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***Experimental and Theoretical Investigation of the
Behavior of R.C. Deep Beams with Openings
Strengthened by CFRP Laminates***

A Thesis

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بِسْمِ اللَّهِ الرَّحْمَنِ الرَّحِيمِ

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

صدق الله العظيم

سورة الإسراء / الآية ٨٥

*To
The Candle That Burned To Show Me the
Way*

Mother

To All My Hope and Love

Brothers

To All My Real Friends with Respect

Wissam Nadir

ACKNOWLEDGMENT

IN THE NAME OF ALLAH, THE MOST GRACIOUS, THE MOST MERCIFUL

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Abstract

Web openings in beams are frequently required to provide accessibility such as windows or doors or to accommodate essential service such as ducts and pipes. However, introducing an opening in the web of a reinforced concrete deep beam can cause an abrupt deterioration in the shear capacity due to, stress concentration at the corners of the openings and discontinuity of the load path of compressive strut.

This research presents an experimental and theoretical investigation for the use of carbon fiber reinforced polymer (CFRP) as a strengthening technique to upgrade the reinforced concrete deep beam with web openings.

Eight reinforced concrete deep beams with cross-section of (100x750mm) and total length (1150mm) were tested under four-point loads. Seven specimens had two rectangular openings with cross-section (100x200 mm), one in each center of the shear span, placed symmetrically about the centerline of the beam. The first specimen solid deep beam (without openings) and the second specimen deep beam with web openings were used as references beams. The other beams were strengthened with CFRP laminate. The variables included the orientation, shape and vertical anchorage of CFRP laminate.

The experimental results indicated that the use of CFRP sheet to upgrade the R.C. deep beams with web openings has significant effect on overall behavior such as the ultimate load, crack width and deflection. The percent of increase in the ultimate load capacity was about (100-190) %. In addition, it is found the vertical anchorage of the CFRP system (full wrap, U-wrap and two face wrap) does not appear help the shear strength if it was compared with each other(S-1, S-2 and S-3). While the inclined strengthening model (S-5) shows a stiffer response compared with other strengthened model, which gives 190% increase in ultimate load than control beam and 8.5% more than (S-1). Also,

the use of CFRP as externally strengthening system decrease the crack width, but, it cannot be changed the formation of inclined crack. All the tested beams failed due to splitting the beam into two pieces though the inclined crack.

In the other side, three dimensional finite element analysis was used to investigate the performance of the R.C. member strengthened by CFRP laminate. ANSYS computer program (version 9.0, 2004) was utilized through this study. Full bond was assumed between the CFRP and concrete surface and between the steel reinforcement and concrete. Brick element SOLID65 was used to represent concrete element while LINK8 and SHEEL41 were used to represent steel reinforcement and CFRP sheets respectively. The comparison between the experimental and theoretical results showed reasonable agreement and asserted the validity of the numerical analysis and methodology developed in this study.

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Notation

The major symbols used in this work are listed below. Any other symbol is explained where it appears in the text.

Symbol	Description
a	Shear span
d	Effective depth
E_c	Concrete modulus of elasticity
E_s	Steel modulus of elasticity
f'_c	Cylinder compressive strength of concrete
f_t	Splitting tensile strength of concrete
f_r	Concrete modulus of rupture
f_y	Yield stress of steel
f_u	Ultimate tensile strength of steel
H	Total depth
L	Total length
L_n	Clear span
P_u	Ultimate load
S_x	Stress in x-direction
U	Degree of freedom
U_y	Deflection or vertical displacement
w_c	Air-dry unit weight of concrete
x, y, z	Cartesian coordinates
ε	Strain
ε_o	Strain at ultimate compressive strength
σ	Stress
ν	Poisson's ratio
ν_s	Poisson's ratio of steel
β_c, β_o	Shear transfer coefficient for closed and opened crack.
$[A]$	Displacement gradient matrix
$[B]$	Strain-displacement matrix
$[D]$	Constitutive matrix
$[J]$	Jacobian matrix
$[K]$	Stiffness matrix
$[N]$	Shape function
$[T]$	Transformation matrix

Abbreviations

Abbreviation	Description
ACI	A merican C oncrete I nstitute
ANSYS	A nalysis S ystem program
ASCE	A merican S ociety of C ivil E ngineers
ASTM	A merican S ociety for T esting and M aterial
CFRP	C arbon F iber R einforced P olymer
FEM	F inite E lement M ethod
FRP	F iber R einforced P olymer
Max.	M aximum.
Min.	M inimum
No.	Number
pp.	From p age to p age
p.	P ages.
R.C.	R einforced C oncrete
Ref.	R eference
Vol.	V olume

CHAPTER ONE

Introduction

1.1 General

Reinforced concrete deep beam has always been a subject of considerable interest in the structural engineering. A deep beam in general, has a depth much greater than the normal, in relation to their span, while the thickness in the perpendicular direction is much smaller than either span or depth.

The ACI code defines deep beams as those, which have clear span to overall depth less than four, or the shear span to effective depth less than two, and should be loaded on the top face and supported on the bottom face. So, the compression struts can be developed between the loads and supports points [1]. Reinforced concrete deep beams have been used in tall buildings, offshore structure, foundation walls, transfer girders to support the load from one or more columns and also found in some walls and pile caps[2] as shown in Figure 1.1.

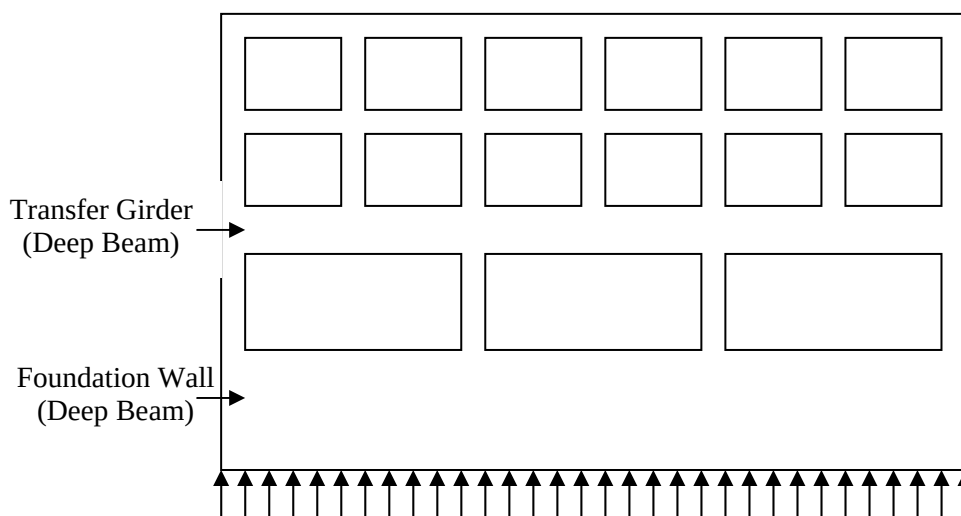


Figure 1.1 Example of deep beam

The presence of web openings in such beam is frequently required to provide accessibility such as doors and windows, or to accommodate essential services such as ventilating, pipes and air conditioning ducts. Enlargement of such openings due to architectural, mechanical requirements or change in the building functions would reduce the element shear capacity [3].

It is well known that what called classical elastic theory of bending is not applicable to problem of deep beams. Such as, the stress pattern is nonlinear as shown in Figure 1.2 and deviates considerably from those derived by Bernoulli and Navier. The deep beam exhibit very complicated stress state by the strut and tie action, the mechanical behavior and design of deep beam are governed by shear, and its load carrying capacity depends on the strength of compressive strut that join the loading and reaction points. The shear capacity of deep beam with web openings is abruptly deteriorated due to stress concentration at the corners of the opening and the discontinuity of the load path of the compressive strut, and its mechanical behavior becomes more complicated [4]. So, alternative materials and techniques should be investigated to restore deteriorated and defected structure.

1.2 Crack Patterns and Modes of Failure

The shear span to effective depth ratio of deep beams determines the failure mode of such beams. Fundamentally, four modes of failure or their combinations can occur [6].

1. Flexural failure
2. Diagonal tension failure
3. Shear-compression failure
4. Splitting failure (Pure shear failure)

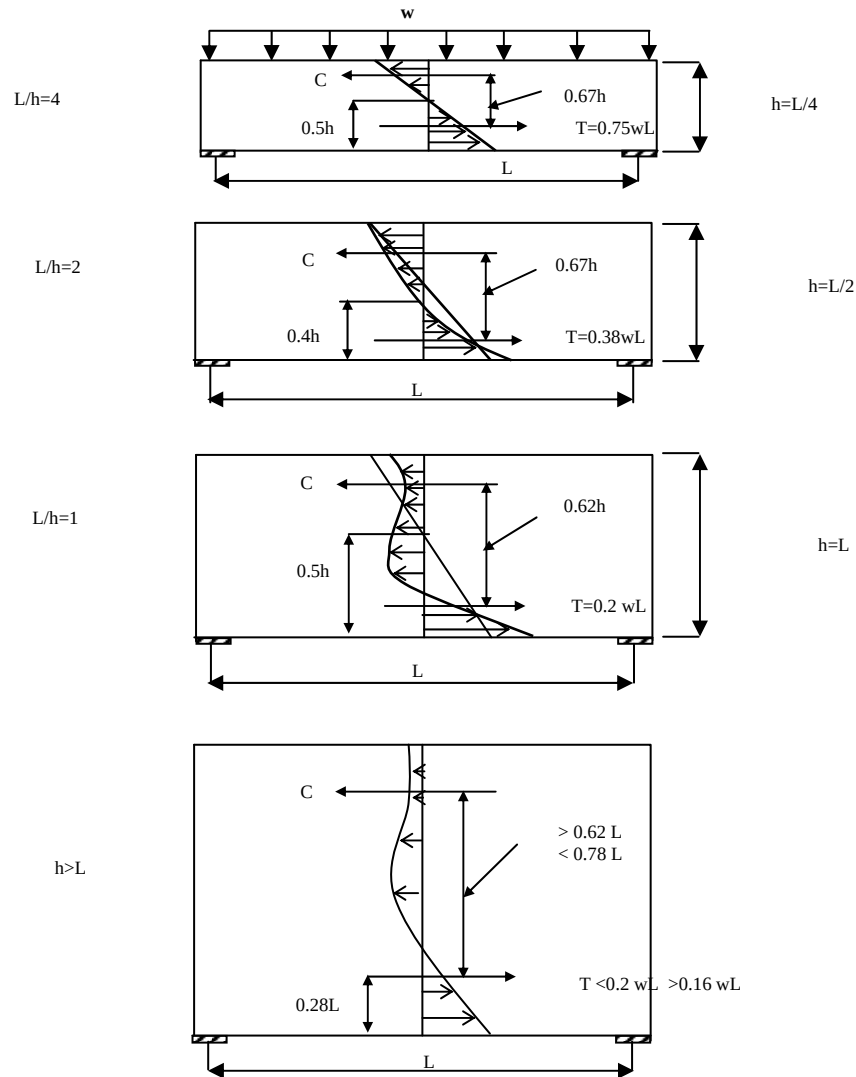


Figure 1.2 Distribution of flexural stresses in homogenous simply supported deep beam [5]

➤ For beams with $2.5 < a/d < 6$

Beams with (a/d) ratio lower than six tend to fail in shear, as shown in Figure 1.3. The flexural crack (a-b) would propagate towards the loading point gradually becoming an inclined crack, which is known as “flexural-shear crack”. With increasing shear force, failure usually occurs in one of two modes. If (a/d) ratio is relatively high, the diagonal crack would rapidly spread to (e), resulting in a collapse by splitting the beam into two pieces. This mode of failure is often called “diagonal-tension

failure”. If (a/d) ratio is relatively low, the diagonal crack tends to stop somewhere at (j), a number of random cracks may develop in the concrete around the longitudinal tension reinforcement. As the shear force is increased, the diagonal crack widens and propagates along the level of the tension reinforcement (Figure 1.3 crack g-h). This failure mode is often called “shear-tension failure” or shear-bond failure. The ultimate load is not much higher than the diagonal cracking load [6].

➤ For beams with $1.0 < a/d < 2.5$

For (a/d) ratio lower than 2.5 and greater than 1.0, the diagonal crack often forms independently. The beam usually remains stable after such cracking. Further increase in the shear force will cause the diagonal crack to penetrate into the concrete compression zone at the loading point, until eventually crushing failure of concrete occurs there. This inclined crack is steeper than that which occurs in flexural – shear case, suddenly develops and proceeds to propagate towards the neutral axis. This failure mode is usually called “shear-compression failure”. For this mode, the ultimate load is more than twice the diagonal cracking load [7].

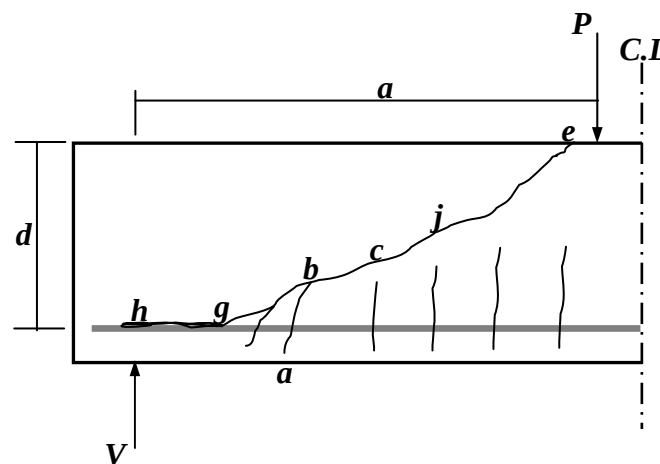


Figure 1.3 Crack patterns for beams with $2.5 < a/d < 6$ [6]

➤ For beams with $a/d < 1.0$

Three distinct types of failure can occur, and these are illustrated in Figure 1.4.

1. Flexure: The beam has a low percentage of bottom steel and fails in tension at mid-span. Such type of failure rarely occurs in practice.
2. Flexural-shear: The beam fails in shear, with main cracks propagating upwards from regions close to the support towards the applied load. In this case, there is a moderate amount of bottom steel and the development of shear-crack is normally preceded by flexural cracks at mid-span.
3. Diagonal splitting: This is the commonest mode of failure. The final shear crack again extends between the applied load and support, but it grows outwards from mid-depth, having been initiated in a more brittle manner than either of other two failure modes [7]

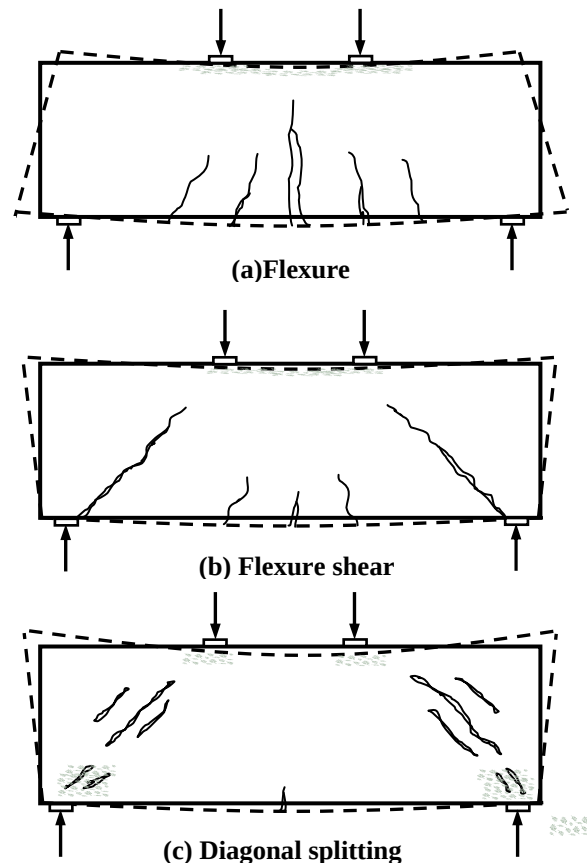


Figure 1.4 Modes of failure of deep beams [7]

1.3 Strengthening Methods

Rehabilitation of deteriorated civil engineering infrastructure such as bridge decks, buildings, beams, girders, parking structures, roads etc., has been a major issue in the last decades. The deterioration of these structures might be due to ageing, poor maintenance, corrosion, mistake in the initial design or construction and accidental situations like earthquakes. The need to upgrade the deteriorated civil engineering infrastructure greatly enhances with the ever increasing demands [8].

Today there are many methods for strengthening reinforced concrete. Figure 1.5 shows six different traditional methods to strengthening the beam in shear [9].

Method (a) is probably the most commonly used. Here, existing concrete on the top of the slab is removed, new stirrups are mounted around the existing cross section and new concrete is cast or sprayed onto the structure. If adhesion between the new and old concrete can be assured, method (a) is good from a technical standpoint. However, the method is both time consuming and in many cases not cost effective.

Method (b) is a little bit easier to carry out than method (a), but there is a risk that the bending reinforcement is cut off during drilling of the holes through the slab.

Case (c) can be used in cases where a limited shear strengthening effect is asked for. A drawback with this method is that the strengthening material is not anchored in the compressive zone. This is also the case for method (d) where steel plates are bonded to the side of the structure.

