilon_{cu} = 0.003$). Rupture of the FRP laminate is assumed to occur if the strain in the FRP reaches its design rupture strain ($\varepsilon_f = \varepsilon_{fu}$) before the concrete reaches its maximum usable strain.

Cover delamination or FRP debonding can occur if the force in the FRP cannot be sustained by the substrate. In order to prevent debonding of the FRP laminate, a limitation should be placed on the strain level developed in the laminate. Equation (3.6) gives an expression for a bond-dependent coefficient k_m.

$$k_m = \begin{cases} \dfrac{1}{60\ \varepsilon_{fu}}\left(1 - \dfrac{nE_f t_f}{360000}\right) \le 0.90 \quad for\ nE_f t_f \le 180000 \\ \dfrac{1}{60\ \varepsilon_{fu}}\left(\dfrac{90000}{nE_f t_f}\right) \le 0.90 \quad for\ nE_f t_f > 180000 \end{cases}$$

Equation (3.6)SI

(ACI 440.2R-02, Eq. 9-2)

The number of plies n used in this equation is the number of plies of FRP flexural reinforcement at the location along the length of the member where the moment strength is being computed.

t_f is the nominal thickness of one ply of the FRP reinforcement,(mm).

3.3.1.4: Strain level in FRP reinforcement

It is important to determine the strain level in the FRP reinforcement at the ultimate-limit state. Because FRP materials are linearly elastic until failure, the level of strain in the FRP will dictate the level of stress developed in the FRP. The maximum strain level that can be achieved in the FRP reinforcement will be governed by either the strain level developed in the FRP at the point at which concrete crushes, the point at which the FRP ruptures, or the point at which the FRP debonds from the substrate. This maximum strain or the effective strain level in the FRP reinforcement at the ultimate-limit state can be found from Equation (3.7).

$$\varepsilon_{fe} = \varepsilon_{cu}\left(\frac{h-c}{c}\right) - \varepsilon_{bi} \le k_m \varepsilon_{fu}$$

Equation (3.7)

(ACI 440.2R-02, Eq. 9-3)

Where:

ε_{bi} = is the initial substrate strain

h = overall thickness of a member, (mm).

c = distance from extreme compression fiber to the neutral axis, (mm).

If the first term in the equation controls, concrete crushing controls flexural failure of the section. If the second term controls, FRP failure (rupture or debonding) controls flexural failure of the section.

The effective stress level in the FRP reinforcement is the maximum level of stress that can be developed in the FRP reinforcement before flexural failure of the section. This effective stress level can be found from the strain level in the FRP, assuming perfectly elastic behavior.

$$f_{fe} = E_f \, \varepsilon_{fe} \qquad \text{Equation (3.8)}$$

Where:

f_{fe} = effective stress in the FRP; stress level attained at section failure, (MPa)

ε_{fe} = effective strain level in FRP reinforcement; strain level attained at section failure, (mm/mm)

3.3.1.5: Application to a singly reinforced rectangular section

For singly reinforced rectangular section Figure (3.3) illustrates the internal strain and stress distribution under flexure at the ultimate limit state.

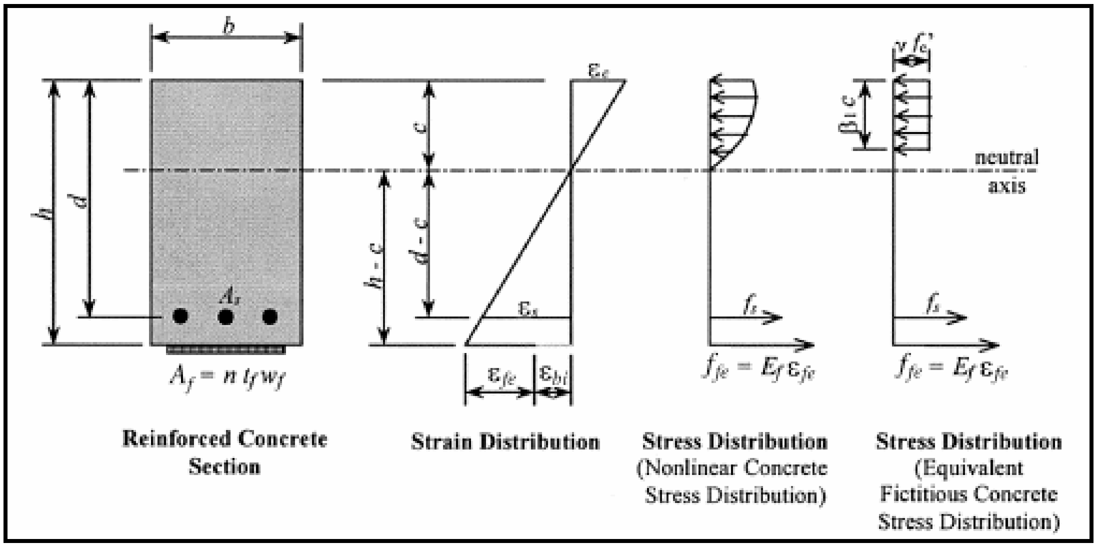

Figure (3.3) - Internal strain and stress distribution for a rectangular section under flexure at ultimate stage (ACI 440.2R-02, Fig. 9.2)

For any assumed depth to the neutral axis c, the strain level in the FRP reinforcement can be computed from Equation (3.7), and the effective stress level in the FRP reinforcement can be computed from Equation (3.8).

Based on the strain level in the FRP reinforcement, the strain level in the nonprestressed tension steel can be found from Equation (3.9) using strain compatibility.

$$\varepsilon_s = (\varepsilon_{fe} + \varepsilon_{bi})\left\{\frac{d-c}{h-c}\right\}$$ Equation (3.9), (ACI 440.2R-02, Eq. 9-8)

d is the distance from extreme compression fiber to the neutral axis, (mm).

The stress in the steel is calculated from the strain level in the steel assuming elastic-plastic behavior.

$$f_s = E_s\varepsilon_s \le f_y$$ Equation (3.10), (ACI 440.2R-02, Eq. 9-9)

With the strain and stress level in the FRP and steel reinforcement determined for the assumed neutral axis depth, internal force equilibrium may be checked using Equation (3.11).

$$c = \frac{A_s f_s + A_f f_{fe}}{\gamma f`_c \beta_1 b}$$ Equation (3.11), (ACI 440.2R-02, Eq. 9-10)

Where:

A_s = area of nonprestressed steel reinforcement, (mm^2)

A_f = area of FRP external reinforcement, (mm^2)

f_c' = specified compressive strength of concrete, (MPa)

b = width of rectangular cross section, (mm)

The terms γ and β_1 in Equation (3.11) are parameters defining a rectangular stress block in the concrete equivalent to the actual nonlinear distribution of stress. If concrete crushing is the controlling mode of failure (before or after steel yielding), γ and β_1 can be taken as the values associated with the Whitney stress block ($\gamma = 0.85$ and β_1 from Section 10.2.7.3 of ACI 318-99).

The nominal flexural strength of the section with FRP external reinforcement can be computed from Equation (3.12). An additional reduction factor ψ_f is applied to the flexural strength contribution of the FRP reinforcement. A factor $\psi_f = 0.85$ is recommended.

$$M_n = A_s f_s\left(d - \frac{\beta_1 c}{2}\right) + \psi_f A_f f_{fe}\left(h - \frac{\beta_1 c}{2}\right)$$ Equation (3.12)

(ACI 440.2R-02, Eq. 9-11)

The stress level in the steel reinforcement under service loads can be calculated based on a cracked elastic analysis of the strengthened reinforced concrete section, as indicated by Equation (3.13).

$$f_{s,s} = \frac{\left[M_s + \varepsilon_{bi} A_f E_f\left(h - \frac{kd}{3}\right)(d - kd)E_s\right]}{A_s E_s\left(d - \frac{kd}{3}\right)(d - kd) + A_f E_f\left(h - \frac{kd}{3}\right)(h - kd)} \leq 0.80 f_y$$ Equation (3.13)

(ACI 440.2R-02, Eq. 9-12)

Where:

E_s = modulus of elasticity of steel, (MPa)

E_f = tensile modulus of elasticity of FRP, (MPa)

f_y = specified yield strength of nonprestressed steel reinforcement, (MPa)

k = ratio of the depth of the neutral axis to the reinforcement depth measured on the same side of neutral axis

The distribution of strain and stress in the reinforced concrete section is shown in Figure (3.4).

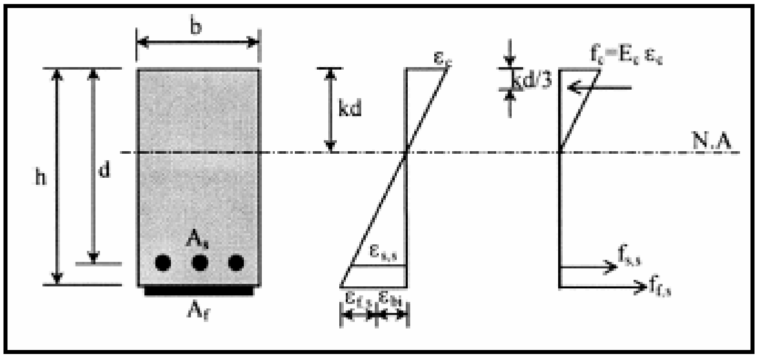

Figure (3.4) - Elastic strain and stress distribution (ACI 440.2R-02, Fig. 9.3)

And the stress in the FRP reinforcement under service loads can be computed using Equation (3.14).

$$f_{f,s} = f_{s,s}\left(\frac{E_f}{E_s}\right)\frac{h-kd}{d-kd} - \varepsilon_{bi}E_f \le \begin{cases} 0.20 f_{fu}, & for\ \ glass\ FRP \\ 0.30 f_{fu}, & for\ \ aramid\ FRP \\ o.55 f_{fu}, & for\ \ carbon\ FRP \end{cases} \qquad \text{Equation(3.14)}$$

(ACI 440.2R-02, Eq. 9-13)

3.3.1.6: Recommendations

- Maximum spacing s_f between parallel plates or fabrics = min (0.2ℓ, 5h, 0.4ℓc) where ℓ= span length, h= total depth, ℓc=length of cantilever (FIB Bulletin No 14).

- Minimum distance to the edge of a beam = concrete cover to existing reinforcement.
- Crossing of strips is allowed, with bonding in the crossing area.

3.3.2: Shear strengthening

FRP systems have been shown to increase the shear strength of existing concrete beams and columns by wrapping or partially wrapping the members.

Shear strengthening using external FRP may be provided at locations of expected plastic hinges or stress reversal and for enhancing post yield flexural behavior of members in moment frames resisting seismic loads only by completely wrapping the section.

The three types of FRP wrapping schemes used to increase the shear strength of prismatic, rectangular beams, or columns are illustrated in Figure (3.5).

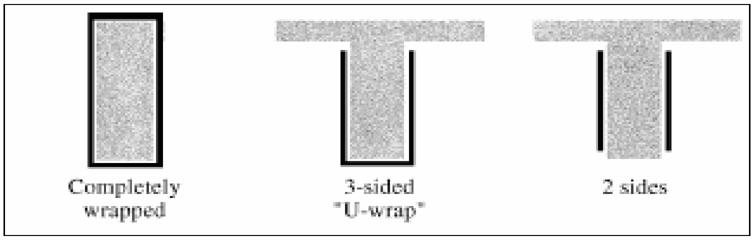

Figure (3.5) - Typical wrapping schemes for shear strengthening using FRP laminates (ACI 440.2R-02, Fig. 10.1)

Completely wrapping the FRP system around the section on all four sides is the most efficient wrapping scheme and is most commonly used in column applications where access to all four sides of the column is usually available. In beam applications, where an integral slab makes it impractical to completely wrap the member, the shear strength can be improved by wrapping the FRP system around three sides of the member (U-wrap) or bonding to the two sides of the member.

In all wrapping schemes, the FRP system can be installed continuously along the span length of a member or placed as discrete strips. See Figure (3.6).

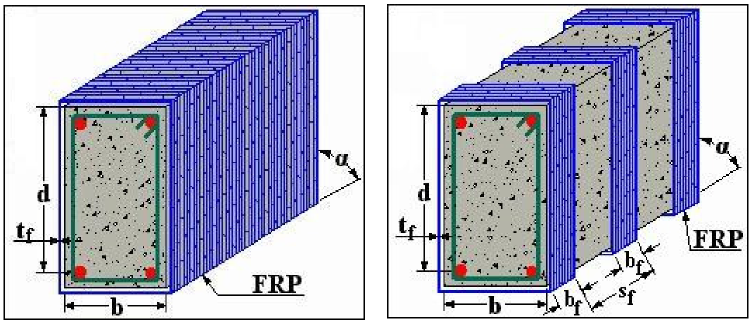

Figure (3.6) - External FRP reinforcement in the form of continuous and discrete strips (Sika CarboDur, 2003)

3.3.2.1: Design analysis

The nominal shear strength of a concrete member strengthened with an FRP system should exceed the required shear strength Equation (3.15).

$$\Phi V_n \geq V_u \qquad \text{Equation (3.15)}$$

The required shear strength on an FRP strengthened concrete member should be computed with the load factors required by ACI 318-99. The shear strength should be calculated using the strength-reduction factor Φ, required by ACI 318-99.

The nominal shear strength of an FRP-strengthened concrete member can be determined by adding the contribution of the FRP reinforcing to the contributions from the reinforcing steel (stirrups, ties, or spirals) and the concrete Equation (3.16).

$\Phi V_n = \Phi(V_c + V_s + \psi_f V_f)$ Equation (3.16)

Where:

V_c = nominal shear strength provided by concrete with steel flexural reinforcement, (N).

V_s = nominal shear strength provided by steel stirrups, (N)

V_f = nominal shear strength provided by FRP stirrups

An additional reduction factor ψ_f is applied to the contribution of the FRP system. See Table (3.2).

Table (3.2) - Recommended additional reduction factors for FRP shear reinforcement (ACI 440.2R-02, Table 10.1)

$\psi_f = 0.95$	Completely wrapped members
$\psi_f = 0.85$	Three sides U-wraps or bonded face piles

The contribution of the FRP system to shear strength of a member is based on the fiber orientation and an assumed crack pattern; Figure (3.7) illustrates the dimensional variables used in shear-strengthening calculations for FRP laminates.

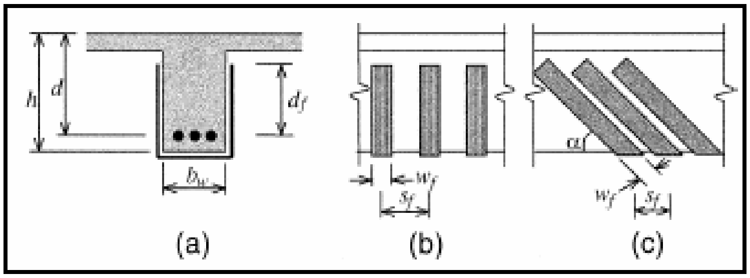

Figure (3.7) - Illustration of the dimensional variables used in shear-strengthening calculations for strengthening using FRP laminates.

(ACI 440.2R-02, Fig.10.2)

The shear strength provided by the FRP reinforcement can be determined by calculating the force resulting from the tensile stress in the FRP across the assumed crack. The shear contribution of the FRP shear reinforcement is then given by Equation (3.17).

$$V_f = \frac{A_{fv} f_{fe} (\sin\alpha + \cos\alpha) d_f}{s_f}$$ Equation (3.17)

(ACI 440.2R-02, Eq. 10-3)

Where

$A_{fv} = 2nt_f w_f$ Equation (3.18), (ACI 440.2R-02, Eq. 10-4)

A_{fv} = area of FRP shear reinforcement with spacing s_f, (mm^2)

d_f = depth of FRP shear reinforcement, (mm)

s_f = spacing FRP shear reinforcing, (mm)

w_f = width of the FRP reinforcing plies, (mm)

α = angle of inclination of stirrups or spirals, degrees

n = number of plies

t_f = the nominal thickness of one ply of the FRP reinforcement,(mm)

The tensile stress in the FRP shear reinforcement at ultimate is directly proportional to the level of strain that can be developed in the FRP shear reinforcement at ultimate.

$$f_{fe} = \varepsilon_{fe} E_f \qquad \text{Equation (3.19)}$$

3.3.2.2: Effective strain in FRP laminates

The effective strain is the maximum strain that can be achieved in the FRP system at the ultimate load stage and is governed by the failure mode of the FRP system and of the strengthened reinforced concrete member. The engineer should consider all possible failure modes and use an effective strain representative of the critical failure mode.

The following subsections give guidance on determining this effective strain for different configurations of FRP laminates used for shear strengthening of reinforced concrete members.