Case (e) one big hole is drilled through the cross section and a steel bolt or dywidag stay is used. This is quite simple method with the same drawbacks as for method (b).

In case (f) post-tensioned steel straps have been wrapped around the section. However, the straps are quite sensitive to impact loads or vandalism.

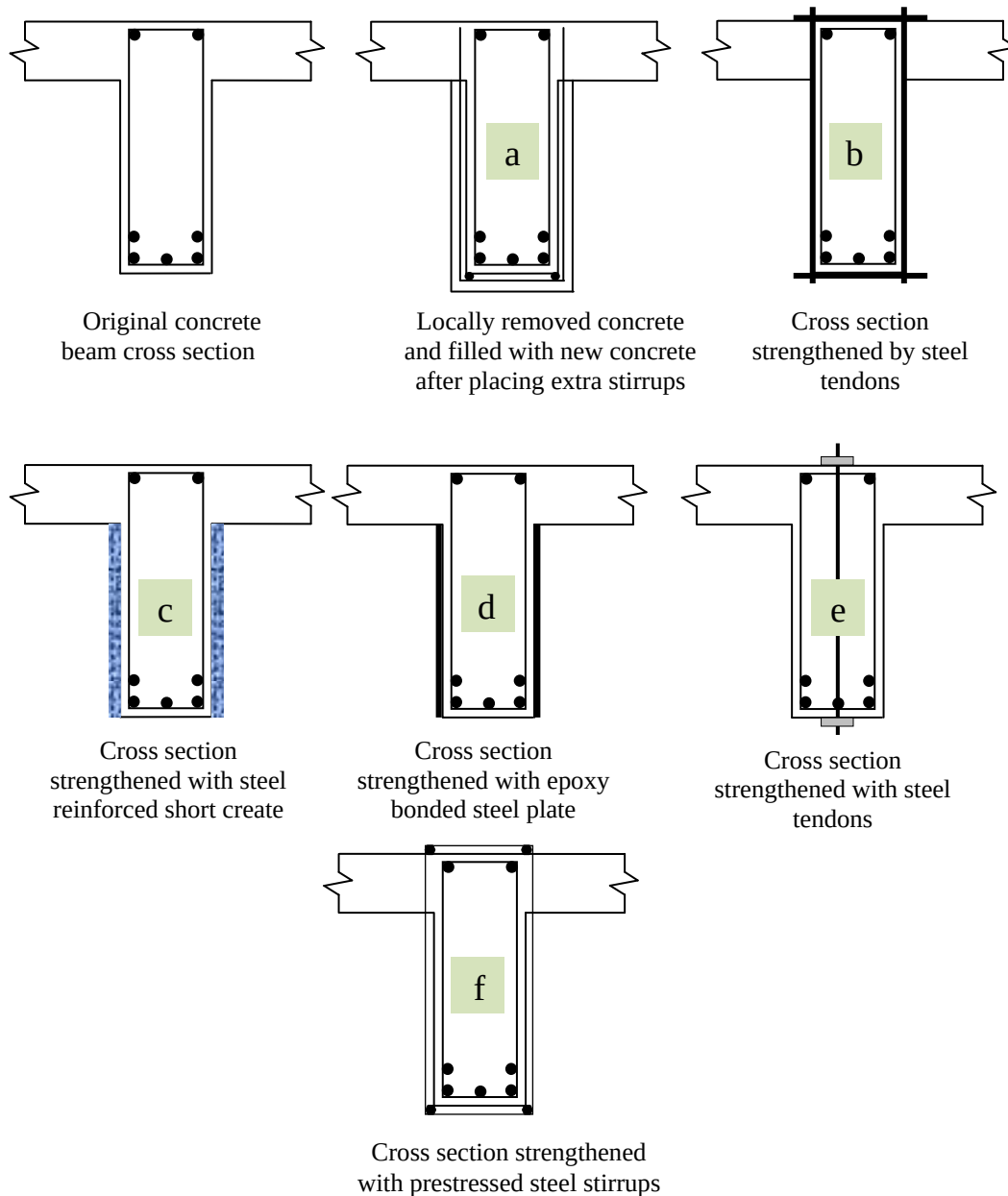


Figure 1.5 strengthening of concrete structure for shear with traditional material [9]

1.4 Fiber Reinforced Polymer Strengthening System.

Reduce cost, coupled with labor savings inherent with its low weight and high strength make FRP an attractive alternative to steel plate for post strengthening. Steel bonding has been a tool for concrete strengthening industry for the past twenty years. However, the high cost of labor to set the usually heavy plates, the difficulties with splicing the plates, and the concern about corrosion of steel plates have limited the use of this techniques [10]. Because of high stiffness to weight ratio, relatively unlimited material length availability, immunity to corrosion and perhaps the biggest advantage of FRP is tailorability, make the use of FRP presents an evaluation, advance of externally bonded strengthening.

In spite of its promise concerns must be taken to existing materials. In some cases it can be difficult to reach the areas that need to be strengthened. Further, the existing documentation of the structure is often very poor and sometimes even wrong. Furthermore, when strengthening is going to be undertaken, all failure modes must be evaluated. For example, a flexure strengthening can lead to a shear failure instead of giving the desired carrying capacity. Also, it should be noted that not only the failure mode of the strengthened member is important. If a critical member in a structure is strengthened, another member can become the critical one and the whole structure must therefore be investigated.

FRP should not be used in the following situation [11].

- The condition of the substrate is unknown or largely deteriorated
- There is ongoing substantial corrosion of the mild-steel reinforcement
- There is no mild-steel reinforcement to provide ductile behavior, etc.

1.4.1 General FRP Properties

FRPs composite material properties include low specific gravity, high strength to weight ratio, and high modules to weight ratio. Most FRP materials are very resistance to corrosion and solvents. FRP materials exhibit linear elastic stress strain behavior until rupture when loaded in direct tension as shown in Figure 1.6.

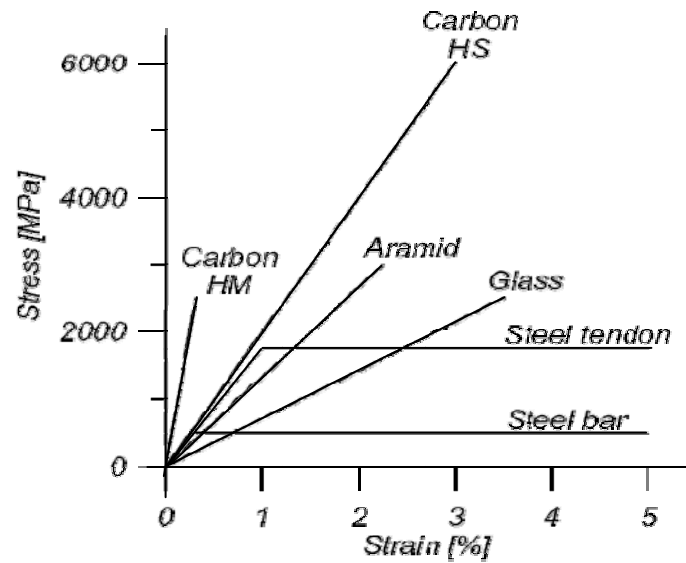


Figure 1.6 Stress-strain relationships of fibers and steel [12]

Another issue that may arise with the use of FRP systems is the effect of high temperature. The resin and adhesive used in FRP systems have low glass transition temperature (T_g). At temperature close to T_g , the FRP starts to lose its ability to transfer stresses from the concrete to the reinforcing fiber. Proper design should take into account high temperature effects and measure undertaken to prevent the FRP system from being subjected to such condition [13].

Externally bonded FRP system should not be used as compression reinforcement due to insufficient testing validity for its use in this type of application. Similar to many engineering materials, FRP materials are also susceptible to creep and fatigue failure. ACI committee 440 proposes limiting the stress level on the FRP by imposing a reduction coefficient factor based on FRP type to prevent such failure [13].

1.4.2 Carbon Fiber Reinforced Polymer.

The technique of strengthening reinforced concrete structures by externally bonded CFRP laminates was started in 1980s and since has attracted researchers around the world. Strengthening with externally bonded CFRP laminates has shown to be applicable to many kinds of structures. Currently, this method has been applied to strengthen such structures as columns, beams, walls, slabs, etc as shown in Figure 1.7. The use of external CFRP reinforcement may be classified as flexural strengthening, improving the ductility of compression members, and shear strengthening.

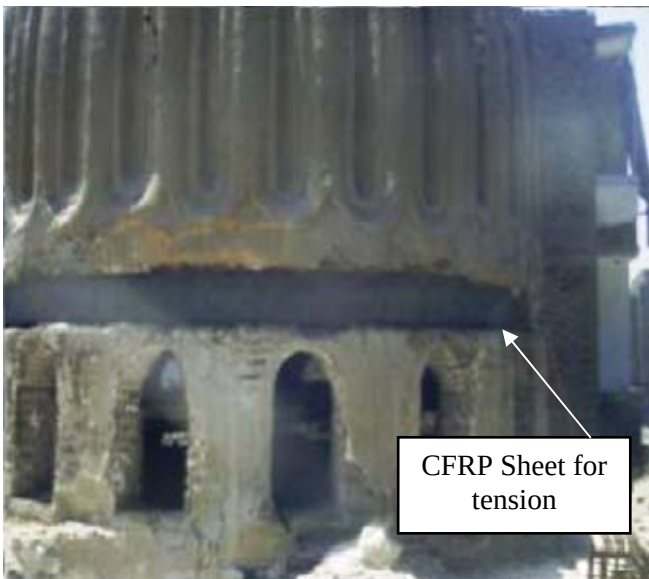
Carbon fibers are the stiffest, more durable, and most expensive fibers. Glass fibers have lower strength and stiffness, compared to carbon fibers but with a lower cost. Strength, stiffness, less susceptible to creep and fatigue rupture and durability of carbon fibers makes it very suitable for strengthening the concrete structure.



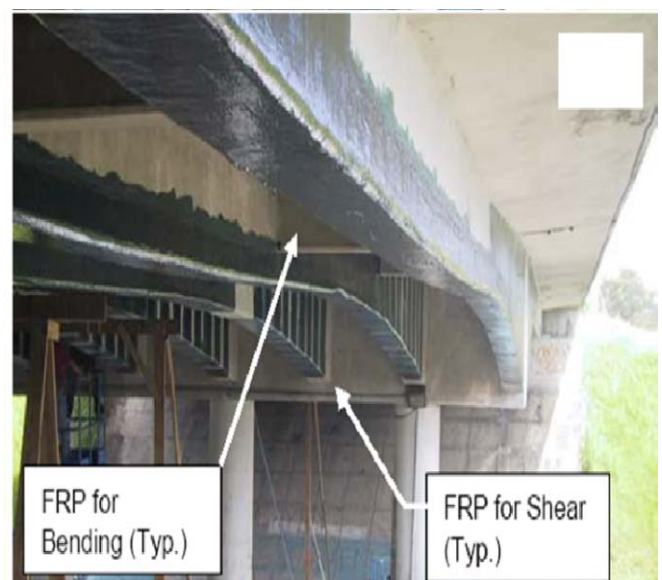
a) Increase axial load in office building [14].



b) Strengthening actual bridge [15].



c) Repair historic dome –Old Cairo [14].



d) Strengthening shear and flexural in Girders of bridge [15]

Figure 1.7 Applications of CFRP strengthening system.

1.5 Objective of the Study

The increase in the use of CFRP composite system for strengthening the concrete structure in shear, flexural, torsion and axial lead to use these system in strengthening the web opening in the reinforced concrete deep beam. The present study aims to:

- 1- Investigate experimentally the behavior of the deep beams with openings strengthened by CFRP laminate.
- 2- Study experimentally the anchorage of CFRP straps in such beams.
- 3- Investigate the different models and techniques for strengthening deep beams with web openings.
- 4- Carry out finite element model to analyze the nonlinear behavior of R.C. deep beams with openings strengthened by CFRP laminate up to failure by using ANSYS(version 9.0) computer program

1.6 Layout of thesis

The present study consists of seven chapters as follows:

The current chapter (**chapter one**) presents a general introduction about the behavior of reinforced concrete deep beam and the application of CFRP. **Chapter two** reviews the most of the previous studies carried out (experimental and theoretical investigation), which are related to the present study. **In Chapter three** an experimental program, detail of specimens, material properties and test procedure are presented. **Chapter four** presents the experimental results. **Chapter five** presents the material modeling used in the finite element analysis, element used, real constant and boundary condition, in addition to nonlinear solution technique. **Chapter six** describes the theoretical result and also a comparison between the experimental and theoretical results. **Chapter seven** presents a summary of the conclusions of the present work and recommendations for further work.

CHAPTER TWO

LITERATURE REVIEW

2.1 General

The initial development of FRP-strengthening technique took place in 1987, in Switzerland, under the leadership of Meier [16]. It was the first on-site repair by externally bonded FRP took place in 1991. Since then, strengthening by externally bond FRP composites has been studied worldwide. The sudden increase in the use of FRP composites was attained after 1995 in Japan. In 1997, more than 1500 concrete structures worldwide had been reinforced with externally bond FRP composites [17].

In this chapter, a brief review is carried out on the experimental and analytical methods that investigated the behavior of reinforced concrete deep beams with and without openings. Also, the behavior of reinforced concrete structure strengthened by CFRP laminate is summarized according to main studies in this field.

2.2 Experimental Work on Reinforced Concrete Deep Beams

In 1965 De Paiva and Siess [18] tested 19 simply support deep beams. All beams had a constant span of 610 mm and the effective depth of these beams was varied from 152, 203, 308 mm which led to (L/d) ratios equal 4, 3, and 2 respectively. The purpose of this study was to present an investigation of the shear strength and behavior of moderately reinforced concrete deep beams. They found that by increasing the percentage of the tension reinforcement, the load capacity of the beam was increased and the mode of failure tends to change from flexural to

shear. The increase in the concrete strength had a negligible effect on the ultimate strength of beams failing in flexure but increased the strength and, in some cases, changed the mode of failure of beams failing in shear. Also, it was found that the addition of vertical and inclined stirrups have no effect on the formation of inclined cracks. The addition of vertical stirrups tends to reduce the deflections at ultimate load.

In 1971 Manuel et al [19] investigated 12 reinforced concrete deep beams in which the variables (a/d) and (L/d) were systematically varied and other major variables were kept constant. They studied the effects of changes in (a/d) and (L/d) ratios on failure. They found that the ultimate strength of reinforced concrete deep beams appears to be significantly influenced by (a/d) ratio and insignificantly by (L/d) ratio, and the diagonal cracking capacity of deep beams is not influenced significantly by (L/d) ratio. The extent of arch action for beams of constant shear span at any level is reduced as the length of beam increases. The influence of (a/d) relates to the mode of failure

In 1972 Kong and Singh [20] tested 45 lightweight concrete deep beams. Different types and amounts of web reinforcement were used. The tested beams were divided into three groups. The beams of group A and group B were different in the spacing of the web reinforcement, and beams of group C were used for a preliminary study of the effect of terminating a longitudinal bar within the shear span. For all beams, the depth (h) was kept constant, the span and clear shear span were varied to change the (L/h) and (x/h) ratios respectively. An attempt was made to isolate the effects of (L/h) and (x/h), on crack control, diagonal cracking loads and ultimate shear loads. They found that (L/h) ratio has little effect on ultimate loads, diagonal cracking loads, crack widths, which are all strongly dependent on the (x/h) ratio. For strength and crack control, the

inclined web reinforcement was very effective for all the (x/h) ratios studied while the effectiveness of other type of web reinforcement depends on the (x/h) ratio. They also found that the addition of a main longitudinal bar, which is terminated within the shear span, would seem to have little harmful effects on maximum crack width, diagonal cracking load, or ultimate load.

In 1982 Smith and Vantsiotis [21] tested 52 reinforced concrete deep beams under two point loads. The objective were to study the effect of vertical and horizontal web reinforcement and the shear span to effective depth ratio on the ultimate shear strength. The test indicated that the web reinforcement produce no effect on formation of inclined crack and seems to moderately affect the shear strength. It was noted that web reinforcement contribution to ultimate shear strength never exceeded the limiting value of $1/3\sqrt{f'_c}$ bd. Horizontal web reinforcement appears little influence on the ultimate shear strength, while its influence was more noticeable in the beam with $a/d < 1.0$. Also, increasing concrete strength increased the ultimate load capacity when the beam has low a/d and seems to diminish as a/d ratio increase.

In 2003 Yang et al [22] tested 21 deep beams to evaluate whether the ACI equation for deep beams applicable to high strength concrete deep beams with steel ratio less than 1%. The design code has been developed for concrete beam without shear reinforcements, and with concrete less than 40 MPa, an average overall depth of 430 mm and average reinforcement ratio of 2.22 %. The test variables included concrete strength, shear span to depth ratio and overall depth within the range of 400-1000 mm. The high strength concrete deep beam exhibit more remarkable size effects with regard to brittle behavior. It was also shown that ACI equation gave similar safety factor on the shear strength

at the first diagonal crack of the high strength-concrete deep beams, but didn't specify a high enough safety factor on their ultimate shear strength due to size effect.