1- Completely wrapped members

For reinforced concrete column and beam members completely wrapped by the FRP system, loss of aggregate interlock of the concrete has been observed to occur at fiber strains less than the ultimate fiber strain.

$$\varepsilon_{fe} = 0.004 \leq 0.75\varepsilon_{fu} \qquad \text{Equation (3.20a)}$$

2- Bonded U-wraps or bonded face plies

FRP systems that do not enclose the entire section (two- and three-sided wraps) have been observed to delaminate from the concrete before the loss of aggregate interlock of the section.

$$\varepsilon_{fe} = \kappa_v \varepsilon_{fu} \leq 0.004 \qquad \text{Equation (3.20b)}$$

The bond reduction coefficient κ_v is a function of the concrete strength, the type of wrapping scheme used, and the stiffness of the laminate. it can be computed from Equation (3.21)through (3.24).

$$K_v = \frac{k_1 k_2 L_e}{11{,}900\,\varepsilon_{fu}} \leq 0.75$$ Equation (3.21)SI

(ACI 440.2R-02, Eq. 10-7)

The active bond length $\boldsymbol{L_e}$ is the length over which the majority of the bond stress is maintained. This length is given by Equation (3.22).

$$L_e = \frac{23{,}300}{\left(n t_f E_f\right)^{0.58}}$$ Equation (3.22)SI, (ACI 440.2R-02, Eq. 10-8)

The bond-reduction coefficient also relies on two modification factors, k_1 and k_2, that account for the concrete strength and the type of wrapping scheme used, respectively. Expressions for these modification factors are given in Equation (3.23) and (3.24).

$$k_1 = \left[\frac{f'_c}{27}\right]^{2/3}$$ Equation (3.23)SI, (ACI 440.2R-02, Eq. 10-9)

$$k_2 = \begin{cases} \dfrac{d_f - L_e}{d_f} & \text{for } U - \text{wraps} \\ \dfrac{d_f - 2L_e}{d_f} & \text{for two sides bonded} \end{cases}$$ Equation (3.24)SI

(ACI 440.2R-02, Eq. 10-10)

3.3.2.3: Reinforcement limits

The total shear reinforcement should be taken as the sum of the contribution of the FRP shear reinforcement and the steel shear reinforcement, Equation (3.25).

$$V_s + V_f \leq 0.66 \sqrt{f'_c}\ b_w d \qquad \text{Equation (3.25)SI}$$

b_w is the web width or diameter of circular section,(mm).

3.3.2.4: Recommendations

- FRP effectiveness increases as the fibers direction becomes closer to the perpendicular of the diagonal shear crack.
- For external FRP reinforcement in the form of discrete strips, the center-to-center spacing between the strips should not exceed the sum of *d*/4 plus the width of the strip. See Figure (3.8), (ACI 440.2R-02, 10.1).

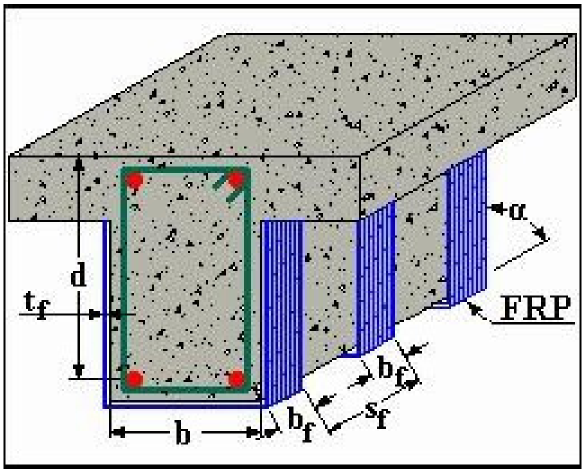

Figure (3.8) - Shear strengthening with discrete strips.

(Sika CarboDur, 2003)

- Proper anchorage means a fully wrapped or a system that is properly anchored in the compression zone, as shown in Figure (3.9) and Figure (3.10). Where practically possible, it is recommended to use for anchoring the whole height of the compression zone, to guarantee an anchoring as good as possible. FRP strips at the sides of the beam only are not recommended as in this case there is a lack of anchorage in both the compression and tension zone.

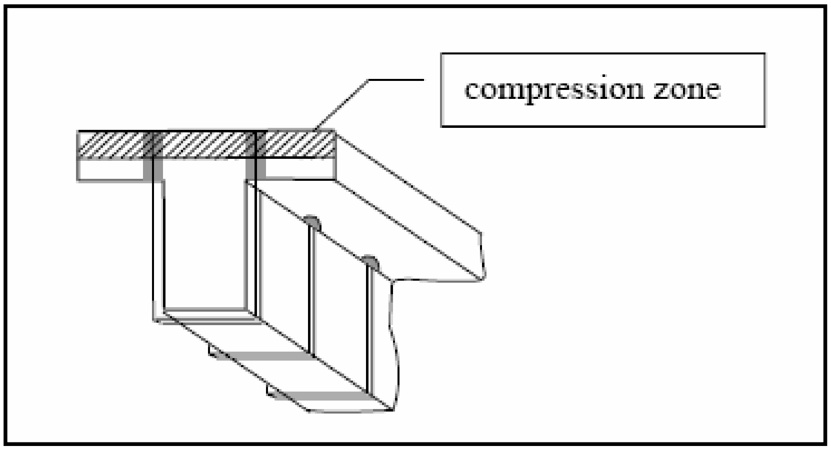

Figure (3.9) - Anchorage in the compression zone.

(FIB Bulletin No 14, 2001)

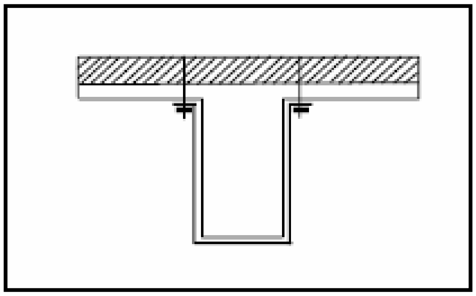

Figure (3.10) - Alternative anchorage in the compression zone.

(FIB Bulletin No 14, 2001)

3.4: Strengthening of RC columns using FRP wrapping

It is often necessary to strengthen RC columns in a building either due to defects in the columns themselves, having to support higher loads than those foreseen in the initial design of the structure, or as the result of accidental damage.

Three principal methods are available for column strengthening:

1. Concrete jacketing.
2. Steel jacketing.
3. Composite jacketing (wrapping of composite around RC column).

Although the use of steel and concrete jacketing is widespread and highly effective, there has been little research into RC columns strengthening by this technique.

Figure (3.11) compares the percentage of articles published in scientific journals relating to the most commonly used strengthening techniques.

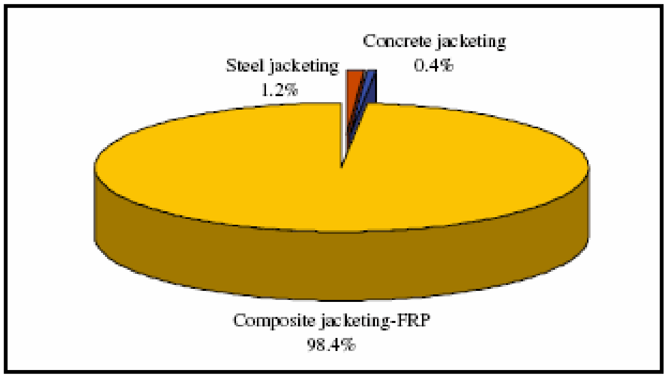

Figure (3.11) - Percentage of research papers related to each strengthening technique of RC columns (Jose M. Adam, 2008)

Compared to conventional steel or concrete jackets, FRP jackets are easier to apply, they do not change the appearance of the structure, and most importantly they do not increase the stiffness of the member.

Also the lightweight, high strength-to-weight ratio, corrosion resistance, and high efficiency of construction are among the advantages of FRP composite materials which make it a good alternative to traditional materials to strengthen existing structures.

The main objectives of confinement are:

(a) To enhance concrete strength and deformation permission,

(b) To provide lateral support to the longitudinal reinforcement

(c) To prevent the concrete cover from spalling.

The tensile strength of concrete is much less in comparison to its compressive strength, the failure of compression members is often happen due to the tensile stress that develops in the perpendicular direction of the compressive load.

If a RC column is confined using a wrapping of CFRP sheets, the failure due to tensile cracks can be prevented. See Figure (3.12).

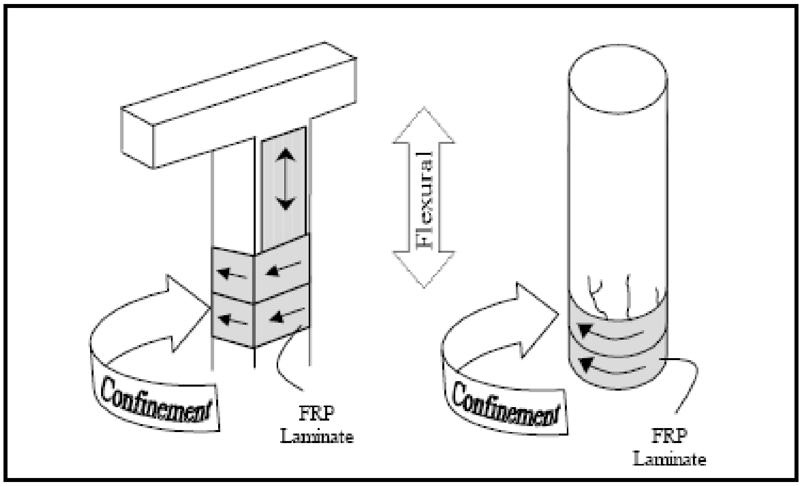

Figure (3.12) – Rehabilitation of columns with FRP composites

Application of CFRP sheet is quite easy, requiring no specialized tools, thus technique is of practical interest.

In addition, the wrapping materials remain exposed to environmental attack, so steel is unsuitable for this purpose.

Wrapping the columns with FRP sheets increase the strength and flexural ductility of rectangular and circular columns reinforced with steel reinforcement and subjected to monotonic, cyclic flexural loading, and it helps in mitigating the deficiency associated with inadequate transverse reinforcement and lap-splice length, and greatly increase their shear resistance.

Also enhancing the confinement through FRP wrapping turns both the shear and bond failures into a ductile one, prevents buckling of longitudinal bars, and increase the longitudinal bar strains corresponding to the ultimate load.

Furthermore, the efficiency in improving the structural performance of the columns due to FRP wrapping is highly affected by column's shape (e.g. circular or rectangular) and the degree of confinement, which is influenced by the type, and thickness of composites (or number of FRP layers) and the intensity of the applied axial load.

FRP composite wrapping provide effective lateral confinement for circular columns, but its effectiveness is much reduced for rectangular columns. Stress concentration at the 90-degree corners and lack of confinement at the flat sides of square/rectangular sections result in reduced confinement effectiveness.

To increase the effectiveness of FRP confinement for rectangular column modifies the column section into an elliptical section, oval, or circular section.

Typical stress-strain curve of cylindrical specimens wrapped with CFRP of varying number of CFRP layers (or jacket thickness) is presented in Figure (3.13).

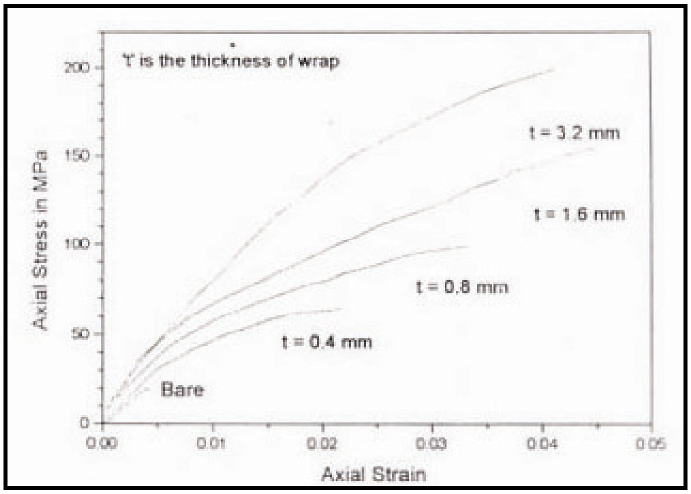

Figure (3.13) – Axial strain circular columns with different levels of confinement (Mukherjee, 2001)

It may be noted that with one layer of CFRP wrap the ultimate strength of the specimens increased by a factor of 2.5.

The ultimate strength went on to increase up to 8 times when 8 layers of the wrap were used.

3.4.1: Analysis of confinement RC columns

Wrapping FRP systems completely around certain types of compression members will confine those members, leading to increases in axial compression strengths. Bonding FRP systems to concrete members can also increase the axial tension strength of the member. Confinement is also used to enhance the ductility of members subjected to combine axial and bending forces. It is accomplished by orienting the fibers transverse to the longitudinal axis of the member.

The axial compressive strength of a nonslender, normal weight concrete member confined with an FRP jacket may be calculated using the confined concrete strength.

- For nonprestressed members with existing steel spiral reinforcement:

$$\Phi P_n = 0.85\Phi\,(0.85\psi_f f'_{cc}(A_g - A_{st}) + f_y A_{st}) \qquad \text{Equation (3.26a)}$$

(ACI 440.2R-02, Eq.11-1(a))

- For nonprestressed members with existing steel-tie reinforcement:

$$\Phi P_n = 0.80\Phi\,(0.85\psi_f f'_{cc}(A_g - A_{st}) + f_y A_{st}) \qquad \text{Equation (3.26b)}$$

(ACI 440.2R-02, Eq.11-1(b))

Where:

P_n = nominal axial load strength at given eccentricity, (N)

f'_{cc} = apparent compressive strength of confined concrete, (MPa)

A_g = gross area of section, (mm^2)

A_{st} = total area of longitudinal reinforcement, (mm^2)

It is recommended to take the additional reduction factor, $\psi_f = 0.95$.

3.4.2: Ductility

Increased ductility of a section results from the ability to develop greater compressive strains in the concrete before compressive failure (Seible et al. 1997). The FRP jacket can also serve to delay buckling of longitudinal steel reinforcement in compression, and to clamp lap splices of longitudinal steel reinforcement.

3.4.2.1: Circular members

The maximum usable compressive strain for an FRP-confined circular member can be found from Equation (3.27).

$$\varepsilon`_{cc} = \frac{1.71(5f`_{cc} - 4f`_{c})}{E_c}$$ Equation (3.27), (ACI 440.2R-02, Eq.11-6)

The confined concrete strength can be computed from Equation (3.28) using a confining pressure given in Equation (3.29) that is the results of the maximum effective strain that can be achieved in the FRP jacket.

$$f'_{cc} = f'_{c}\left[2.25\sqrt{1+7.9\frac{f_l}{f'_c}} - 2\frac{f_l}{f'_c} - 1.25\right]$$ Equation (3.28)

(ACI 440.2R-02, Eq.11-2)

$$f_l = \frac{k_a \rho_f \varepsilon_{fe} E_f}{2}$$ Equation (3.29), (ACI 440.2R-02, Eq.11-3)

The confining pressure provided by an FRP jacket installed around a circular member with a diameter *h* can be found using the reinforcement ratio given in Equation (3.30).

$$\rho_f = \frac{4nt_f}{h}$$ Equation (3.30)

The efficiency factor κ_a for circular sections can be taken as equal to 1.0.

If the member is subjected to combined compression and shear, the effective strain in the FRP jacket should be limited based on the criteria given in Equation (3.31).

$$\varepsilon_{fe} = 0.004 \leq 0.75\varepsilon_{fu} \qquad \text{Equation (3.31)}$$

3.4.2.2: Noncircular members

Confining square and rectangular sections, while not effective in increasing axial strength, is effective in improving the ductility of compression members.

The maximum usable compressive strain for an FRP-confined square or rectangular member can be found from Equation (3.27) with f'_{cc} from Equation (3.28), (3.29) and (3.31).

The reinforcement ratio for rectangular sections can be found from Equation (3.32).

$$\rho_f = \frac{2nt_f(b+h)}{bh} \qquad \text{Equation (3.32), (ACI 440.2R-02, Eq.11-7)}$$

The efficiency factor for square and rectangular sections should be determined from Equation (3.33) that based on geometry, aspect ratio, and the configuration of steel reinforcement.

$$k_a = 1 - \frac{(b-2r)^2 + (h-2r)^2}{3bh(1-\rho_g)} \qquad \text{Equation (3.33), (ACI 440.2R-02, Eq.11-8)}$$

Where:

r is the radius of the edges of a square or rectangular section confined with FRP, in. (mm), and ρ_g ratio of the area of longitudinal steel reinforcement to the cross-sectional area of a compression member.

3.4.3: Recommendations

- Sharp corners on rectangular columns should be rounded to minimum radius of 10 mm in order to avoid stress concentrations in the fabric.

- Both horizontally and spirally running fibers can achieve effective confinement.

- The maximum number of layers = 20-25 (FIB Bulletin No 14).

- In case of eccentric compressive loads of high magnitude, longitudinally directed fibers can be also applied. These fibers are to be anchored with transverse fibers at the top as well as the bottom of the member.

Chapter 4

Case Studies

4.1: Introduction

FRP systems have emerged as an alternative to traditional materials and techniques for strengthening of existing concrete structures.

The number of projects using FRP systems has increased dramatically from a few 10 years ago to several thousand today.

This chapter explains different case studies of strengthening RC structures using CFRP.