2.3 Experimental Work on Deep Beams with Web Openings

In 1973 Kong and Sharp [23] tested 24 simply supported reinforced concrete deep beams to study the effect of web openings on the ultimate and cracking load. This study presented an empirical formula to evaluate the ultimate shear capacity (Q_{ult}) of reinforced concrete deep beam with opening located at critical shear zone as shown below. This equation was derived for beams subjected to two point loads, applied symmetrically about mid span as shown in Figure 2.1.

$$Q_{ult} = C_1 \left[1 - 0.35 \frac{K_1 X}{K_2 D} \right] f_t D K_2 b + C_2 \sum A \frac{y}{D} \sin^2 \alpha \quad \dots\dots\dots 2.1$$

Where

C_1 = coefficient equal 1.35.

C_2 = coefficient equal 300 MPa for deformed bar and 130 MPa for plain bar.

K_1 and K_2 = coefficients defining the opening position as shown in Figure 2.1.

D , b and X = depth, width and clear shear span of beam respectively (mm), f_t = cylinder splitting tensile strength (MPa), A = area of rebar.

α = angle of intersection between a typical web bar and load path.

y = depth at which web typical web bar intersection the load path (mm).

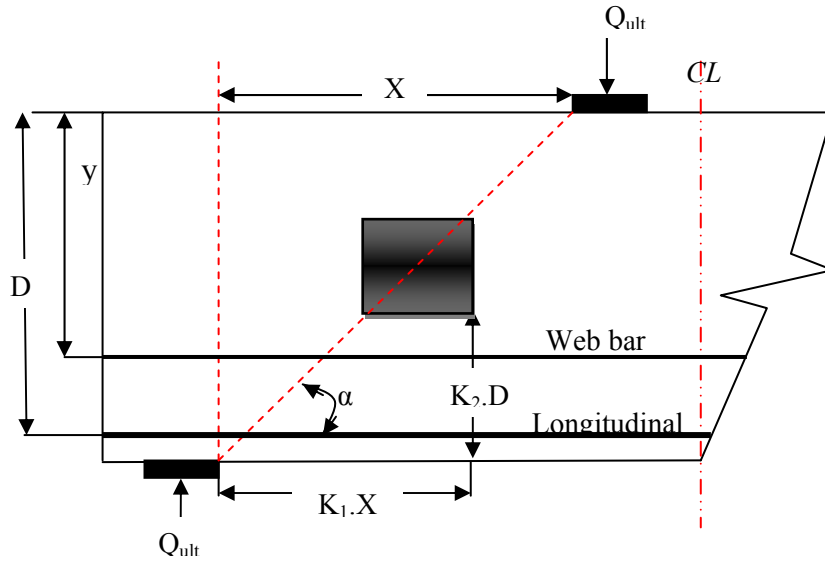


Figure 2.1 Kong's equation notation(1973)[5]

In 1977 Kong et al [24] made a simple structure idealization proposed for predicting the ultimate shear strength of deep beam with web openings. The structure idealization was based on simplifying assumption derived from ultimate load of 56 small scale deep beams. 17 normal weight concrete deep beams including one of full size tested to support the following idealization.

$$Q_{ult} = C_1 \left[1 - 0.35 \frac{X}{D} \right] f_t \cdot D \cdot b + C_2 \sum A \frac{y}{D} \sin^2 \alpha \quad \dots\dots\dots 2.2$$

$$Q_{ult} = C_1 \left[1 - 0.35 \frac{K_1 X}{K_2 D} \right] f_t \cdot D \cdot K_2 \cdot b + C_2 \sum \lambda A \frac{y_1}{D} \sin^2 \alpha_1 \quad \dots\dots\dots 2.3$$

The first term of each equation represented the contribution of the concrete to the ultimate shear strength (Q_{ult}). While the second term represent the contribution of the reinforcement. The first equation used for solid deep beams and for deep beams with opening which didn't interfere significantly with natural load path, joining the load and reaction points as shown in Figure 2.2 (a). While the second equation used for the other cases as show in Figure 2.2(b). The general agreement between the measured and computed results from equations above was reasonably good.

Notation:

The entire coefficients in equation 2.2 and 2.3 are same in 2.1

λ = empirical coefficient, equal to 1.5 for web bars and 1.0 for main longitudinal bars.

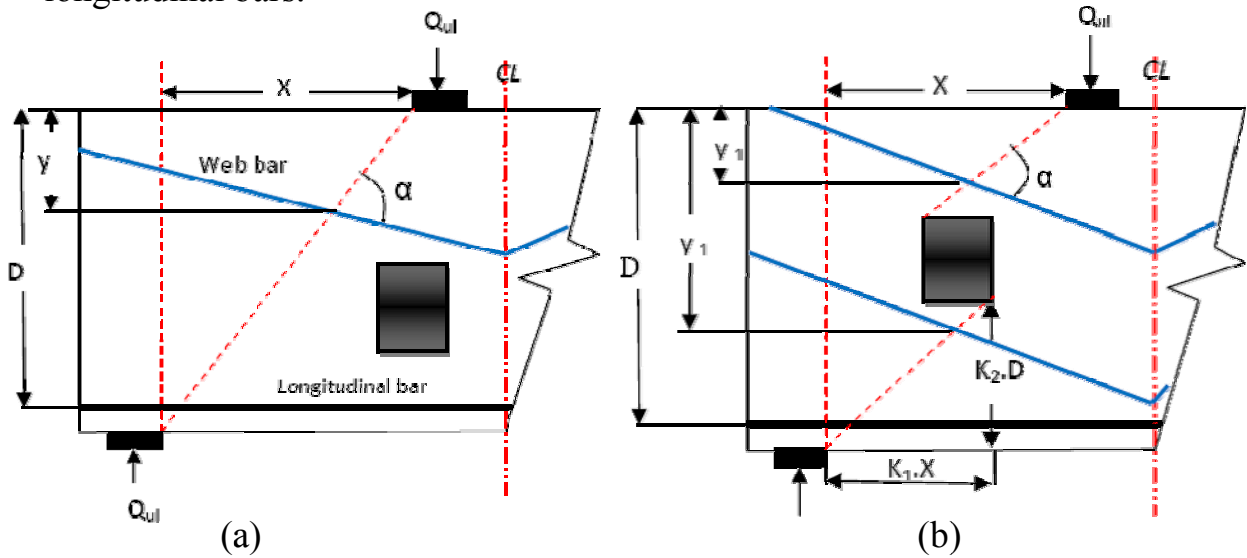


Figure 2.2 Definition of the symbols in equations 2.2 and 2.3[24].

In 1979 Ray and Reddy [25] defined the practical region for web opening in deep beam as shown in Figure 2.3. The region divided into four equal quadrants (1 to 4) by the XX' and YY' axes. It was found the strength of the beam increases when the opening is located away from what can be called loaded quadrant to the unloaded quadrant and vice-versa. For the opening located completely outside the shear region the beam can be assumed as solid deep beam. According to the practical region shown in Figure 2.3, it was found that the maximum size of web opening for practical applicability should not exceed $X_N/2 * 0.6D/2$.

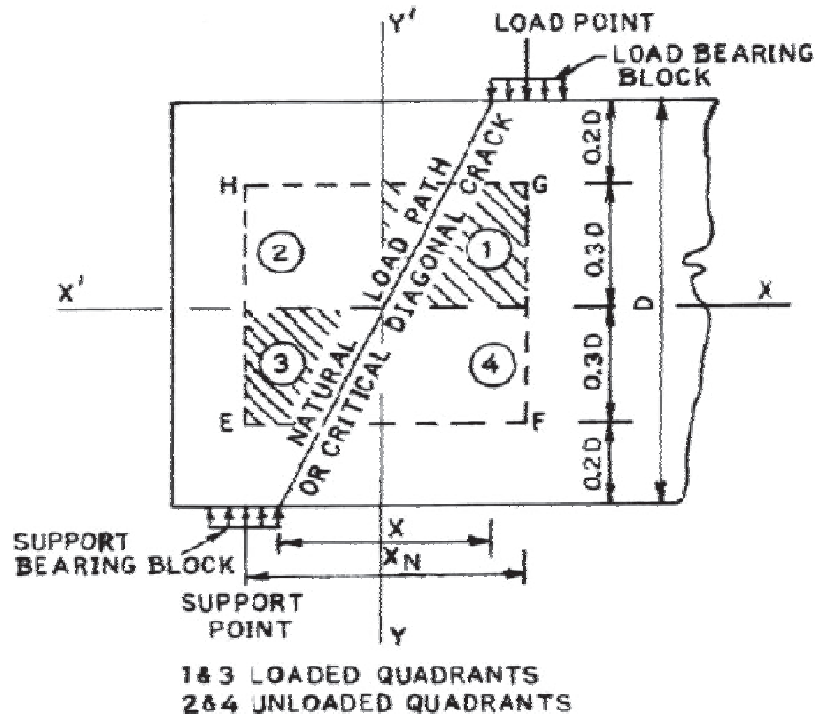


Figure 2.3 practical regions for web opening in deep beam [25].

In the comprehensive test program (Ray and Reddy, 1979; Ray, 1980, 1982) [26] it was concluded that strength of deep beams with web openings affected mainly by.

- i. The shear span to depth ratio.
- ii. The amount of interception of diagonal crack by the opening.
- iii. The location of the center of the opening in the web region.
- iv. The dimensions of the opening.

From the experimental program they found analytical solution for the problem of the web opening by proposing the following simplified assumptions.

- i. The effect of the opening lying within the region FEGH in the web of the deep beam was considered in Figure 2.4.
- ii. The size of the opening is limited to $a_1x \leq x_N/2$ and $a_2D \leq 0.6D/2$ as shown in Figure 2.4.

- iii. The eccentricities e_x and e_y of the opening were limited to the maximum of $X_N/4$ and $0.6D/4$ in the X and Y direction respectively as shown in Figure 2.4 .

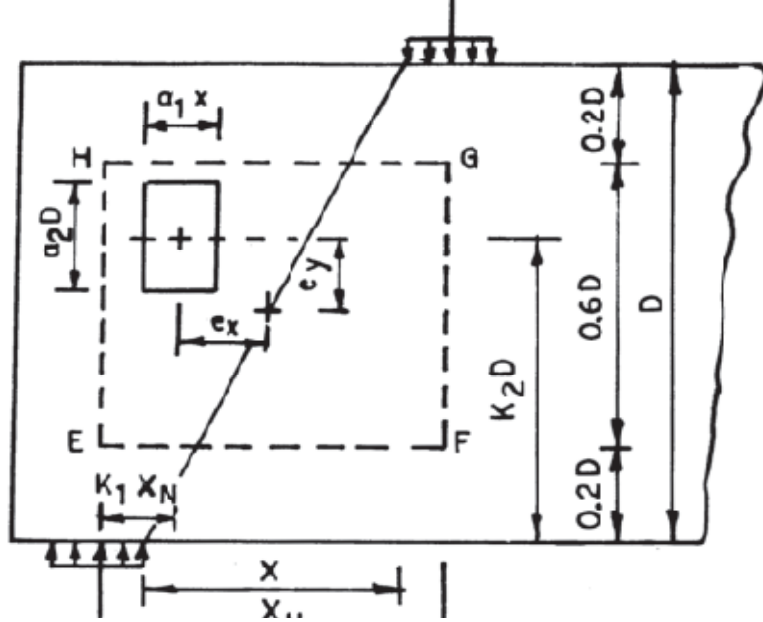


Figure 2.4 Typical opening in the web and other dimension [25]

According to these assumptions the following equations below were proposed for deep beam with web openings. The result of equations comparing with test result of 86 deep beams with web opening (Kong et al 1973, 1977, 1978, Singh 1978, Ray 1980, 1982) and gave good agreement.

$$Q_u/bD = (P_u/2bD) = 0.1 f_c (\lambda_1) (\lambda_2) (\lambda_3) + 0.0085 \psi_s \rho_s f_{sy} + 0.01 \psi_w K_w r_w f_s \dots 2.4$$

Where

$$\lambda_1 = \left[1 - \frac{1}{3} \left(\frac{K1.XN}{K2.D} \right) \right] \quad \text{for } \frac{K1.XN}{K2.D} \leq 1.0, \text{ and } = \frac{2}{3} \quad \text{for } \geq 1.0$$

$$\lambda_2 = (1-m)$$

(m) is the ratio of path length intercepted to total path length along the natural load path = 0 for non-intercepted.

$$\lambda_3 = \left[0.85 \pm 0.3 \left(\frac{e_x}{X_{net}} \right) \right] \left[0.85 \pm 0.3 \left(\frac{e_y}{Y_{net}} \right) \right]$$

$$e_x \leq X_N/4 \quad e_y \leq 0.6D/4$$

$$X_{net} = (X_N - a_1X) \quad Y_{net} = (0.6D - a_2D)$$

Ψ_s = empirical coefficient = 0.65, Ψ_w = 0.5 is reasonable factor

$$\rho_s = A_s / bD * 100 (\%), r_w = \sum_n \frac{Aw}{bD} * 100 (\%)$$

K_w = 0.85 for horizontal web bar

Cot β for vertical web bar

1.15 for inclined web bar

In 1985 Somsak and Nandivararm [27] present experimental investigation of steel fiber reinforced concrete deep beams with openings. Sixteen specimens with opening tested up to failure. The variable included fiber content (0.5%, 1.0%, and 1.5%), the position of the openings (five positions in the shear span were investigated) and the loading condition. It was appeared from the test results that the addition of the steel fiber to the concrete matrix results in a significant increase in both the first crack load and the ultimate strength. Both values had approximately doubled when steel fiber was 1.5%. Also, it was shown the reduction in strength was large when the openings placed closer to the support on the natural load path while the beam with openings away from the natural load path was much stronger than those with opening intercept the load path.

In 1986 Hagne et al [28] tested thirteen deep beams. It was examine the effects of the nature and magnitude of stress distribution of the following parameters, beam dimension, presence of openings, and the position of the openings by using photoelastic techniques. It was found that for shallower beam the mid-span extreme fiber tension increased as the position of the opening shift toward the center of the span. However, for the deeper beams the bottom fiber stress at the mid-span remain almost unaffected by varying the position of the opening along the span. The mid-span bending stresses along the depth of the beam compared with the stresses predicted by simple bending theory, it was found that the

classical bending theory significantly underestimate the mid-span tension and overestimate for compression.

Yang et al (2006) [29] studied experimentally and analytically the effect of web openings in reinforced concrete deep beams. Thirty-two reinforced high strength concrete deep beams with and without openings were tested under two top point loading. Test variables included concrete strength, shear span to depth ratio and dimensions of the opening. The result indicated that the depth and width of opening didn't affect the mid span deflection at initial load stage, but it significantly affected the deflection after occurrence of diagonal cracks. The influence of concrete strength on the ultimate shear strength significantly decreased in deep beams with openings rather than solid deep beam. Also, it was found that the equations proposed by Kong et al was suitable for reinforced high strength concrete deep beams with web openings.

Yang et al (2007) [30] tested 15 reinforced concrete deep beams with openings. All beams tested had the same overall geometrical dimension. The web opening was located at the center of the shear span to completely interrupt the concrete strut. The main variables considered were the opening size and amount of inclined reinforcement. It was found that the diagonal crack width and shear strength of tested beams were significantly dependent on the effective inclined reinforcement. As the inclined reinforcement increase the diagonal crack width and its development decrease and shear strength of the beams tested improved. Numerical procedure based on the upper analysis of the plasticity theory was proposed to estimate the shear strength of deep beam with web opening and showed a good agreement with the test results.

Yang and Ashraf (2008) [31] tested twenty-two reinforced concrete continuous deep beams with openings and two companion solid deep beams were tested to failure. The main variables investigated were the configuration of the web reinforcement around the openings, location of the openings, and the shear span to depth ratio. The test result showed that the development of the diagonal crack width nearly independent on the configuration of web reinforcement of beam having openings within exterior shear span. A more effective control of diagonal crack width was exhibited by beam with inclined web reinforcement than beam with vertical and horizontal web reinforcement.

In 2008 Ridha [32] investigated experimentally and theoretically, the strengthening of reinforced concrete deep beams by using steel fibers. The experimental work could be divided in two parts; the first part consists of twelve solid deep beams to show the effect of steel fiber on the ultimate and cracking load, deflection and concrete strain. The second part consists of casting of six deep beams with web openings to study the same previous parameters. The results obtained from the experimental work demonstrated that when the steel volume fraction increased, the ultimate and cracking load increased for deep beams with and without web openings. Also, it was found that the effect of steel fiber increases when (a/d) decreases in solid deep beams, but didn't effect in the deep beam with web opening. Deep beams with web openings were stiffer when the fiber contain increased from 0.0% to 0.5% and 1.0%. Nonlinear finite element program have been developed to include the effect of steel fiber and good agreement with the experimental work was achieved.

2.4 Experimental Studies on Concrete Structure Strengthened by FRP

Berset in 1992 performed the first research focusing on shear strengthening of reinforced concrete beam with composites materials, by testing reinforced concrete beams with externally bond (GFRP) laminate. Berset proposed a simple analytical method to compute the contribution of externally reinforcement to the shear capacity similarly to stirrups contribution, and based on maximum allowable strain which determine from experimental [17].