4.2: Examples of Different Strengthening Applications Using CFRP

Sika Ltd, Hertfordshire, UK, has developed over a number of years a procedure for strengthening, using a system approach comprising an advanced epoxy structural adhesive and a range of carbon fiber /epoxy polymer laminates. This combined system is known as the Sika CarboDur system.

Sika has been involved with over 200 CarboDur projects worldwide, many of which would not have been possible using traditional strengthening techniques.
The Sika CarboDur materials properties are given in Appendix.

For the case studies included in this section, the application of the CarboDur system has generally followed the procedure outlined below:

- The substrate surfaces were prepared to remove all contaminants.

  -Concrete: fine and coarse aggregate exposed and fine gripping texture achieved by blast cleaning,

  -Timber: surfaces planed or grit blasted,

  -Masonry: fine gripping texture achieved by blast cleaning.

- Substrate surfaces vacuum cleaned to remove dust.
- CarboDur carbon fiber laminates cut to length at factory or on site.
- Preprepared laminate bonding surface cleaned.
- Sikadur 30 structural adhesive applied to laminate and substrate.
- Carbon fiber laminate pressed into position on substrate by hand.
- Carbon fiber laminate bedded into adhesive using hard hand rubber roller to extrude excess adhesive and produce void free bondline.
- Surplus adhesive removed from substrate and laminate.

4.2.1: King College Hospital, London (Shaw, 1997):

- **The problem**

In June 1996, this was the first project in the UK to use a carbon fiber strengthening system for external poststrengthening. The project involved the overall refurbishment and extension of the Normanby College suite for the new Joint Education Center. As part of the refurbishment brief, an extra floor was required for accommodation. This was achieved by converting the roof slab to a floor slab thereby increasing the live load capacity to $3.0kNm^{-2}$. The construction of the existing building consisted of a reinforced concrete frame with cast in situ troughed floors. The upper storey extension was designed as a lightweight steel frame structure, however calculations on the existing roof slab established that in its present form, the slab could not sustain the additional live load.

To resolve the problem, three options were considered:

- Demolish the existing roof and construct a new floor.
- Provide a secondary steel frame to support existing slab or new separate floor.
- Externally poststrengthen the roof slab by external plate bonding.

The first option was considered time consuming and expensive. The second option was not feasible owing to the long spans. Therefore the option of poststrengthening was preferred, based on its cost effectiveness and speed of application. The initial design showed that steel plates 75mm wide and 6.0mm thick were required to provide the additional reinforcement.

- **The solution**

The original roof slab was formed from 400 mm deep tapered ribs 80mm wide at the bottom located at 600 mm centers spanning 11.0 m. Because the plate width to thickness ratio was less than 50 and it would be necessary to slice the plates for handling purposes within a restricted working area, the use of the CarboDur strengthening system was specified by the structural engineers, Lawrence Hewitt Partnership. To achieve the desired strengthening requirements, the laminates used were 50 mm wide, 1.2 mm thick with an E-modulus of 155 kN/mm^2.

In competitive tender, Concrete Repairs Ltd (CRL) secured the subcontract to supply and install the CarboDur system for the main contractor. CRL prepared the floor rib soffits by needle gunning and vacuuming; this method of preparation was checked on site by performing pull-off tests using a 'limpet'. The pull-off values achieved were in the region of 3.0 Nmm^{-2} with a failure in the concrete. The 50mm wide CarboDur laminates arrived on site in 250 m long rolls, preboxed for protection. The individual laminates were then cut on site using a guillotine, cleaned to remove surface contaminants and coated with the epoxy adhesive. At the same time the concrete bond surface was coated with adhesive Figure (4.1). The laminates were then offered up to the beams and rollered into position Figure (4.2) to ensure good adhesive contact and to eliminate voids. The next day antipeel bolts were installed at the ends of the laminate by drilling through the CarboDur laminate into the concrete and using chemical anchors. The strengthening work was completed within the four weeks programme at a cost of around £60 000 using a total of 1100 m of CarboDur laminates.

Figure (4.1) – Application of adhesive to CarboDur laminate

(Shaw, 1997)

Figure (4.2) – Positioning an 11.0 m laminate to underside of rib

(Shaw, 1997)

4.2.2: Strengthening of masonry walls in an office building, Zurich, Switzerland (Shaw, 1997):

- **The problem**

Two existing six storey apartment houses built in the 1930s were converted into a large office building. Consequently, a complete redesign of the structural load bearing system was necessary. Furthermore, many items of the present building codes had to be taken into consideration because they differed considerably from those at the time of the original construction, in particular with respect to the earthquake and wind load standards. Amongst many other alterations, old wooden floors and all of the inner load bearing walls and one entire façade had to be removed and replaced by reinforced concrete slabs and columns. Only parts of the interior unreinforced masonry (URM) fire wall remained in place. These alterations changed both the stiffness and the load bearing capacity of the whole structure.

In the longitudinal direction, two new concrete walls were calculated to resist the earthquake loads. But for the critical transversal direction, only two internal concrete load bearing walls of the staircase and parts of the URM fire wall were available to transmit the horizontal loads down into the foundation. The interior URM fire wall had, therefore, to be strengthened considerably.

- **The solution**

Three strengthening options were considered:

a. Demolishing and reconstructing a new fire wall,
b. Strengthening the existing wall by applying a reinforced shotcrete skin,
c. Strengthening the wall using the CarboDur carbon fibre strengthening system.

The carbon fiber option was chosen based on the following advantages:

a. Creates minimum interference with other construction work.
b. No dimensional changes in wall thickness.
c. Cost effective solution to resist earthquake loads.
d. Maintenance free system.
e. No special tools or heavy equipment required on site
f. Short duration time on site resulting in a reduction in programme time.

- **Strengthening details**

The CarboDur strengthening system was utilised on one side of the wall for three storeys using 100 mm wide strips, 1.2 mm in thickness laid diagonally across the wall Figure (4.3). The practical advantage of using the CarboDur laminates was lightness of the material. One long length of laminate was used, therefore, eliminating the need for lap joints. In addition the crossover detail was very simple.

The existing render was removed from the wall and the surface was grit blasted in the areas to be bonded to achieve an open textured profile. Local protuberances were removed mechanically. Prior to the application of the adhesive, the surfaces were vacuum cleaned to remove dust. The CarboDur laminates were anchored in the adjacent new reinforced concrete column. In order to achieve optimal adhesion between the carbon fiber laminate and the grouting mortar, the anchorage zone of the laminates was slightly curved and provided with a special bonding bridge. In addition to this, steel ties were placed across the laminates and fixed into the concrete with an epoxy resin. All anchorage zones were then grouted with a compatible epoxy mortar.

Figure (4.3) – Application of the CarboDur plates to the masonry wall.

(Shaw, 1997)

To ensure the highest quality of work, specialist contractors were used together with the following on site quality assurance programme:

a. Continuous visual over-all inspection by a supervising engineer,
b. Adhesion tests on the prepared surfaces,
c. Dewpoint control of the substrates prior to bonding and grouting,
d. Sampling of all epoxy batches, as used and mixed on the site, to measure compressive strength and flexural modulus,
e. Recording of all delivery documents, including production numbers and expiry dates.

With this strengthening, the lateral resistance and the ductility of the interior URM fire wall could be increased many times over at reasonable costs. It took no more than four days to carry out all the strengthening work. This was the first ever project where carbon fiber laminates had been used to strengthen a masonry wall. See Figure (4.4).

Figure (4.4) - URM fire wall strengthened with CarboDur carbon fiber strengthening system (Shaw, 1997)

4.2.3: Strengthening of floors in an old apartment house, Budapest, Hungary (Shaw, 1997):

- **The problem**

The owner changed the use of the building from an apartment house to an office building. The existing floor slab construction was not sufficient for the new loading requirements and had to be strengthened.

- **The solution**

The floor slab soffit was strengthened with 50mm wide, 1.2mm thick CarboDur laminates. The total length was 170 m. See Figure (4.5).

Figure (4.5) – Application of CarboDur laminates

(Shaw, 1997)

4.2.4: Strengthening of a hospital in Switzerland:

(Sika CarboDur, 2003)

- **The problem**

Insufficient flexural and shear reinforcement of new columns. The columns were built because of openings cut in an existing concrete wall.

- **The solution**

After cutting the openings for new doors into the wall steel plates were bonded for flexural reinforcement. Bands of SikaWrap Hex 230C were added as shear reinforcement for the new structure.

4.2.5: Strengthening of school building in Switzerland, 2001 (Sika CarboDur, 2003)

- **The problem**

To add air condition pipes to some class rooms of a school building, holes with diameters of 32 and 37 cm had to be drilled into existing RC girders. This cut the longitudinal rebars that were bent up at the ends as shear reinforcement.