Abdulla et al (2003) [33] tested 10 reinforced concrete beams with openings. CFRP was used for strengthening the region of the opening. The experimental program included strengthening five beams with openings, while four beams kept without strengthening and the last one was solid beam used as control beam. It was found that, when the opening with height of 0.6 from the beam depth reduce the capacity by 75%. The application of the CFRP sheet around the opening was greatly decreased the beam deflection, controlled the cracks around the openings and increased the ultimate load capacity of the beam. The failure occurred due to a combination of shear cracking of concrete and bond failure of CFRP sheets glued to concrete surface.

Zhang et al (2004) [2] tested 16 reinforced concrete deep beams without shear reinforcement. They were divided into four group four beams in each group. One beam in each group was a control beam, while the others were strengthened by various configuration of CFRP shear reinforcement. The result of the test was demonstrate the feasibility of the use externally applied CFRP system to restore or increase the shear capacity of deep beam. It was found in a regular beam situation when the

shear span to effective depth ratio has large value the anchorage for vertical CFRP shear reinforcement will greatly improved the shear strength. But, when the shear span to effective depth becomes smaller value, or the beam behave like a deep beam the anchorage of the vertical CFRP shear reinforcement will not likely to improve the shear strength as much as a regular beam case. Also, it was noted the effected stresses in the CFRP laminates at the failure didn't reach the ultimate tensile stress of CFRP laminate.

Islam et al (2005) [34] explores the prospect of strengthening structurally deficient deep beams by using an externally bonded fiber reinforced polymer (FRP) system. Six identical beams were fabricated and tested up to failure. One of these beams was tested in its virgin condition to serve as reference, while the remaining five beams were tested after being strengthened using carbon fiber wrap, strip or grids. Test results have shown that the use of a bonded FRP system leads to a much slower growth of the critical diagonal cracks and enhances the load-carrying capacity of the beam to a level quite sufficient to meet most of the practical upgrading requirements.

Elgwady et al [35] examined six corbels strengthened by CFRP to study the effectiveness of using CFRP as an external strengthening method to increase the load carrying capacity of the corbel. Different configurations were used. The test result indicated that the proposed technique had potential in improving ultimate load carrying capacity of the short cantilever. The use of CFRP as diagonal strip increased the ultimate load carrying capacity of the corbel by 70% of the control specimen. All other corbels tested had ultimate load carrying capacity higher than control corbel with difference ranging from 8 % to 30 %.

Enochsson et al (2007) [36] tested 11 slabs with openings, loaded with distributed load. Six slabs with opening had been strengthened with CFRP sheet. These slabs were compared with traditionally reinforced slab. All slabs had same dimensions (2.6*2.6 m) but difference in size of opening, small opening (0.85*0.85 m), large (1.2*1.2 m), located at the center of the slab. The test results showed that the load carrying capacity can be increased when compared with homogenous slab. For the CFRP strengthened slabs the load carrying capacity were increased with (24-125 %) in comparing with control slab of small and large opening respectively, and (22-110%) in comparison to homogenous slab. The general results noted that the slab with large opening had a noticeable higher load carrying capacity and stiffer load deflection response than the slab with small opening. The reason for this behavior can be the slab with large opening behaves closer to the system of four beams than slab.

In 2007 Ali [37] investigated experimental and theoretical the behavior of reinforced concrete beams, strengthened or repaired by CFRP for both flexural and shear. It was observed that the use of CFRP sheet connected to the tension side of beams could enhance the ultimate load capacity up to 160% in flexural over the ultimate load for the reference beam. The repaired beams reach 95% to 97% of ultimate load in comparison of those strengthened by same way. The shear group shows the use of CFRP sheet could enhanced the ultimate shear capacity up to 200% as compared with reference beam. ANSYS computer program was used to simulate a numerical model for reinforced concrete beam strengthened or repair by CFRP. Full bond between the CFRP and concrete interface was considered. Comparison between the experimental and numerical results asserted the validity of the numerical analysis and the methodology developed.

El Maaddawy and Sherif (2009) [3] examined the potential use of externally bonded carbon fiber reinforced polymer as a strengthened solution to upgrade reinforced concrete deep beams with web openings. A total of 13 deep beams with opening were constructed and tested under four point loads. Test specimens had a cross section 80*500*1200 mm. Two square open, one in each shear span, were placed symmetrically about the centerline of the beam. Test parameter included the opening size, location and the presence of the CFRP sheets. Externally bonded of CFRP shear strengthening around the opening was found very effective in upgrading the shear strength of reinforced concrete deep beams. The strength gain caused by CFRP sheets was in the range of 35% to 73%. Kong's equation was modified to include the effect of CFRP sheet.

Hwang et al [38] presented experimentally and analytically studies related to seismic retrofitted of reinforced concrete frames containing partition walls using the CFRP laminate. The test result indicated that the use of CFRP laminate with sufficient end anchorage was quite effective in enhancing the shear strength of partition walls. The application of the CFRP sheets without anchor system was worthless. Also, it was found the strengthening low-rise walls by vertical CFRP sheet were more effective than the application of horizontal CFRP sheet. Experimental and analytical results indicated that the shear resisting mechanism of reinforced squat walls can be modeled as the strut and tie action.

2.5 Nonlinear Finite Element Analysis of Reinforcement Concrete Deep Beams

In 1986 Mahmood [39] analyzed reinforced concrete deep beams using a nonlinear finite element technique. The nonlinearities included the stress-strain behavior of concrete and its anisotropy under varying biaxial stress conditions, cracking and crushing of concrete, yielding of steel, the nonlinear bond stress-slip phenomenon and the post-cracking shear transfer by aggregate interlock. Concrete was represented by using quadrilateral isoparametric elements, or quadrilateral elements with incompatible mode. Main steel was represented by either using simple truss elements or constant strain triangular elements connected to the concrete nodes by zero length spring linkage elements with proper stiffness to simulate bond and dowel action. The incremental-iterative procedure with variable stiffness matrix was adopted in the non-linear solution algorithm. Analytical results obtained, such as deflections, crack pattern, ultimate loads and failure mechanism showed reasonable agreements with published experimental and analytical results.

In 1997 Aziz [40] tested 28 reinforced crushed stone concrete deep beams under two equal point loads. Test specimens were divided into three series with and without web reinforcement. The beams were tested for different shear span to depth ratio ranging from 1.05 to 2.22, and different effective span to effective depth ranging from 2.53 to 7.41. The investigation objectives were to study the effect of shear to effective depth ratio, effective span to effective depth ratio, compressive strength of concrete, and web reinforcement. It was found that the ultimate shear stress decreases with increasing shear span to effective depth ratio and maximum size of aggregate, while it increases with increasing compressive strength of concrete and amount of shear reinforcement.

In 2002 Al-Shraify [41] used three dimensional finite element models to simulate the behavior of reinforced concrete continuous deep beams. Concrete was modeled using 20-node isoparametric quadratic element, while the reinforcing bars were modeled as axial members embedded within the concrete element. Perfect bond between the concrete and steel assumed to occur. In general a good agreement between the finite element solution and the experimental work obtained. From the parametric study it was found that by increasing compressive strength from 30 to 50 an increase in shear strength by 23%. Also when the clear span to effective depth ratio was decrease by 37% the failure load was increased by 46%.

In 2005 Mosheer [5] developed a nonlinear finite element procedure to simulate the behavior of reinforced concrete deep beam with web openings. Three dimensional finite elements under static load with material nonlinearity were developed to account the nonlinear stress-strain relationship of concrete and steel. The result of finite elements and available of experimental and analysis data of simply support and continuous deep beams show good agreement, with difference about 6% as average for the ultimate load and 9% as average for deflection. Some selected parameters such as opening location, web reinforcement, shear span to depth ratio, compressive strength and boundary condition were studied. It was found that providing an opening in the shear zone causes sharp decreasing in ultimate loading by (31-56%) for simply supported deep beams and (8-30%) for continuous deep beams.

Al-kaisy (2005) [42] investigate the behavior of reinforced concrete normal beams with large transverse openings subjected to flexure by using a three-dimensional nonlinear finite element model. Different types of reinforced concrete beams with large rectangular transverse openings have been analyzed by the finite element method and are compared with the experimental data. Several parametric studies have been carried out to investigate the effect of some important finite element and material parameters. In general good agreement between the finite element solutions and the experimental results has been obtained. The numerical results for the analyzed beams show that the ultimate load increases by about 22% when the length of opening decreases from 800mm to 400 mm, and it decreases by 32% when the length of opening increases from 800 mm to 1200 mm. Also, as the depth of the opening decreases from 180 mm to 100 mm, the ultimate load increases by about 14%, and it decreases by 46% when the depth of opening increases from 180 mm to 260 mm.

2.6 Nonlinear Finite Element Analysis of Reinforced Concrete Beams Strengthened by FRP

ANSYS finite element model was produced by **Kachlakev et al.(2001) [43]**, to study the effects of shear strengthening by comparing the behavior of two full-scale reinforced concrete beams. Experimental beams replicated the transverse members from the Horsetail Creek Bridge, which were deficient in shear reinforcement. Three-dimensional finite element models were developed using layered elements for the FRP composites. It was found that the general behavior through the linear and nonlinear ranges up to failure of the finite element models show good agreement with observations and data from the experimental full-scale beam tests. The addition of GFRP to the control beam shifts

the behavior of the actual beam and model from a sudden shear failure near the ends of the beam to flexure failure by steel yielding at the mid-span. This finite element model can be used in additional studies to develop design rules for strengthening reinforced concrete bridge members by using FRP.

Santhakumar et al. (2004) [44] presented a numerical study to simulate the behavior of retrofitted reinforced concrete shear beams. The study was carried out on an unretrofitted RC beam designated as control beam and RC beams retrofitted using CFRP composites with $\pm 45^\circ$ and 90° fiber orientations. The effect of retrofitting on uncracked and precracked beams was studied too. The finite elements adopted by ANSYS were used in this study. The load deflection plots obtained from the numerical study showed good agreement with the experimental plots. There was a difference in behavior between the uncracked and precracked retrofitted beams though not significant.

Al-Shmmmary (2006) [45] presented a three-dimensional nonlinear finite element model suitable for the analysis of reinforced concrete beams flexural strengthened with CFRP plates under static load. The ANSYS computer program has been used to investigate the behavior of the different composite CFRP reinforced concrete beams. The analytical results of load-deflection response along the examined beams have been compared with available experimental tests. In general, a good agreement between the finite element solutions and experimental results has been obtained with difference about (2 %). Parametric studies have been carried out to investigate the effect of some important material parameters. It was found that the flexural strengthening of R.C. beams with CFRP is effective with an increment in the ultimate load about (30-45) %.

2.7 Summary

1. The shear span to depth ratio (a/d) is an important factor effect on the behavior of R.C. deep beams. A large increase in the shear capacity beyond the inclined crack for $a/d < 2.5$, this increase mainly attributed to arch action that seems to decrease with increase a/d .
2. The clear span to depth ratio (L/d) has lesser effect on the behavior of R.C. deep beams if compared with shear span to depth ratio (a/d).
3. The shear capacity of deep beam increased by increase concrete strength, but this didn't affect on the behavior of the deep beam with web openings.
4. The inclined web reinforcement is the best way to be used in R.C. deep beams.
5. Addition of web reinforcement (vertical, horizontal and inclined) didn't effect on the formation of inclined crack.
6. CFRP system has been shown very effective in upgrading the concrete structure (slab and beam) with opening.
7. The critical position through the web of concrete beam is in the center of shear span and near the support.
8. The structure response of reinforced concrete deep beams with openings preliminary depend on the degree of interruption of the load path (compressive strut).
9. When the beam behaves like deep beam, the vertical anchorage of CFRP sheet was approximately not improves the shear capacity.

Its well known from the literature above there is slight technical data except Ref. [3] about strengthening reinforced concrete deep beam with web opening by CFRP sheet and it deferens from present study in the strengthening model. The present work is providing an examination of the use of CFRP sheet to strengthen the deep beams with web opening.

CHAPTER THREE

EXPERIMENTAL WORK

3.1 General

The main objective of the present work is to study the effect of CFRP in improving the shear capacity of reinforced concrete deep beams with web openings. This chapter describes the details of test specimens, strengthening system, material properties and test procedure.

3.2 Description of Specimens

Eight simply supported RC deep beams with and without web openings having a total span (L) of 1150 mm, overall depth (H) 750 mm, and width 100 mm, with shear span to overall depth ratio (a/H), 0.466 and effective length to depth ratio (l_n/H), 1.267, to ensure that deep action will be developed.

All beams tested under two points top loading as shown in Figure 3.1. Two Ø 16 mm deformed bar were provided as longitudinal tension reinforcement. The vertical and horizontal shear reinforcement was omitted from design to emphasize the effect of CFRP. The ends of all beams extended 100 mm beyond the support's centerlines and the steel bar had a 90° hook of length 250 mm at each ends to provide sufficient anchorage. The concrete cover of 30 mm was adopted to prevent splitting failure.

Seven specimens had two rectangular openings (100X200 mm) symmetrically about the center of specimens. The center of opening of all beams positioned at the center of shear span region, which is the critical load path.

3.3 CFRP Strengthening System

Strengthening system is chosen carefully according to some considerations, mainly, crack pattern around the opening, practical applied in the actual and economic. The first beam (C-1) was kept without opening and without strengthening (solid deep beam) as shown in Figure 3.1, it is considered as a control beam for comparison. While the second beam (C-2) had two openings and kept without strengthening as a control beam.

The strengthening beams (S-1, S-2 and S-3) were strengthened by the same shape and dimension as shown in Figure 3.1 but there is a difference in the configuration of wrapping system. It was strengthened by full wrap, U-wrap and two side's wraps respectively. The opening was surrounded with four CFRP stirrups of 50 mm width, one on each side of the opening also one below and are above the opening, all glued on the concrete surface by 400 mm length and 50 mm width as horizontal stabilizer CFRP sheets. The aims of these strengthened systems are to investigate the effect of anchorage in the improving the shear capacity of deep beam.

The sixth beam (S-4) was strengthened by the shape as shown in Figure 3.1. This system of strength was adopted to overcome the problem of stress concentration at the corners of the opening. The seventh beam (S-5) was strengthened by pair of loop at 45° around the opening as shown in Figure 3.1. The last beam (R-1) was loaded by 70% of the ultimate load of (C-2). This beam will be strengthened by

the successful system that will be found in the test

NOTE: all stirrups of CFRP had a width of 50 mm and one layer of thickness 0.131 mm.