- **The solution**

To avoid a decrease in the load bearing capacity L-shaped CFRP plates and straight CFRP plates had to be bonded to the structure before cutting the holes. The anchorage holes for the plate ends were made using an improved cutting device based on the corner saw with diamond tipped saw chain. After full cure of the structural adhesive the two round openings were then cut out. See Figure (4.6).

Figure (4.6) – Application of CFRP plates

Chapter 5

Shear Strengthening Analysis

5.1: Introduction

This chapter is provided to illustrate the action of shear strengthening using the ACI 440 equations of shear strengthening design on different reinforced concrete beams. The analysis was carried out using previous experimental results (Abdel-Jaber, 2003). A comparison between theoretical and experimental results will be discussed.

An excel sheet is provided based on the analysis examples in the ACI 440 manual.

Also, Sika CarboDur FRP analysis software will be discussed.

5.2: Comparison between experimental results and ACI equations

5.2.1: Experimental results (Abdel-Jaber, 2003)

Four different configurations of CFRP shear reinforcement were used to compare results with ACI equations for the same beams.

The four reinforced concrete beams are 150 mm wide by 200 mm deep and 1800 mm long. The material used for strengthening is Sika CarboDur S1012, a pultruded unidirectional CFRP. Ultimate tensile strength equal to 3100 N/mm^2 and modulus of elasticity equal to 155 kN/mm^2.

Beam details and test results were summarized in Table 4.1

Table (5.1) – Beam details and test results (Abdel-Jaber, 2003)

Beam	Reinforcement details	Shear strengthening	t_f (mm)	w_f (mm)	s_f (mm)	f_{cu} (N/mm)	V_u (kN) experimental
Control beam	3 $\phi 16$ No stirrups	No CFRP				63.85	90
B 6	3 $\phi 16$ No stirrups	Multiple vertical strips, $\beta = 90^o$	1.2	20	50	59.03	111
B 8	3 $\phi 16$ No stirrups	Four horizontal strips, $\beta = 0^o$	1.2	20	60	50.93	107
B 10	3 $\phi 16$ No stirrups	Diagonal strips, $\beta = 73^o$	1.2	20	60	65.23	130
B 12	3 $\phi 16$ No stirrups	Several vertical plates, $\beta = 90^o$	1.2	100	100	59.31	195

5.2.2: Results using ACI equations

The ultimate shear strength of a concrete member strengthened with FRP plates can be calculated using ACI 440 equations as follows:

For beam 10, using information given in Table (5.1), we determine:

$f_{fu} = \varepsilon_{fu} E_f$

$\varepsilon_{fu} = 0.02$

$$L_e = \frac{23{,}300}{\left(nt_f E_f\right)^{0.58}} = 20.5 \text{ mm}$$

$$k_1 = \left[\frac{f'_c}{27}\right]^{2/3} = 1.8$$

$$k_2 = \frac{d_f - 2l_e}{d_f} = 0.8$$

$$K_v = \frac{k_1 k_2 L_e}{11{,}900\,\varepsilon_{fu}} = 0.123 \le 0.75$$

$$\varepsilon_{fe} = \kappa_v \varepsilon_{fu} = 0.0025 \le 0.004$$

$$f_{fe} = \varepsilon_{fe} E_f = 381.7 \text{ N/mm}^2$$

$$A_{fv} = 2nt_f w_f = 48 \text{ mm}^2$$

$$V_f = \frac{A_{fv} f_{fe}\left(\sin a + \cos a)d_f\right)}{s_f} = 76.26 \text{ kN}$$

$$V_c = 33.3 \text{ kN}$$

$$\phi V_n = \phi\,(V_c + V_s + \psi_f V_f) = 88.0 \text{ kN}$$

Same calculations were carried out for the remaining beams using the same procedure.

Theoretical and experimental results of the four beams are summarized in Table 5.2

Table (5.2) – Theoretical and experimental results

Beam	V_u(kN) Theoretical	V_u(kN) Experimental	$\frac{Vu(\text{exp.})}{Vu(theo.)} \times 100$ %
B 6	81	111	137 %
B 8	66	107	162 %
B 10	88	130	147 %
B 12	160	195	122 %

5.2.3: General discussion

It was noted from the results that the experimental values are higher than theoretical values. This is logical due to the fact that theoretical values are always lower than experimental to provide safety.

Comparison between experimental and theoretical values was drawn in the figure below.

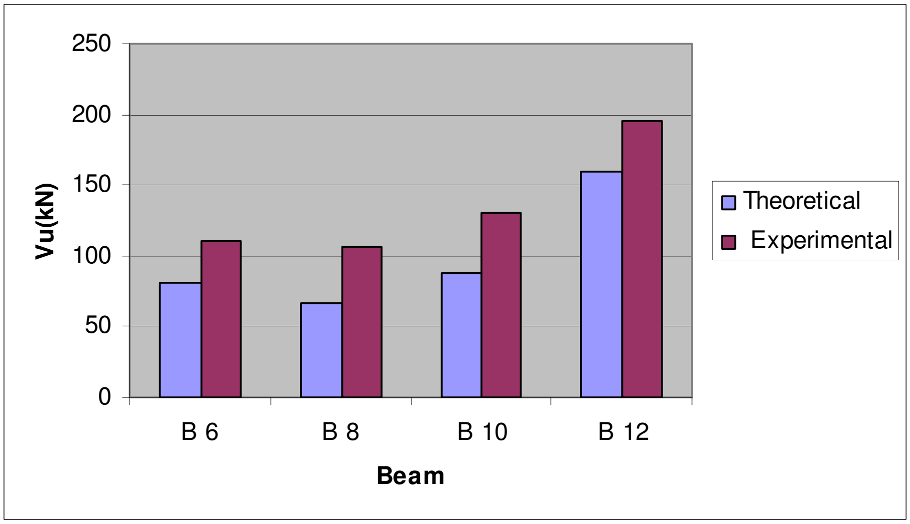

Figure (5.1) – Comparison of experimental results to the results using ACI 440 equations

The ACI equation for shear strengthening can be applicable for the following cases:

- Vertical discrete strip, and;
- Diagonal discrete strip.

This equation can't be applied for other different cases, such as:

- Vertical one sheet or strip (can't find the value of s_f).
- Horizontal one sheet or strip (can't find the value of s_f and d_f).
- Multiple horizontal strips (can't find the value of s_f and d_f).
- Multiple continuous vertical or horizontal strips (can't find the value of s_f and d_f).
- Completely wraps (can't find the value of K_2).

Due to this fact, it is recommended that the ACI 440 code should consider more cases to produce new equations that can be applied to solve other configuration for shear beam strengthening.

5.3: Excel model

An excel sheet was produced to combine all ACI 440 equations for shear beam strengthening in one model to allow the user to deal with the calculations in an easier way.

The same beam results discussed before (Abdel-Jaber, 2003) was reinvestigated using the Excel spread sheet model.

These calculations are provided in Appendix attached to the end of this dissertation.

5.4: Sika software

The aim of this software is to assist the user in calculating the FRP dimensions required to provide:

- Flexural strengthening
- Shear strengthening
- Confinement

The equations used in this program are given in the FIB Bulletin No. 14, (July 2001), "Design and use of Externally Bonded FRP Reinforcement for RC Structures", which approved by International Federation for Structural Concrete.

This software was user friendly, simple and reliable design tool for the selection of FRP dimensions to provide flexural strengthening, shear strengthening or confinement of reinforced concrete sections. Essentially, the software is divided into two procedures. The first is related to definitions of input data, solution options and print options, whereas the second involves presentation of results, selection of appropriate number of FRP strips or layers and printing of information.

See example for shear strengthening using Sika software in Appendix.

The following results were concluded after using Sika software:

- The FRP materials used for strengthening are one selection from Sika CarboDur system.
- Can't give results about failure of the members.
- In the case of shear strengthening the design depends on the value of additional shear, which mean there is no check if a strengthened section was applicable for sustaining the required shear strength or not.

Chapter 6

Summary, Conclusions and Recommendations

6.1: Summary

Rehabilitation of aged structures has long been considered as one of the most important problems in the structural engineering community.

Moreover, the general public's awareness of and concern for the safety of structure have been continuously growing. Many structures have been rehabilitated to extend their lifespan or restore their original strength. There are two possible solutions to their rehabilitation:

Repair: is the rehabilitation of a damaged structure or a structural component with the aim of restoring the original capacity of the damaged structure.

Strengthening: is the process of increasing the existing capacity of a non-damaged structure or structural component to higher level.

FRP systems have emerged as an alternative to traditional materials and techniques for the rehabilitation and strengthening of existing concrete structures.

In addition to their high strength to weight ratio, corrosion resistance and high fatigue strength, FRP can be easily bonded to reinforced concrete members.

In Chapter 1, a brief introduction to rehabilitation of structures using composite materials (FRP) has been presented.

The old and new rehabilitation methods have been explained in Chapter 2, including the materials used in each method, also the choice of rehabilitation models are presented in this chapter.

Strengthening technique with FRP for main RC structural elements (beams and columns) using ACI 440 equations has been explained in Chapter 3.

Some practical examples of different strengthening applications using FRP are presented in Chapter 4.

In Chapter 5, a comparison between the results of shear strengthening beams using ACI 440 equations and the experimental results (Abdel-Jaber, 2003) for the same beams has been discussed.

Also, an Excel spread sheet model using ACI 440 equations for shear strengthening was produced to use for shear strengthening calculations.

The advantages and disadvantages of using Sika CarboDur FRP analysis software for strengthening system have been discussed.

6.2: Conclusions

1. The ultimate shear strength values calculated using ACI 440 equations for different configurations of CFRP have lower values than experimental results for the same beams.

2. The ACI 440 equations for shear strengthening can be used for the following cases:

   - Vertical discrete strip.
   - Diagonal discrete strip.

3. The ACI 440 equations for shear strengthening can't be used for other different cases, such as the following:
   - Vertical one sheet or strip (can't find the value of s_f).
   - Horizontal one sheet or strip (can't find the value of s_f and d_f).
   - Multiple horizontal strips (can't find the value of s_f and d_f).
   - Multiple continuous vertical or horizontal strips (can't find the value of s_f and d_f).
   - Completely wraps (can't find the value of K_2).

4. An Excel spread sheet was produced using ACI 440 shear strengthening equations can be used for shear strengthening of RC beams using different configurations of externally bonded CFRP plates.

5. Selection of FRP materials and its properties to provide flexural strengthening, shear strengthening and confinement of RC sections can be done using Sika CarboDur FRP analysis software. It is a simple and easy software, but this program can't give results about failure of the members. Also, in the case of shear strengthening this software has no check if a strengthened section was applicable for sustaining the required shear strength or not.

6.3: Recommendations

The following recommendations are suggested:

1. ACI 440 code needs to consider new equations to cover all cases for shear strengthening.
2. In cases of strengthening over supports of continuous beams or slabs, FRP should be anchored in the compression zone, (FIB Bulletin No 14).
3. Many bond-related failures can be avoided by following these general guidelines for detailing FRP sheets or laminates, (ACI 440.2R-02):
   - Do not turn inside corners;
   - Provide a minimum 13 mm radius when the sheet is wrapped around outside corners.
4. Future research is needed to provide information in areas that are still unclear or are in need of additional evidence to validate performance, such as:
   - Methods of fireproofing FRP strengthening systems;
   - Behavior of FRP strengthened members under elevated temperatures;
   - Behavior of FRP strengthened members under cold temperatures;
   - Behavior of members strengthened with FRP systems oriented in the direction of the applied axial load;
   - Effects of concrete strength on behavior of FRP strengthened members;
   - Maximum crack width and deflection prediction and control of concrete reinforced with FRP systems.
   - Different configurations of FRP shear strengthening members.

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APPENDIX

Sika CarboDur composite strengthening systems

Mainly Flexural Strengthening Products (Plates)				
Product name	Elastic modulus E_f (kN/mm^2)	Ultimate tensile strain ε_{fu}	Width b_f (mm)	Thickness t_f (mm)
Carbodur S512	165	0.0170	50	1.2
Carbodur S612	165	0.0170	60	1.2
Carbodur S812	165	0.0170	80	1.2
Carbodur S1012	165	0.0170	100	1.2
Carbodur S1212	165	0.0170	120	1.2
Carbodur S1512	165	0.0170	150	1.2
Carbodur S614	165	0.0170	60	1.4
Carbodur S1214	165	0.0170	120	1.4
Carbodur M514	210	0.0135	50	1.4
Carbodur M614	210	0.0135	60	1.4
Carbodur M914	210	0.0135	90	1.4

Mainly Shear Strengthening Products (L-shaped Strips)				
Product name	Elastic modulus E_f (kN/mm^2)	Ultimate tensile strain ε_{fu}	Width b_f (mm)	Thickness t_f (mm)
Carboshear 4/20/50	120	0.019	40	1.4
Carboshear 4/30/70	120	0.019	40	1.4
Carboshear4/50/100	120	0.019	40	1.4

shear strengthening design

Beam : B 6

$f'_c =$	59.03	MPa	$b =$	150	mm
$f_{fu} =$	3100	MPa	$h =$	200	mm
$E_f =$	155000	MPa	$d =$	165	mm
$w_f =$	20	mm	$d_f =$	200	mm
$t_f =$	1.2	mm	$V_c =$	31.7	kN
$s_f =$	50	mm	$V_s =$	0	kN

$\beta^o =$ 0

$\beta(rad)=$ 0

$n =$ 1

configurations of FRP: 1 ,for two sides bonded(1)

,for u-raps(2) ,for completely wraps(3)

Solution

$$\boxed{\varepsilon_{fu} = \frac{f_{fu}}{E_f}}$$

$\varepsilon_{fu} =$ 0.02

$$\boxed{L_e = \frac{23{,}300}{(nt_f E_f)^{0.58}}}$$

$L_e =$ 20.5 mm

$$\boxed{K_1 = \left(\frac{f'_c}{27}\right)^{2/3}}$$

$K_1 =$ 1.7

$$\boxed{K_2 = \begin{cases} \dfrac{d_f - L_e}{d_f} & \text{for U-wraps} \\[2ex] \dfrac{d_f - 2L_e}{d_f} & \text{for two sides bonded} \end{cases}}$$

K_2 = 0.8

$$\boxed{\kappa_v = \frac{k_1 k_2 L_e}{11{,}900\varepsilon_{fu}} \leq 0.75}$$

k_v = 0.12

$$\boxed{\varepsilon_{fe} = \begin{cases} 0.004 \leq 0.75\varepsilon_{fu} & \text{for completely wrapping} \\ \kappa_v \varepsilon_{fu} \leq 0.004 & \text{for u-wraps or bonding two sides} \end{cases}}$$

ε_{fe} = 0.0023

$$\boxed{f_{fe} = \varepsilon_{fe} E_f}$$

f_{fe} = 357.15 MPa

$$\boxed{A_{fv} = 2nt_f w_f}$$

A_{fv} = 48 mm2

$$\boxed{V_f = \frac{A_{fv} f_{fe}(\sin\alpha + \cos\alpha) d_f}{s_f}}$$

V_f = 68572.3 N

V_f = 68.6 K N

$$\boxed{\phi V_n = \phi(V_c + V_s + \psi_f V_f)}$$

ΦV_n = 81 K N

shear strengthening design

Beam : B 8

f'_c =	50.93	*MPa*	b =	150	*mm*
f_{fu} =	3100	*MPa*	h =	200	*mm*
E_f =	155000	*MPa*	d =	165	*mm*
w_f =	20	*mm*	d_f =	200	*mm*
t_f =	1.2	*mm*	V_c =	29.4	*k N*
s_f =	60	*mm*	V_s =	0	*k N*

β^o = 0

$\beta(rad)$= 0

n = 1

configurations of FRP: 1 ,for two sides bonded(1)
,for u-raps(2) ,for completely wraps(3)

Solution

$$\boxed{\varepsilon_{fu} = \frac{f_{fu}}{E_f}}$$

ε_{fu} = 0.02

$$\boxed{L_e = \frac{23{,}300}{(nt_f E_f)^{0.58}}}$$

L_e = 20.5 mm

$$\boxed{K_1 = \left(\frac{f'_c}{27}\right)^{2/3}}$$

K_1 = 1.5

$$\boxed{K_2 = \begin{cases} \dfrac{d_f - L_e}{d_f} & \text{for U-wraps} \\[2ex] \dfrac{d_f - 2L_e}{d_f} & \text{for two sides bonded} \end{cases}}$$

$K_2 =$ 0.8

$$\boxed{\kappa_v = \frac{k_1 k_2 L_e}{11{,}900\varepsilon_{fu}} \leq 0.75}$$

$k_v =$ 0.10

$$\varepsilon_{fe} = \begin{cases} 0.004 \leq 0.75\varepsilon_{fu} & \text{for completely wrapping} \\ \kappa_v \varepsilon_{fu} \leq 0.004 & \text{for u-wraps or bonding two sides} \end{cases}$$