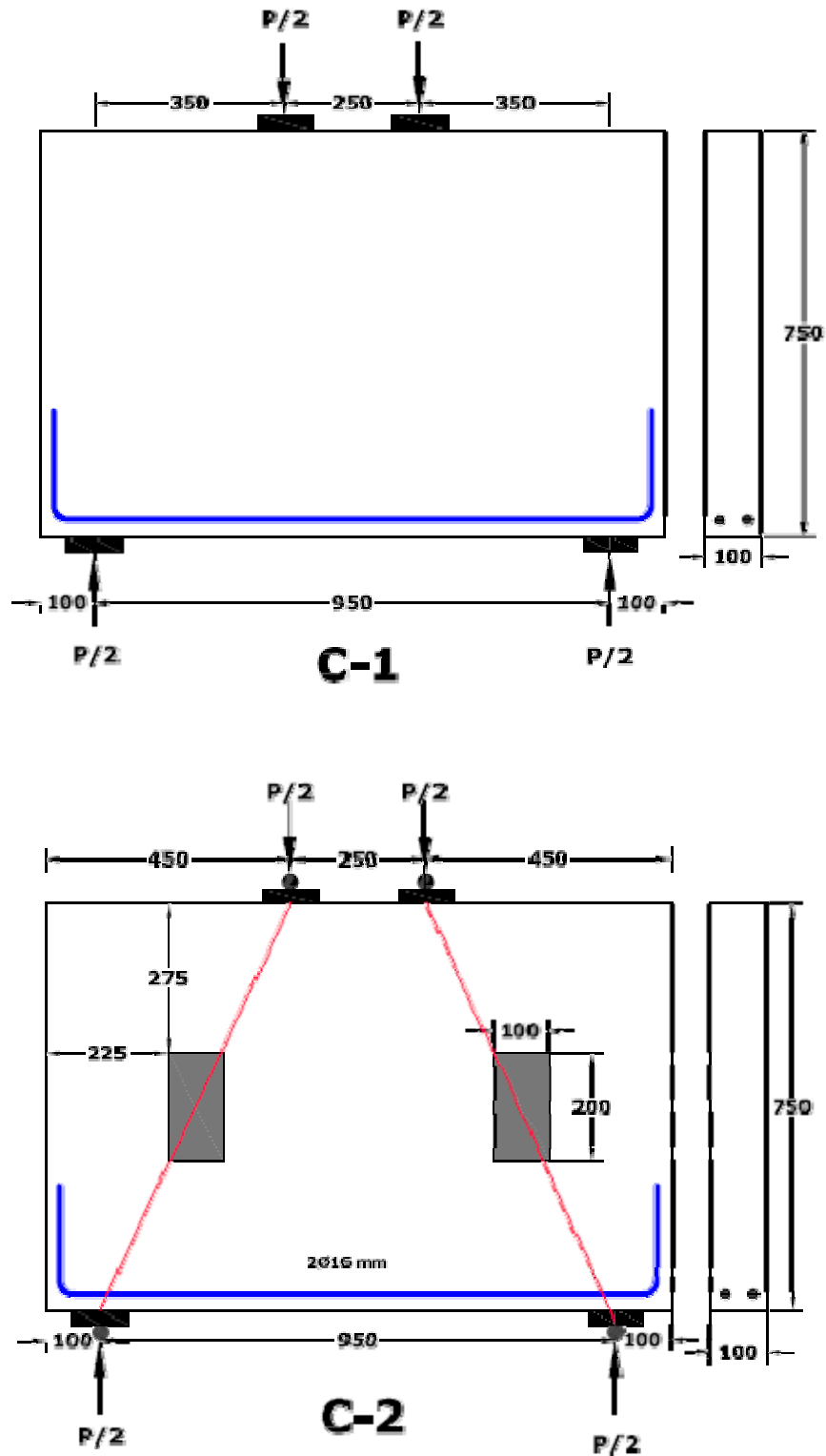
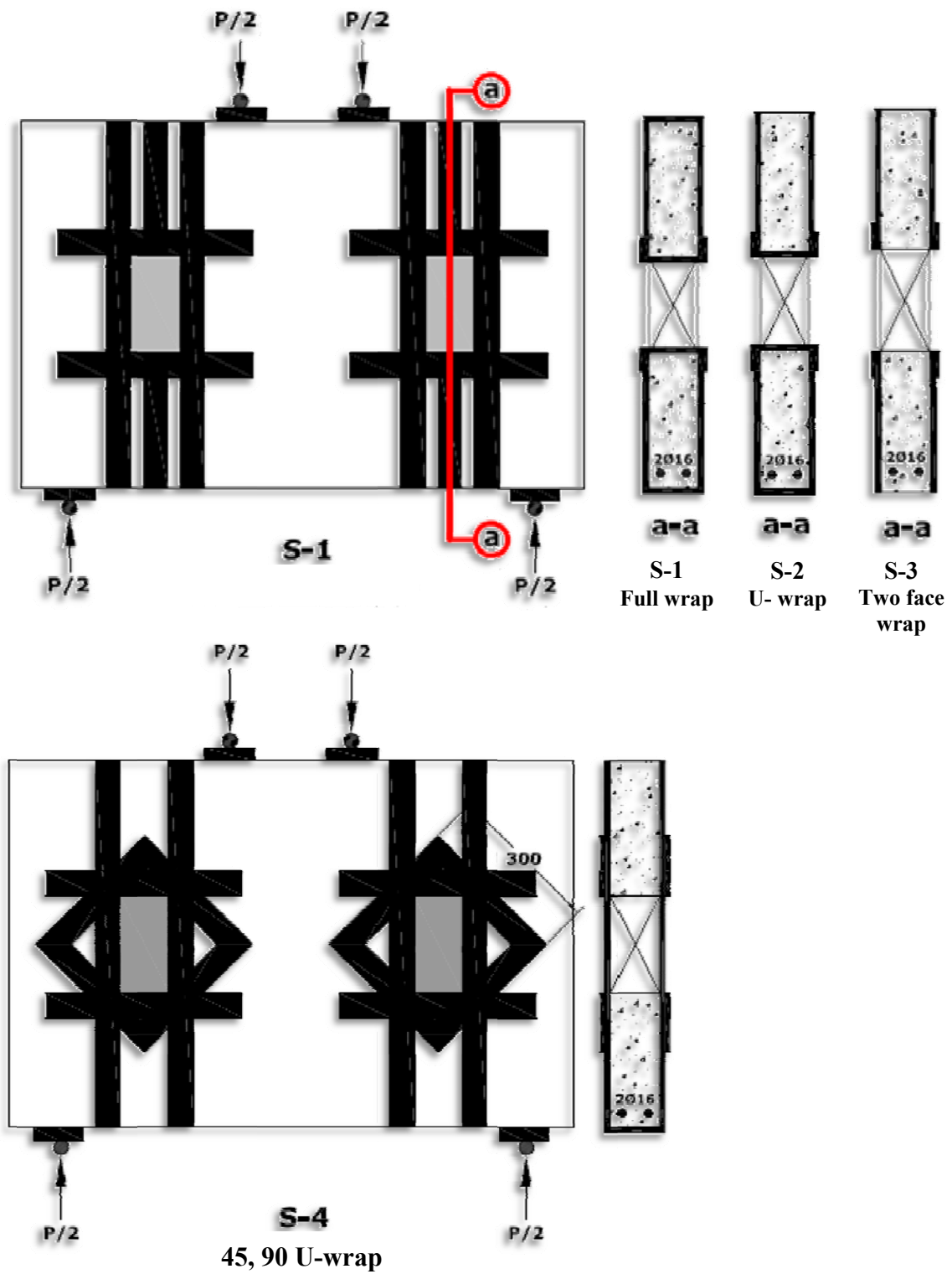


Figure 3.1 Detail and geometry of specimens (all dimensions in mm)

*Figure 3.1 Continued*

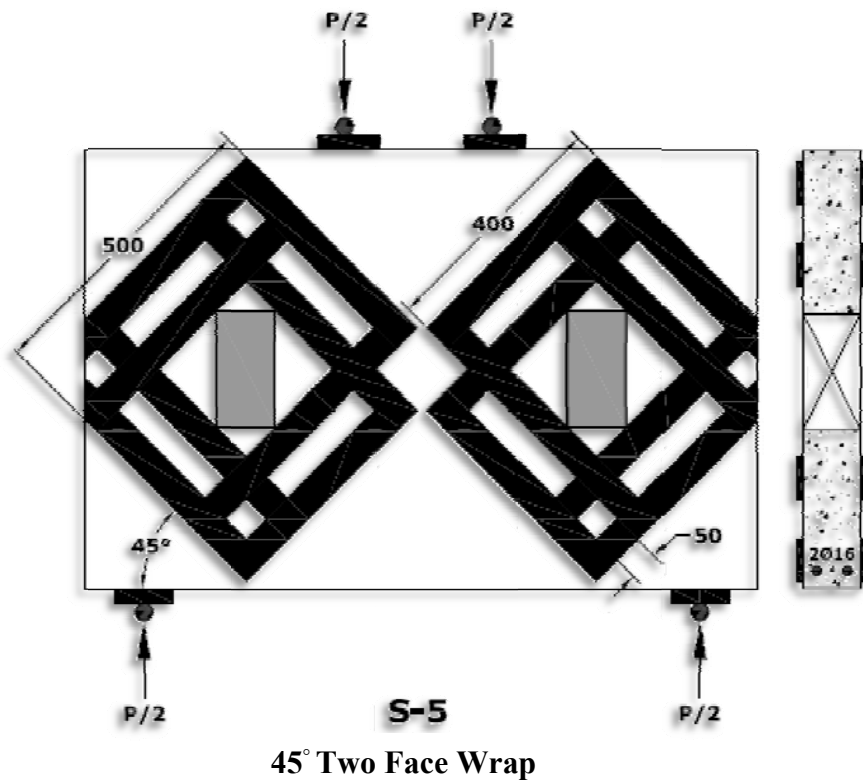


Figure 3.1 Continued

3.4 Material Properties of Test Specimens

Standard tests according to American Society for Testing and Material (ASTM) and Iraqi Specification were conducted to determine the properties of material.

3.4.1 Cement

Ordinary Portland cement (type I) was used in this work. The chemical and physical test results are given in Table 3.1. They confirm to the Iraqi Specification NO. 5/1984[46].

Table 3.1 Chemical and Physical Test Result.

Chemical Analysis			
Oxide	Test result	Limits according to IQS	Conformed to IQS
CaO%	62.82	-----	
SiO ₂ %	21.18	-----	
Al ₂ O ₃ %	4.16	-----	
Fe ₂ O ₃ %	4.48	-----	
MgO%	1.55	≤ 5%	OK
SO ₃ %	2.23	≤ 2.5% if C3A < 5% ≤ 2.8% if C3A > 5%	OK
Free laminate%	1.23	≤ 4%	OK
Loss on ignition %	3.18	≤ 4%	OK
Insoluble residue%	0.86	≤ 1.5%	OK
L.S.F%	0.89	0.66 – 1.02	OK
C3S%	49.05	-----	
C2S%	23.72	-----	
C3A%	3.44	≤ 3.5%	OK
C4AF%	13.63	-----	
Physical Test			
Setting time, initial	115	≥ 45 min	OK
Final	210	≤ 600 min	
Fineness (Blaine) m ² /Kg	345	≥ 250	OK
Compressive strength(MN/m ²)			
3 days	21	≥ 15	OK
7 days	29.8	≥ 23	OK
Soundness (auto clave)%	0.15	≤ 0.8%	OK

3.4.2 Fine Aggregate (Sand)

Natural sand locally available was used. The result of sieve analysis is given in Table 3.2 is compared with Iraqi Specification NO.45/1984[47].

Table 3.2 Grading of Fine Aggregate

No.	Sieve size	Passing %	
		Fine aggregate	Iraqi Specification
1	10 mm	100	100
2	4.75mm	96.97	90 – 100
3	2.36 mm	83.84	75 – 100
4	1.18 mm	66.67	55 – 90
5	600 μ m	42.43	35 – 59
6	300 μ m	16.17	8 – 30
7	150 μ m	3.03	0 – 10

3.4.3 Course Aggregate (gravel)

Locally available gravel of 19 mm maximum size was used. The gravel was washed by clean water several times, and then it was left on air to dry. The grading of coarse aggregate is given in Table 3.3. The obtained result is compared with Iraqi Specification NO.45/1984 [47].

Table 3.3 Grading of Coarse Aggregate

No.	Sieve size	Passing %	
		Coarse aggregate	Iraqi Specification
1	19 mm	100	100
2	14mm	96	100
3	10 mm	88	85 – 100
4	5 mm	17	0 – 25
5	2.36 mm	1.2	0 - 5

3.4.4 Concrete Mix, Cast and Curing

Normal weight concrete was used to cast all specimens. It was decided to choose a mix of 1:1.5:3 (by weight) cement, sand, gravel respectively and 0.5 water cement ratio.

The concrete was mixed in a horizontal rotary mixer. Mixing of concrete was performed according to ASTM C192-1995[48]. In all mixes, firstly, the coarse and fine aggregate are stoked and blended in 2/3 of the required water for sixty seconds. Then, cement and rest water were added and mixed for three minutes. After mixing, the concrete was poured into lightly oiled mould in two layers and well compacted by plunger mechanical vibrator (3000 rpm), 5 seconds for each insertion.

All specimens were removed from their mold after 24 hours of the casting, and then burlap sacks were placed over the beams and kept wet until 28 days. Figure 3.2 shows the equipment of work and curing procedure. Mechanical testing (compression strength and tensile strength) were made for each specimens. The result of these tests is given in table 3.4.

Table 3.4 Concrete material properties of test beams

Type of test Beam name	1 f_c (N/mm ²)	2 f_t (N/mm ²)	3 E_c (N/mm ²)
C-1	25.17	2.62	23580
C-2	24.96	2.704	23481
S-1	26.06	2.462	24897
S-2	27.15	2.826	24490
S-3	26.97	2.536	25297
S-4	25.78	2.511	23864
S-5	25.7	2.631	23827
R-1	24.21	2.716	23216

- Each value in this table is the average of three specimens.

Note (Type of Test)

1. Concrete compressive strength by cylinder of (150*300 mm).
2. Concrete splitting tensile strength by cylinder of (150*300 mm).
3. Concrete modulus of elasticity is calculated according to ACI 318

$$[1]. E_c = 4700\sqrt{f'_c}$$

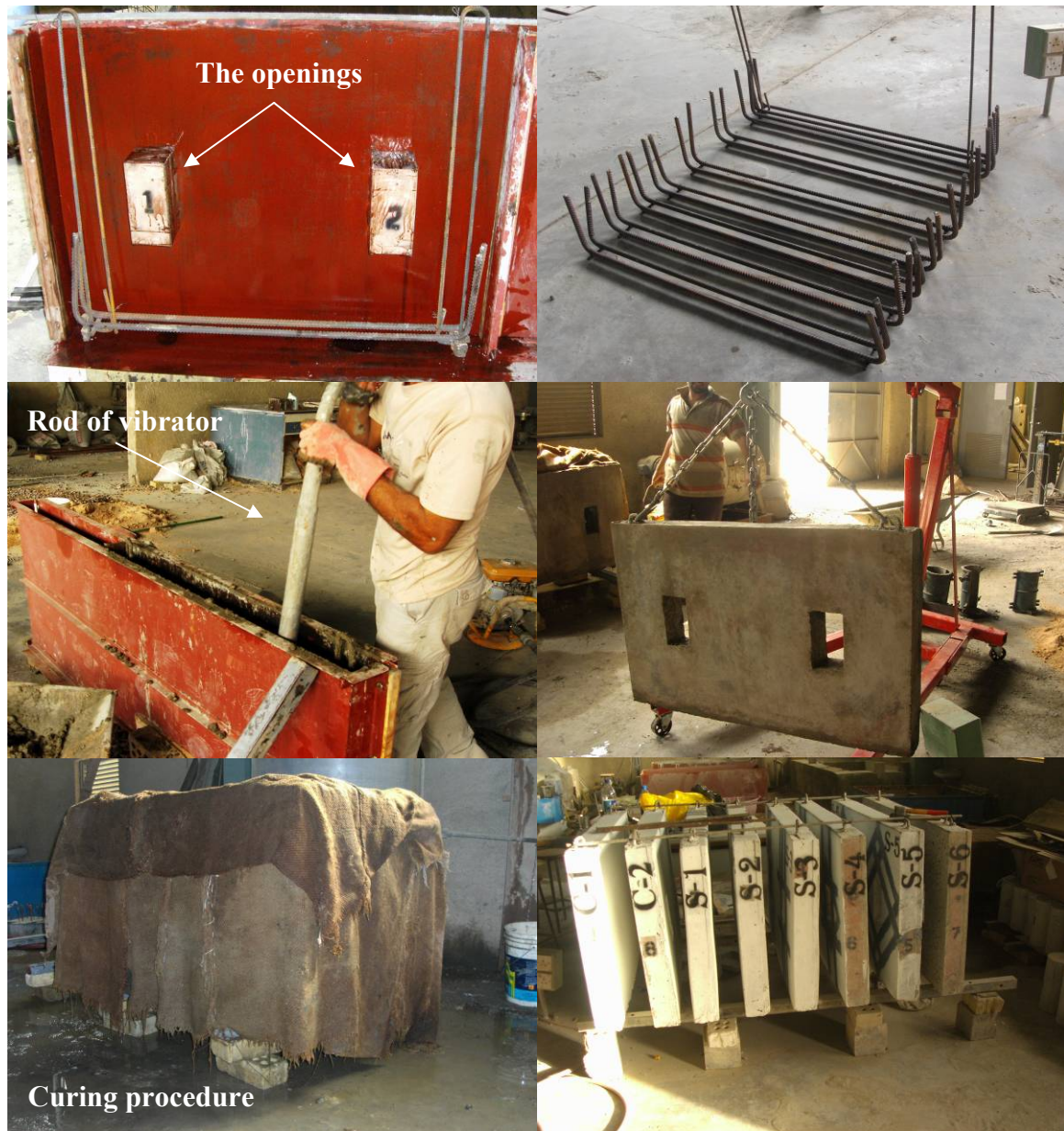


Figure 3.2 Equipment and curing procedure.

3.4.5 Properties of Steel Reinforcement

For all beams one size of steel reinforcing deformed bars was used, bar size of Ø 16 mm Ukrainian organize. Table 3.5 gives the properties of steel reinforcement according to ASTM A370-05a [49].

Table 3.5 Test Result of Reinforcement Bar.

Bar size mm	Yield stress N/mm ²	Ultimate strength N/mm ²	Modulus of elasticity N/mm ²
16	521	620	200000

➤ All these tests were conducted with the help of the Construction and Martial laboratories of Babylon University.

3.4.6 Carbon Fiber Reinforced Polymer (CFRP)

When loaded in tension, CFRP doesn't exhibit any plastic behavior (yield) before rupture. Sika wrap® Hex-230C was used for strengthening as shown in Figure 3.3. The mechanical properties of CFRP laminate are written in Table 3.6 according to manufacturing specifications of Sika Company [50].

Table 3.6 Technical Properties of CFRP Laminate [50].

Properties	SikaWrap® Hex-230C
Tensile strength N/mm ²	4300
E-modulus N/mm ²	238000
Elongation at break (strain)	1.8%
Thickness mm	0.131
Density Kg/m ³	1760

➤ It is too difficult to test the CFRP sheet and epoxy in the available laboratories. So that, the technical data sheets provided from the manufacturer are adopted.

3.4.7 Epoxy Adhesive Properties

The most suitable adhesive material with CFRP sheet is Sikadur[®]-330 as shown in Figure 3.3. The adhesive type consists of two compounds, compound A (white colour) and compound B (grey colour). The mix ratio was 4:1 as A:B. Its main properties as supplied by the manufacturer are shown in Table 3.8[51].

Table 3.8 Technical Properties of Adhesive Material [51].

Properties	Sikadur [®] - 330
Tensile strength N/mm ²	30
E-modulus N/mm ²	4500
Elongation at break (strain)	0.9%
Open time (minute)	30 at +35° c
Full cure	7 days

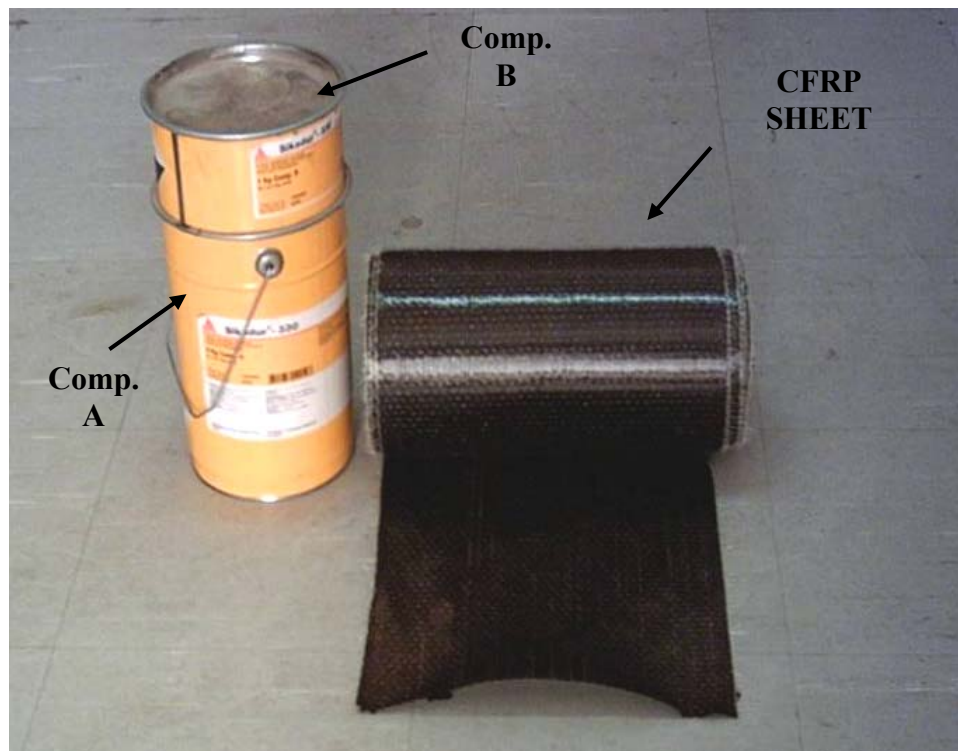


Figure 3.3 CFRP sheet and epoxy.

3.5 Application of CFRP Composite System to Existing Reinforced Concrete Element

The procedure that was used in applying CFRP composite system is summarized below. These steps were followed according to the recommendation of CFRP manufacturer and ACI committee 440.

3.5.1 Preparation of Concrete Surface

- Grinding the concrete surface by scraper machine at the location of gluing the CFRP on the concrete to remove the weak surface.
- Rounded the edge of the beam (approximately 15 mm) to prevent stress concentration in the CFRP sheets.
- Clean and remove the dust from the concrete surface by water jutting.

3.5.2 CFRP System Application

- Mix part A and B together according to manufacturer technical data by slow speed electrical drill till it becomes grey color.
- Apply a thin layer of mixed epoxy on the concrete surface (approximately 1.5 mm) this will impregnate the carbon fiber sheet after they are placed on the concrete element.
- Apply a coat of mixed epoxy on the clean carbon fiber sheet.
- Place carbon fiber sheet on the concrete surface over the area that was previously coated with epoxy.
- Use a ribbed roller with direction of the sheet to remove air bubbles that are trapped behind the carbon fiber sheet.

Figure 3.4 shows the procedure of application CFRP system on concrete element.



Figure 3.4 application of CFRP system on concrete element

3.6 Instrument and Test Procedure

3.6.1 Supporting and Loading Condition

All beams were tested up to failure as simply supported by four 30 mm diameter steel roller. A steel bearing plate of (100 *100 *20 mm) inserted between the concrete and steel roller to prevent local failure at the supporting and loading points.

A lateral supports were provided to each side of beam to prevent lateral movement as shown in Figure 3.5.

3.6.2 Instrument

A hydraulic machine of 2000 kN capacity was used in Civil Engineering Department of AL-Kufa University as shown in Figure 3.5. The deflection corresponding to the applied load was measured at every load step by a dial gage of accuracy 0.01 mm installed at the mid span of the beam. While the strain of concrete and CFRP sheets were measured by an ELE extensometer of accuracy 0.002 mm. The inclined crack width measured by crack meter.

3.6.3 Test Procedure

All the beams were painted with white color to observe the crack development. At the first the specimens loaded by 5 kN to seat the support and the load system, then reduced to zero. The load increment was 5 kN along the test. Eight pair of demic used at the center of the beam to measure the strain of concrete over the depth of the specimens, and CFRP strain was measured at critical location. The strain at the concrete and CFRP sheet was recorded at several load stage.



Figure 3.5 Test set-up of beam specimen R-1

Structure laboratory/ College of Engineering

Al-Kufa University

CHAPTER FOUR

EXPERIMENTAL RESULTS AND DISCUSSIONS

4.1 Introduction

In this chapter, the general behavior and observation of tested beams were reported and discussed. The experimental variable includes five strengthening models by CFRP sheet. Eight beams are tested to failure and these beams analyzed according to mid-span deflection, crack pattern and the ultimate load.

4.2 Control Beams

➤ Specimen C-1

In spite of low flexural reinforcement ratio (0.6%), it is less than 1% the specimen failed by inclined cracks joining the load and support points.

The first visible cracks are narrow flexural cracks in the mid-span region at about 180 kN. When the applied load reached to 240 kN, a sudden inclined crack propagate from the support with harsh noise upward to the point load. With increase the applied load the inclined crack become wider and propagates rapidly. Some of small shear cracks merged and developed into the inclined crack. Then the collapse happened by splitting the beam into two pieces by a new inclined shear crack parallel to the initial one without any warning at load of 535 kN as shown in Figure 4.1.

The measured cracks may be called service cracks or visible cracks because it measured when they visible. Therefore, the cracks may be occurs before this stage of loading.



Figure 4.1 Crack pattern of beam C-1 at failure

The load deflection curve for specimen (C-1) is shown in Figure 4.2 and Figure 4.3 shows the width of inclined crack.

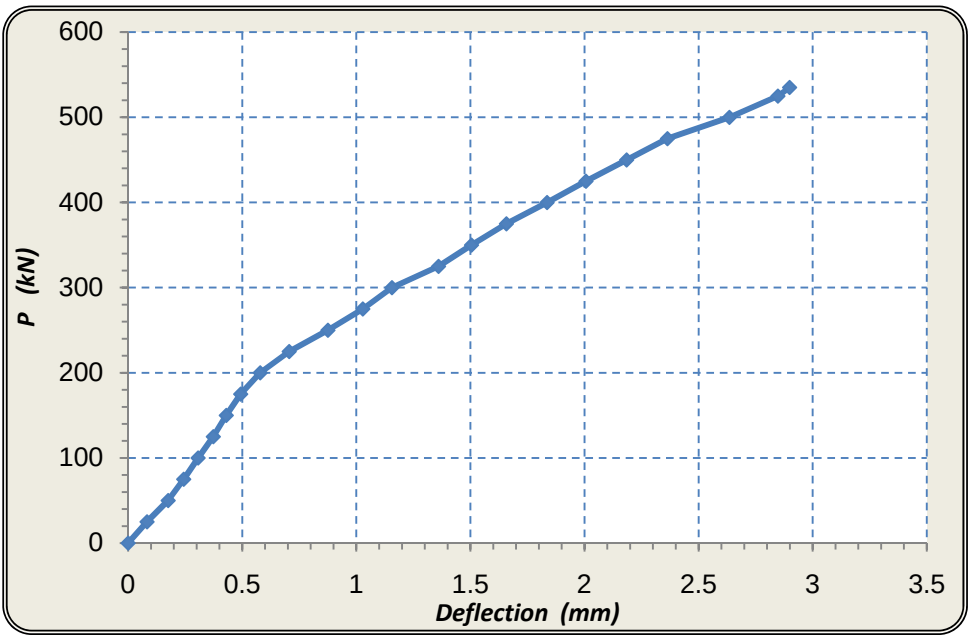


Figure 4.2 Load-Deflection curve for control beam C-1

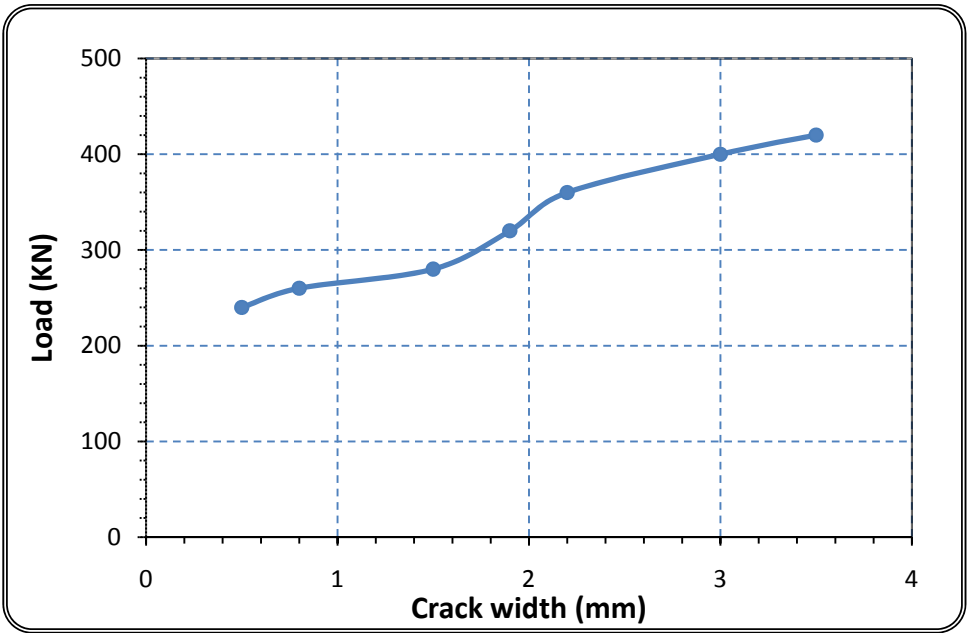


Figure 4.3 Crack width versus applied load for control beam C-1

➤ Specimen C-2

In specimen C-2, (control unstrengthened deep beam with opening) cracks are first observed around the corners of the openings at load about 110kN, these cracks appeared due to stress concentration at the opening's corner. As the load increase these cracks propagate and widen rapidly towards supporting and loading points. A new shear cracks through the shear span are merged with the initial one. Some limited flexural cracks are noticed at load about 115 kN. Then, the beam failed by abrupt development of the cracks at the corners at load of 155 kN as shown in Figure 4.4.

It can be concluded that the opening in C-2 reduces the shear capacity when compared with that of specimen C-1 by about (71%), which reflects the effect of web openings and their location on the behavior of R.C. deep beams. Therefore, the zone of the opening will be strengthened by using CFRP laminate.

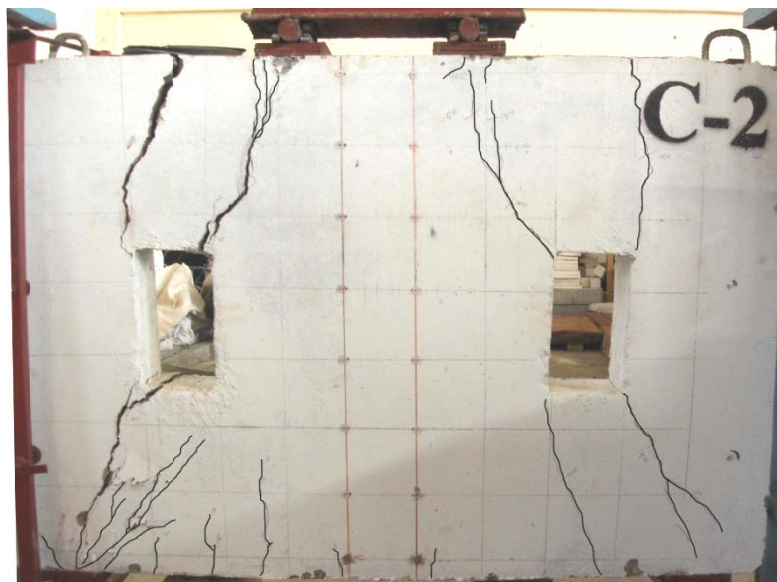


Figure 4.4 Crack pattern of beam C-2 at failure

Figure 4.5 shows comparison of load-deflection curves for specimen C-1 and C-2.

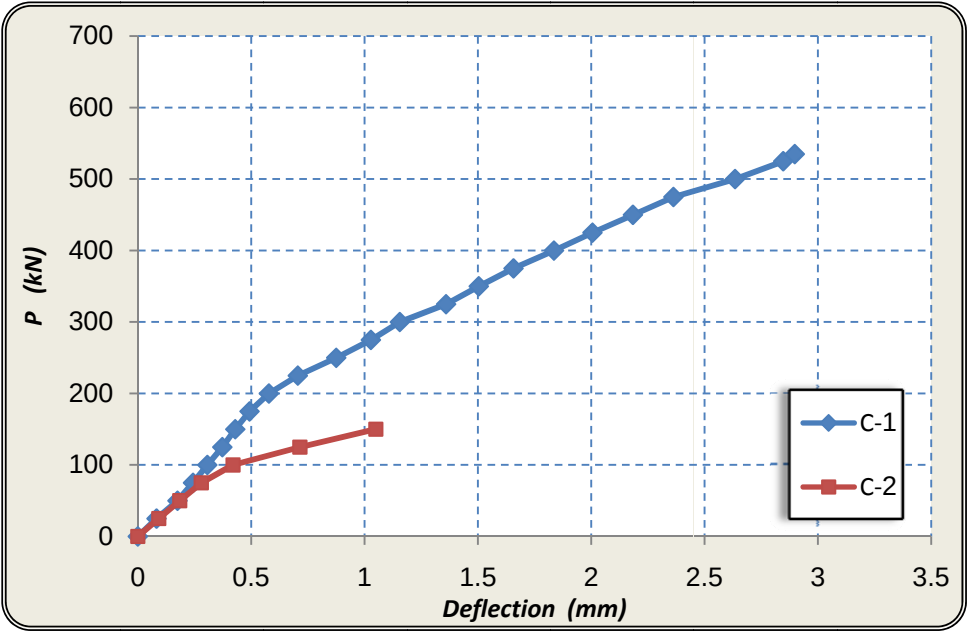


Figure 4.5 Load-Deflection curve for controls beam C-1 and C-2

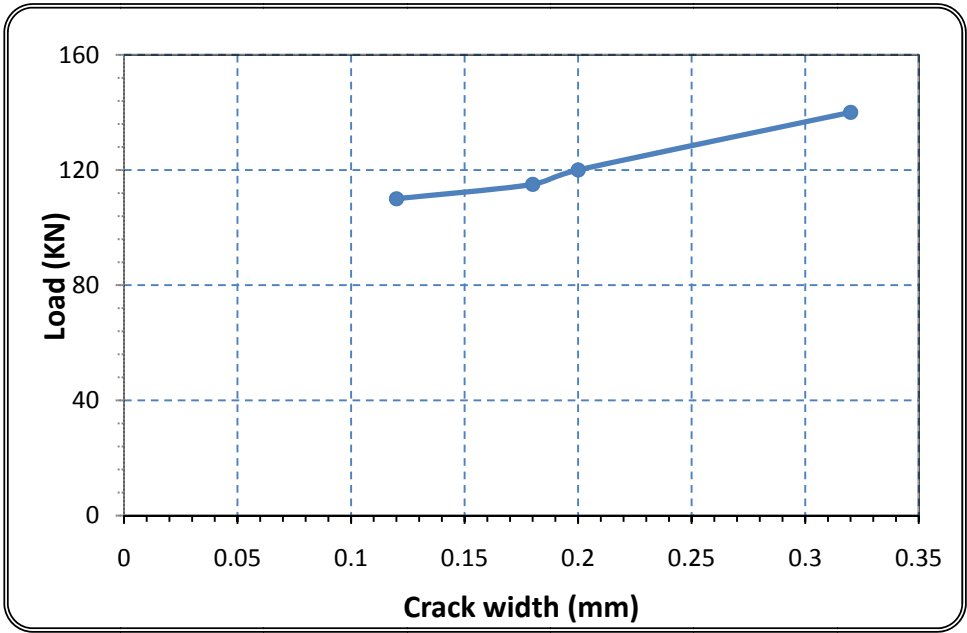


Figure 4.6 Crack width versus applied load for control beam C-2

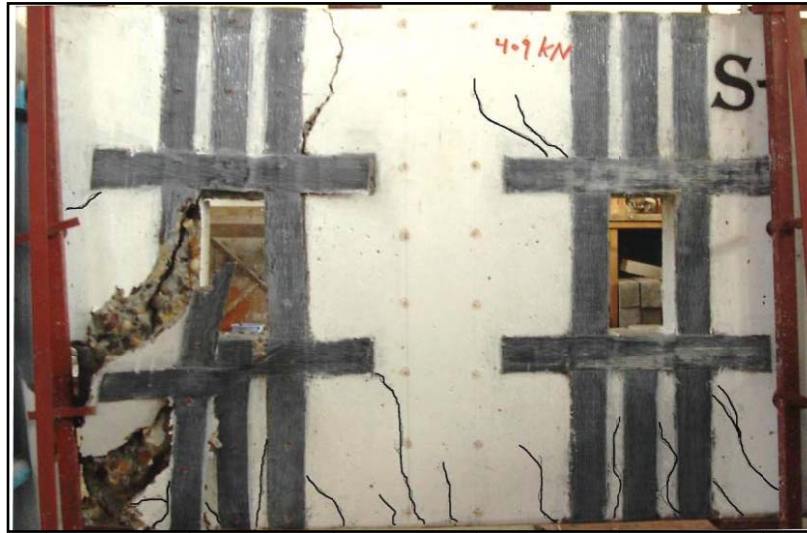
2.3 Strengthened Beams

➤ Specimens S-1, S-2 and S-3

The strengthening systems S-1(full wrap), S-2(U-wrap) and S-3 (two face wrap) were tested and shows approximately similar behavior up to failure.