$\varepsilon_{fe} =$ 0.0021

$$\boxed{f_{fe} = \varepsilon_{fe} E_f}$$

$f_{fe} =$ 323.7 MPa

$$\boxed{A_{fv} = 2nt_f w_f}$$

$A_{fv} =$ 48 mm2

$$\boxed{V_f = \frac{A_{fv} f_{fe} (\sin\alpha + \cos\alpha) d_f}{s_f}}$$

$V_f =$ 51788.6 N

$V_f =$ 51.8 K N

$$\boxed{\phi V_n = \phi(V_c + V_s + \psi_f V_f)}$$

$\Phi V_n =$ 66.0 K N

shear strengthening design

Beam : B10

$f'_c =$	65.23	*MPa*	$b =$	150	*mm*
$f_{fu} =$	3100	*MPa*	$h =$	200	*mm*
$E_f =$	155000	*MPa*	$d =$	165	*mm*
$w_f =$	20	*mm*	$d_f =$	200	*mm*
$t_f =$	1.2	*mm*	$V_c =$	33.3	*k N*
$s_f =$	60	*mm*	$V_s =$	0	*k N*
$\beta^o =$	73				
$\beta(rad)=$	1.27409				
$n =$	1				

configurations of FRP: 1 ,for two sides bonded(1) ,for u-raps(2) ,for completely wraps(3)

Solution

$$\boxed{\varepsilon_{fu} = \frac{f_{fu}}{E_f}}$$

$\varepsilon_{fu} =$ 0.02

$$\boxed{L_e = \frac{23{,}300}{(nt_f E_f)^{0.58}}}$$

$L_e =$ 20.5 mm

$$\boxed{K_1 = \left(\frac{f'_c}{27}\right)^{2/3}}$$

$K_{1} =$ 1.8

$$\boxed{K_2 = \begin{cases} \dfrac{d_f - L_e}{d_f} & \text{for U-wraps} \\[2ex] \dfrac{d_f - 2L_e}{d_f} & \text{for two sides bonded} \end{cases}}$$

$K_2 =$ 0.8

$$\boxed{\kappa_v = \frac{k_1 k_2 L_e}{11{,}900\varepsilon_{fu}} \leq 0.75}$$

$k_v =$ 0.12

$$\boxed{\varepsilon_{fe} = \begin{cases} 0.004 \leq 0.75\varepsilon_{fu} \\ \kappa_v \varepsilon_{fu} \leq 0.004 \end{cases}}$$

for completely wrapping

for u-wraps or bonding two sides

$\varepsilon_{fe} =$ 0.0025

$$\boxed{f_{fe} = \varepsilon_{fe} E_f}$$

$f_{fe} =$ 381.7 MPa

$$\boxed{A_{fv} = 2nt_f w_f}$$

$A_{fv} =$ 48 mm2

$$\boxed{V_f = \frac{A_{fv} f_{fe} (\sin\alpha + \cos\alpha) d_f}{s_f}}$$

$V_f =$ 76266.5 N

$V_f =$ 76.3 K N

$$\boxed{\phi V_n = \phi(V_c + V_s + \psi_f V_f)}$$

$\Phi V_n =$ 88.0 K N

shear strengthening design

Beam : B 12

$f'_c =$	59.31	MPa	$b =$	150	mm
$f_{fu} =$	3100	MPa	$h =$	200	mm
$E_f =$	155000	MPa	$d =$	165	mm
$w_f =$	100	mm	$d_f =$	200	mm
$t_f =$	1.2	mm	$V_c =$	31.8	kN
$s_f =$	100	mm	$V_s =$	0	kN
$\beta^o =$	90				
$\beta(rad) =$	1.5708				
$n =$	1				

configurations of FRP: 1 ,for two sides bonded(1)
,for u-raps(2) ,for completely wraps(3)

Solution

$$\boxed{\varepsilon_{fu} = \frac{f_{fu}}{E_f}}$$

$\varepsilon_{fu} =$ 0.02

$$\boxed{L_e = \frac{23{,}300}{(nt_f E_f)^{0.58}}}$$

$L_e =$ 20.5 mm

$$\boxed{K_1 = \left(\frac{f'_c}{27}\right)^{2/3}}$$

$K_1 =$ 1.7

$$\boxed{K_2 = \begin{cases} \dfrac{d_f - L_e}{d_f} & \text{for U-wraps} \\ \dfrac{d_f - 2L_e}{d_f} & \text{for two sides bonded} \end{cases}}$$

$K_2 =$ 0.8

$$\boxed{\kappa_v = \frac{k_1 k_2 L_e}{11{,}900\varepsilon_{fu}} \le 0.75}$$

$k_v =$ 0.12

$$\boxed{\varepsilon_{fe} = \begin{cases} 0.004 \le 0.75\varepsilon_{fu} & \text{for completely wrapping} \\ \kappa_v \varepsilon_{fu} \le 0.004 & \text{for u-wraps or bonding two sides} \end{cases}}$$

$\varepsilon_{fe} =$ 0.0023

$$\boxed{f_{fe} = \varepsilon_{fe} E_f}$$

$f_{fe} =$ 358.3 MPa

$$\boxed{A_{fv} = 2nt_f w_f}$$

$A_{fv} =$ 240 mm2

$$\boxed{V_f = \frac{A_{fv} f_{fe} (\sin\alpha + \cos\alpha) d_f}{s_f}}$$

$V_f =$ 171972.3 N

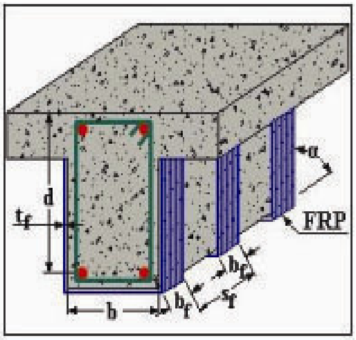
$V_f =$ 171.97 K N

$$\boxed{\phi V_n = \phi(V_c + V_s + \psi_f V_f)}$$

$\Phi V_n =$ 160.0 K N

Strengthening of Reinforced Concrete Structures with Sika® CarboDur® Composite Strengthening Systems

SHEAR STRENGTHENING



Method of Anchorage
Open jacket

Cross Section Geometry
Width b = 0.3 m
Static depth d = 0.46 m
Angle between fibres direction and member axis α = 90 degrees

Concrete
Strength class C 20/25
Characteristic strength f_{ck} = 20 N/mm²
Mean strength f_{cm} = 28.2 N/mm²

Composite Materials
Elastic modulus E_f = 120 kN/mm²
Ultimate tensile strain ε_{fu} = 0.019
Limiting strain $\varepsilon_{f,lim}$ = 0.006
Type of fibres Carbon (CFRP)

Safety Factors
Constant k = 0.8
Debonding safety factor $\gamma_{f,b}$ = 1.3
Limiting strain safety factor $\gamma_{f,l}$ = 1.25
Carbon FRP fracture safety factor $\gamma_{f,f}$ = 1.2

Type of Application
Discrete strips
Width b_f = 0.04 m
Spacing s_f = 0.15 m

Increase of Shear Capacity

Additional shear $V_{fd} = 200$ kN

Results

Required FRP thickness $t_f = 6.53$ mm
Applied FRP thickness $t_f = 7.00$ mm
Additional shear $V_{fd} = 206.18$ kN

Applied FRP

Thickness 1.4 mm
Number of layers required 5

إعادة تأهيل المباني باستخدام مواد مركبة

اعداد

سناء فاروق ريحان

المشرف

الأستاذ الدكتور سميح قاقيش

ملخص

إن حجم المباني الخرسانية القائمة التي تحتاج إلى تطوير، تقوية و/أو إصلاح لكـــي تتحمـــل أوزان أثقل يزداد في مختلف أنحاء العالم.

أصبح استخدام ألياف البوليمر (كربون، زجاج) لتقوية المباني الخرسانية بديلا للطرق التقليدية مثل استخدام صفائح الحديد.

يقدم هذا البحث المواضيع التالية:

- التعرف على الطرق المختلفة لإعادة تأهيل المباني الخرســـانية وخـــصائص المـــواد المستخدمة.
- اعطاء لمحة عامة عن صفات وخصائص ألياف البوليمر المستخدمة لتقوية المبـــاني الخرسانية.
- اعطاء طريقة التصميم الإنشائية المتبعة لتقوية المباني الخرسانية باســـتخدام أليـــاف البوليمر.
- اعطاء أمثلة مختلفة لمباني خرسانية قائمة تم تقويتها باستخدام ألياف البوليمر.
- مقارنة بين قيم السقوط النظرية والتجريبية لجسور خرسانية تم تقويتها بطرق مختلفة باستخدام ألياف البوليمر لتقاوم القص.
- اعداد ورقة اكسل لتصميم تقوية القص في الجسور الخرسانية وذلك باستخدام معادلات الكود الأمريكي.