The first visible cracks are inclined cracks near the corners of openings at approximately 200 kN (50-55% of the ultimate load). As the applied load process the inclined cracks propagate upward to support and loading point, while the shear cracks at the shear span growth beside and parallel to CFRP straps and it makes an effort to cross the CFRP straps. Limited flexural cracks noted in the mid span region at approximately same load level (50-55% of ultimate load). Some of these cracks change their direction to the nearest corner of opening. The horizontal stabilizer straps are deboned from the concrete surface before the failure. The beams failed by a sudden split of the concrete above and below the openings due to rapidly crack propagation from the opposite corners to support and load points as shown in Figure 4.7. A peel of concrete splitting with CFRP straps. In all these systems the vertical strips did not deboned.

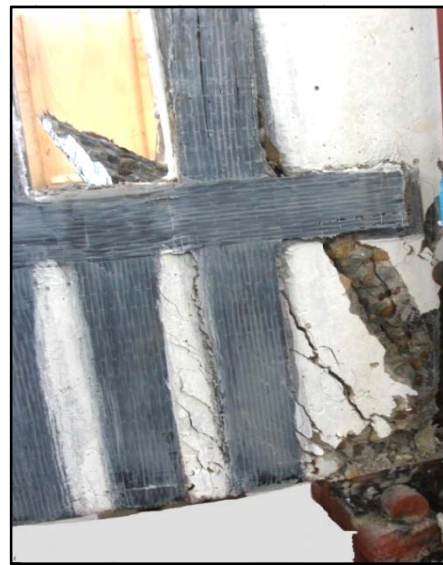
It can be deducted from the behavior of (S-1, S-2 and S-3) as shown in Figure 4.8 that the ultimate load increased approximately 160% with respect to specimen C-2. Therefore, this result gives good induction about using the CFRP in improvement the shear capacity of the beam with web opening. At the same time, the comparison between the three models (full wrap, U-wrap and two face wrap) shows no important effect of the vertical anchorage and this may be due to the enough development length glued on the concrete surface.



(a)

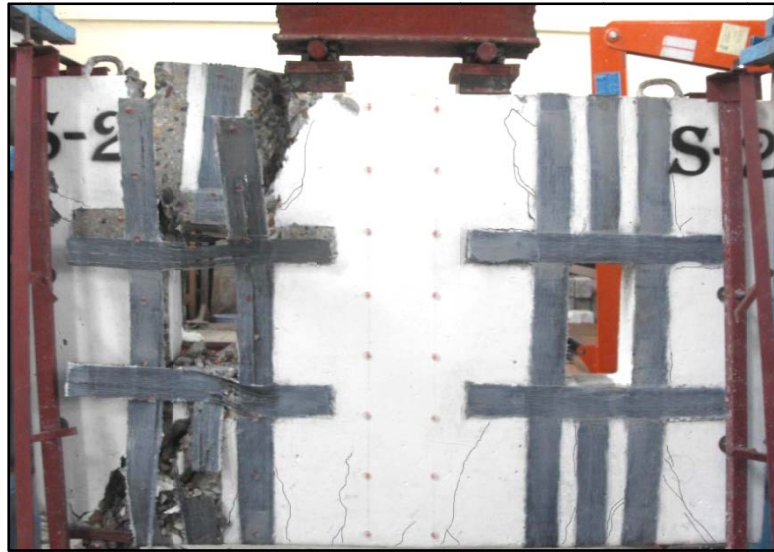


(b)

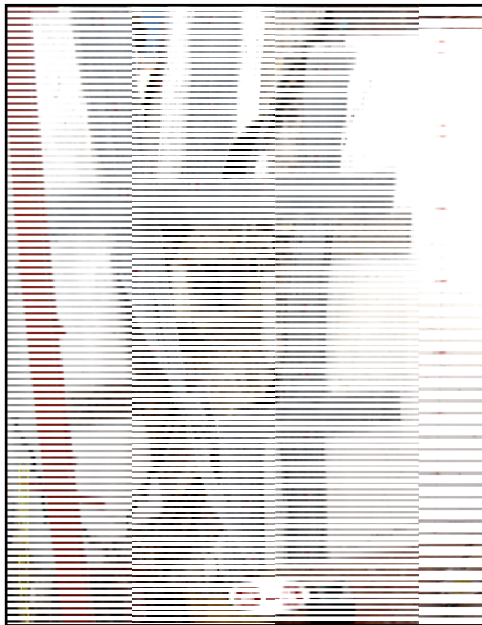


(c)

Figure 4.7 shows (a) crack pattern of specimen **S-1**, (b) peel of concrete splitting with CFRP straps, (c) distribution the cracks between the straps.



(d)



(e)



(f)

Figure 4.7 continued (d) Crack patterns of S-2 at failure, (e and f) crack pattern of specimen S-3 at failure

Figure 4.8 and 4.9 show the load-deflection curves and development of crack width through the applied load for the strengthened system (S-1, S-2 and S-3).

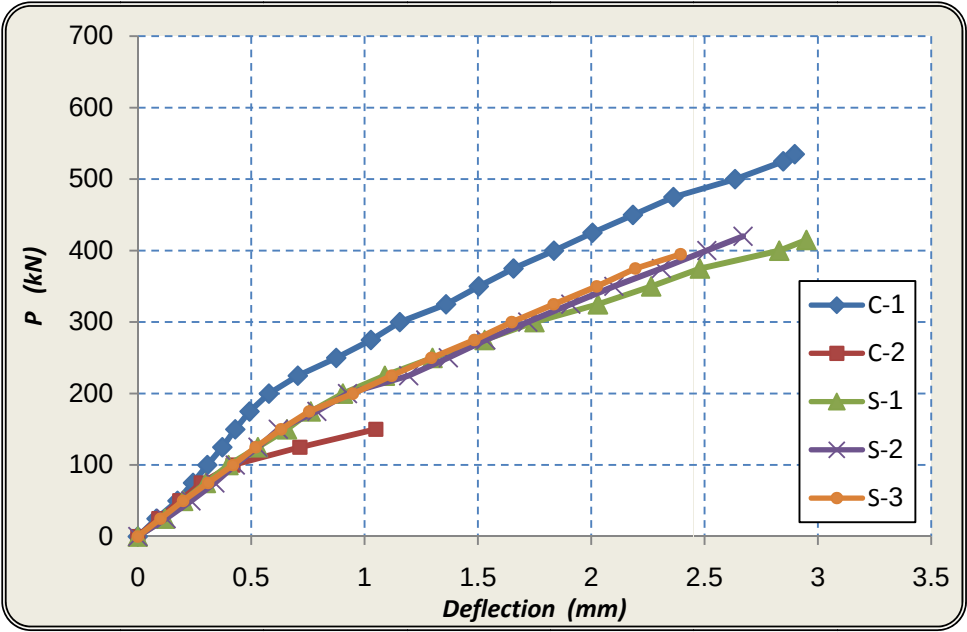


Figure 4.8 Load-Deflection curve for beams C-1, C-2, S-1, S-2 and S-3

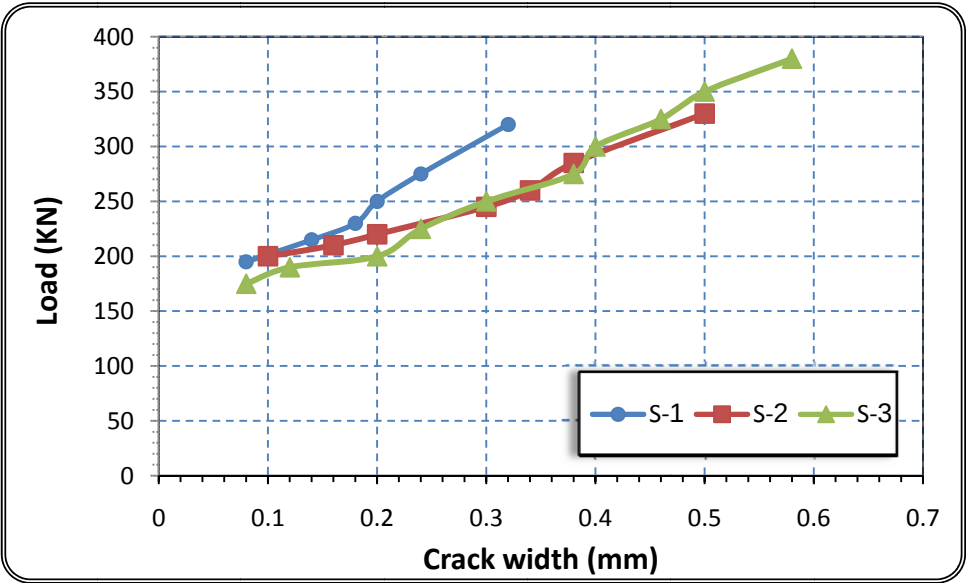


Figure 4.9 Crack width versus applied load for beams S-1, S-2, S-3

➤ Specimen S-4

In specimen S-4 the cracks appear earlier than the previous strengthening models, where the first visible cracks noted near the support at load 120 kN, and the shear cracks propagate quickly through the mid zone between the two vertical straps. As the applied load reach 210 kN flexural cracks are observed, while the horizontal and diagonal straps started delaminating from the concrete surface at approximately 250 kN. Then the failure accrued due to rapidly propagate the crack to the opposite corners of the support and loading points at 315 kN as shown in Figure 4.10. It is expected that this specimen will give the best results. The interpretation of this behavior that no enough area of concrete surface to glue the CFRP straps, and no middle strap provided between the two straps that give the cracks freedom in propagate through the shear zone, that main the distribution of straps play very important role to hamper the propagation of cracks.



Figure 4.10 Crack patterns of specimen S-4 at failure

This model of strengthening increased the ultimate load approximately 100% as shown in Figure 4.11.

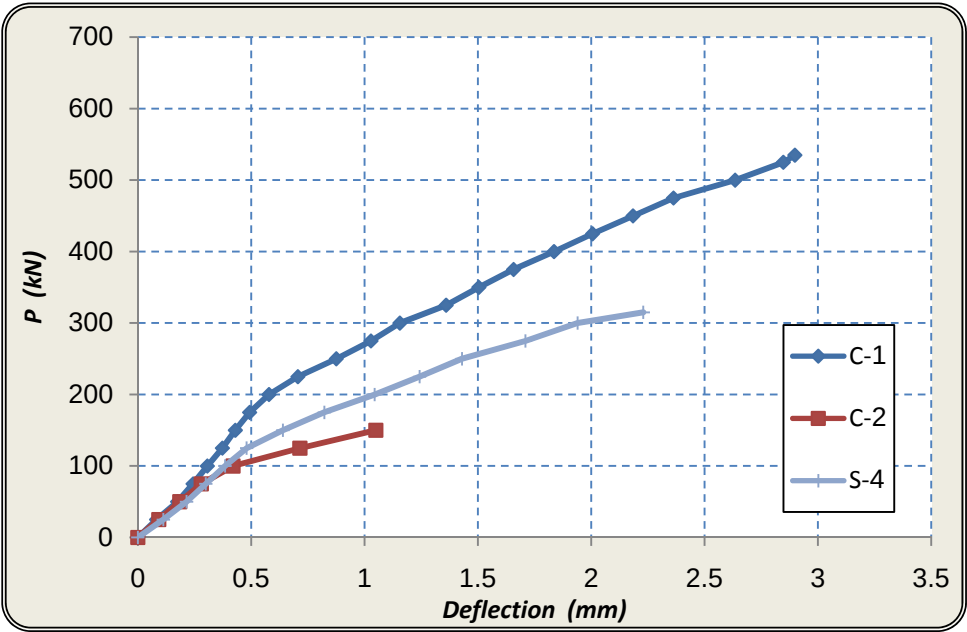


Figure 4.11 Load-Deflection curve for beams C-1, C-2 and S-4

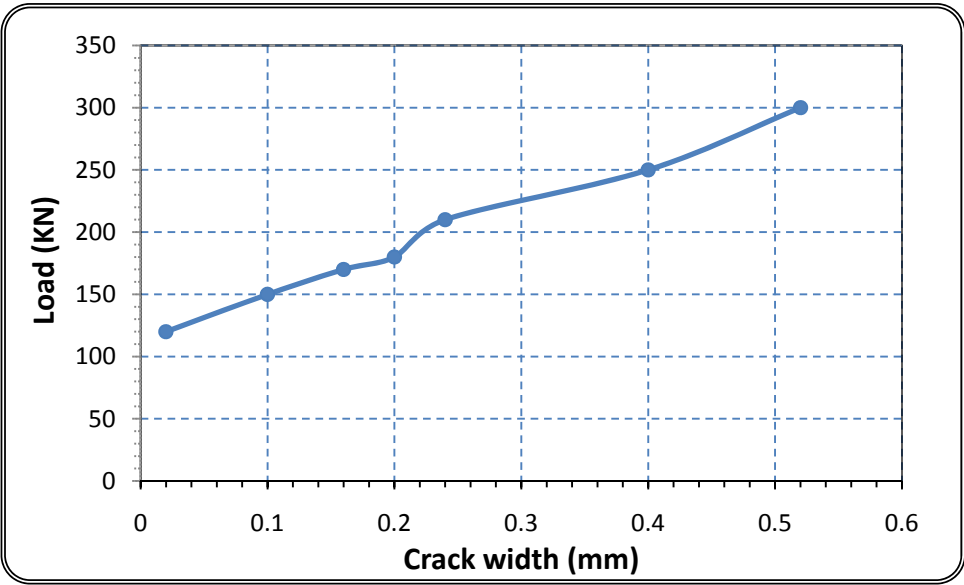


Figure 4.12 Crack width versus applied load for beams S-4

➤ **Specimen S-5**

Specimen S-5 strengthening with CFRP straps orthogonal on the expected crack pattern. The first visible crack is a diagonal crack at the support at approximately 160 kN. Through the applied load process the cracks distribution through the spacing between the straps and flexural cracks appear at 180 kN. Then, the failure happens by sudden diagonal cracks from the opposite corners to support and point loads. The CFRP straps are delaminating from the concrete surface with peel of concrete at 450 KN as shown in Figure 5.13a, b, and c.

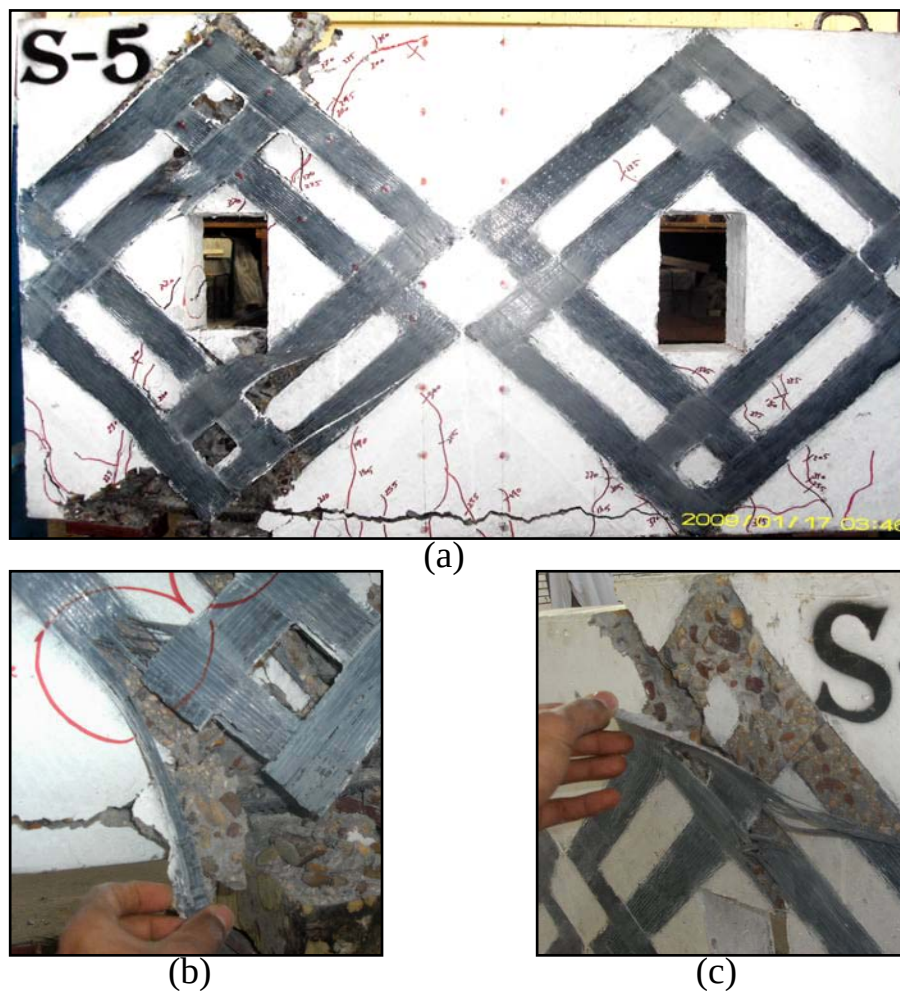


Figure 4.13 Crack patterns of specimen S-5 at failure

Figure 4.14 and 4.15 show the load-deflection curve and development of crack width with applied load for specimen S-5.

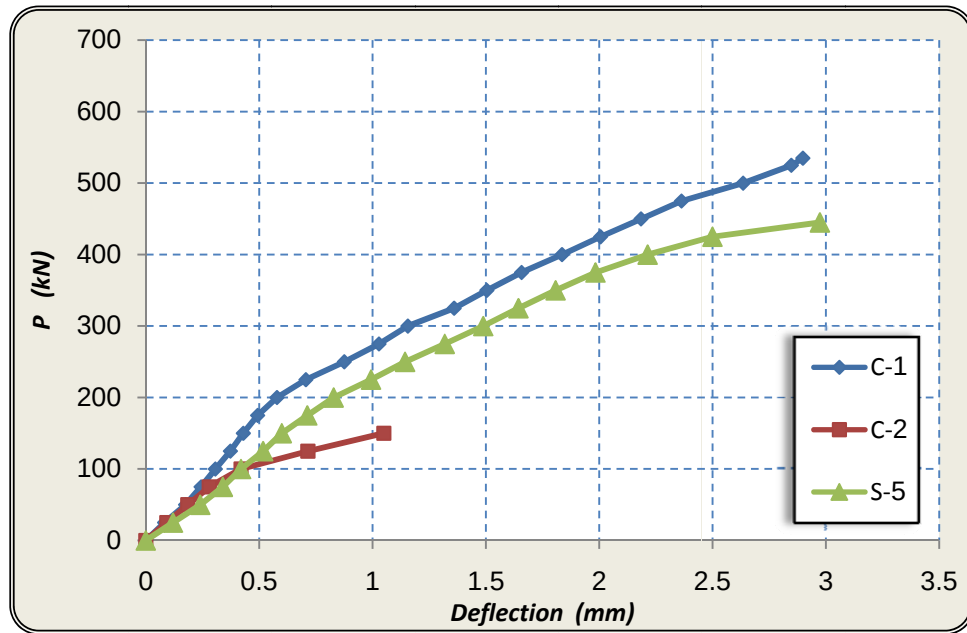


Figure 4.14 Load-Deflection curve for beams C-1, C-2 and S-5

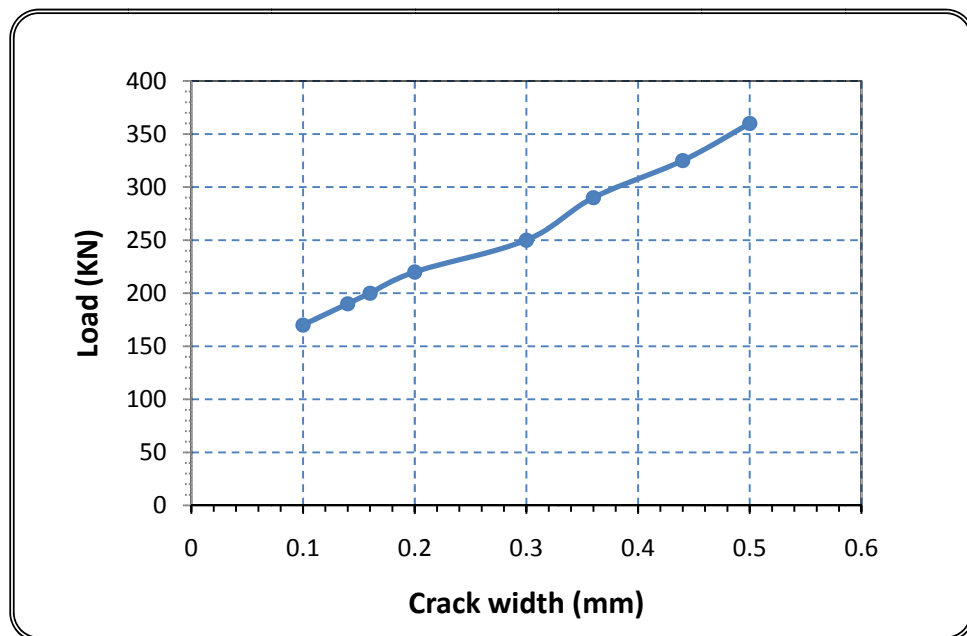


Figure 4.15 Crack width versus applied load for beams S-5

➤ Specimen R-1

It is decided to loaded R-1 approximately to cracking load (110 kN) according to control beam C-2(cracking load), then strengthened by S-5 system because it the best system. The test results of specimen R-1 give similar behavior of specimen S-5, but less stiffness than specimen S-5 due to precracking.



(a)



(c)



(d)

Figure 4.16 (a) Crack patterns of specimen **R-1**, (b) and (c) the peel of concrete with CFRP straps

Figure 4.18 and 4.19 show the load-deflection curve and development of crack width with applied load for specimen R-1.

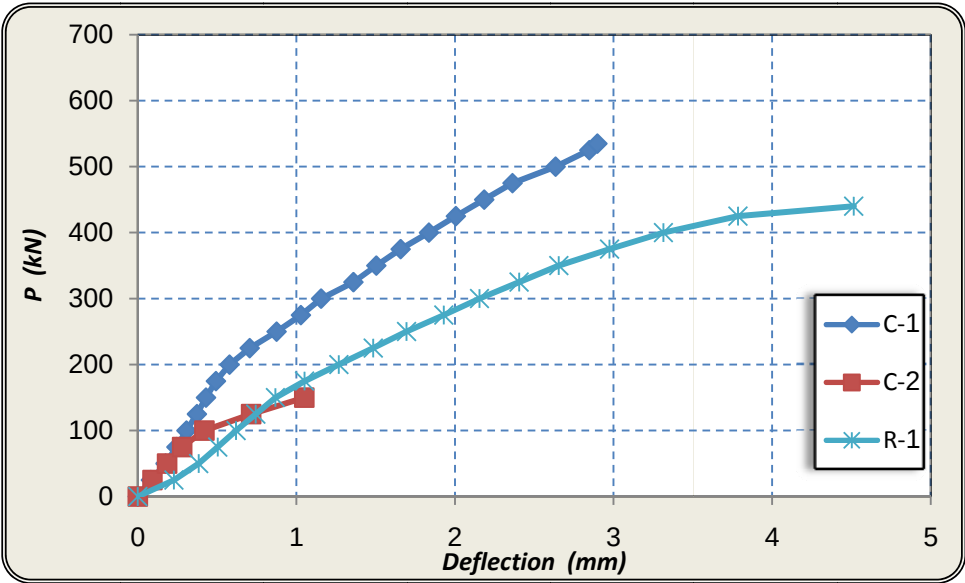


Figure 4.18 Load-Deflection curve for beams C-1, C-2 and R-1

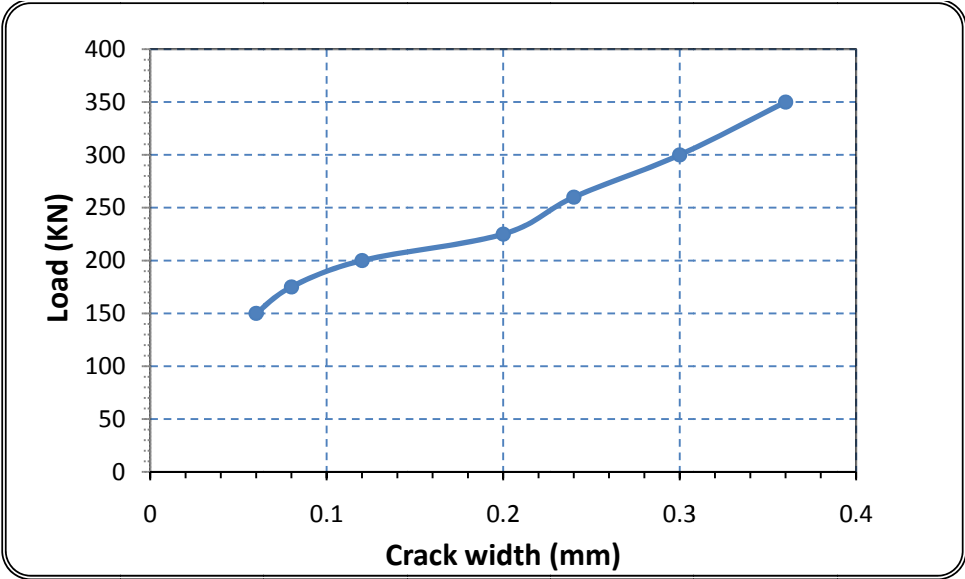


Figure 4.19 Crack width versus applied load for beams R-1

Figure 2.20 and 2.21 shows summary of load-deflection curves and crack width for all tested specimens.

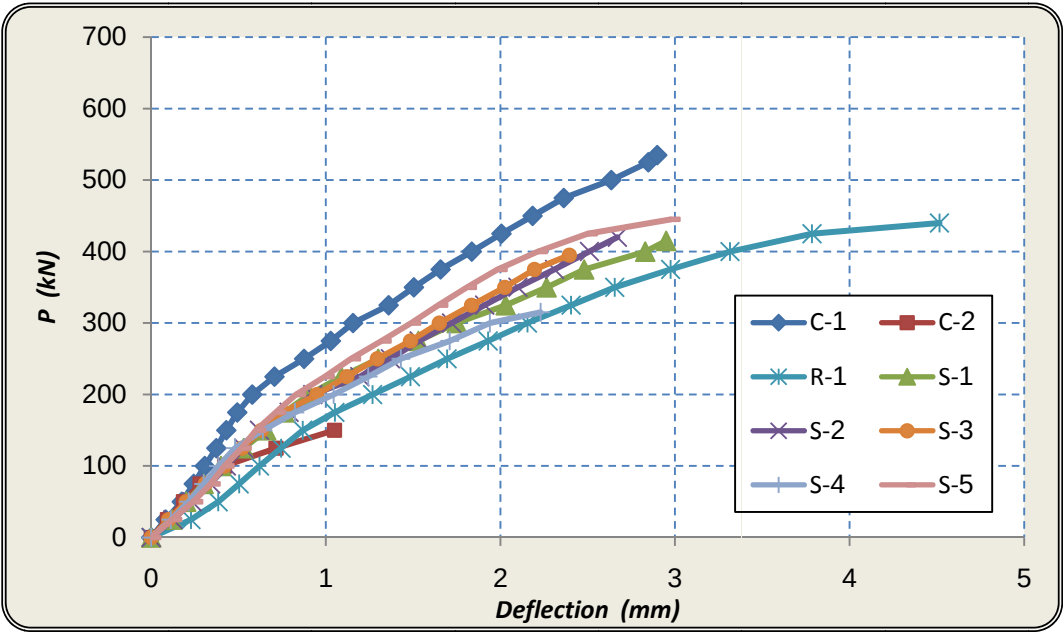


Figure 4.20 Load-Deflection curve for all tested beams

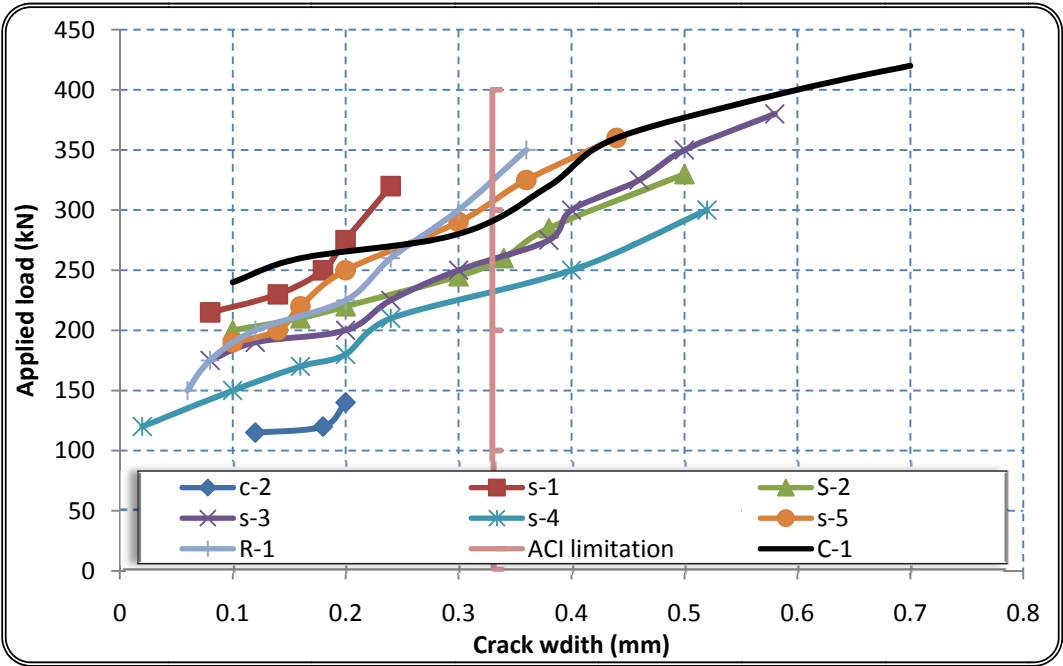


Figure 4.21 crack width for all tested beams

Table 4.1 shows the cracking, failure loads and the increase in the cracking and ultimate loads with respect to the control beam C-2 for all the test beams as a summary of results obtained.

Table 4.1 Summary of Tested Beams

Beam symbol	Cracking load(kN)	Failure load (kN)	Cracking load increase %	Failure load increase %
C-1	240	535	-----	-----
C-2	110	155	-----	-----
S-1	180	410	63.6	165 [*]
S-2	179	415	62.7	167
S-3	175	395	59	155
S-4	120	315	9	100
S-5	160	450	45	190
R-1	120	440	-----	184

$$* = \frac{P_{u(S-1)} - P_{u(C-2)}}{P_{u(C-2)}} * 100$$

CHAPTER FIVE

FINITE ELEMENT AND MATERIAL PROPERTIES MODELING

5.1 Introduction

In any three-dimensional finite element analysis, the performance of any structural member under load depends on the behavior of the type of material used to construct the member. In concrete members which are made of different materials, concrete and reinforcing bars are brought together to behave as a composite system. The steel can be considered as a homogeneous material that exhibits a similar stress-strain relationship in tension and compression. While, the behavior of concrete is monitored to have grossly heterogeneous internal structure because it is dependent on the properties of each of its components; namely, cement mortar, aggregates and air voids.

One of the main objectives of the finite element analysis of structures is to determine the response of the structure under loading. A typical load-deformation response for a monotonically loaded member is essentially linear up to a certain limit of load. Beyond this limit a nonlinear load-deformation response occurs. Such response is due to a combination of nonlinear material behavior (material nonlinearities), large deformation in the structure (geometric nonlinearities), and interface nonlinearity for composite members (changing status).

In the analysis of reinforced concrete structures, the effect of large deformations can be neglected for the majority of cases. This is due to the early onset of material nonlinearity, with large deformations that

occurring only close to the structural collapse, especially in composite concrete beam where slippage and separation between different materials may become noticeable. At nonlinear stage of behavior, it is not possible to solve the governing equilibrium equations directly; therefore, resort has to be made to more sophisticated solution strategies. In the present work, ANSYS computer program (**AN**alysis **SY**stem) is used to create the finite element model.

5.2 Finite Element Formulation

The three-dimensional finite element analysis is represented by a finite number of elements and nodes. To drive the element equation, the principle of virtual work is usually used [52]. This principle states, "A virtual change of the internal energy must be equal to the identical change in external work due to the applied loads".

$$W_{int.} = W_{ext} \dots \dots \dots (5.1)$$

Where:

$W_{int.}$ = internal work (strain energy)

$W_{ext.}$ = external work (by applied forces)

The virtual internal work is:

$$W_{int} = \int_v \{\partial \varepsilon\}^T \{\sigma\} \cdot dv \dots \dots \dots (5.2)$$

Where:

$\{\partial \varepsilon\}$ = elements of virtual strain vector = $[\partial \varepsilon_x \ \partial \varepsilon_y \ \partial \varepsilon_z \ \partial \varepsilon_{xy} \ \partial \varepsilon_{yz} \ \partial \varepsilon_{xz}]^T$

$\{\sigma\}$ = elements of real stress vector = $[\sigma_x \ \sigma_y \ \sigma_z \ \sigma_{xy} \ \sigma_{yz} \ \sigma_{zx}]^T$

V = volume of the element

By using the general stress-strain relationship, stresses $\{\sigma\}$ can be determined from the corresponding strain $\{\varepsilon\}$ as:

$$\{\sigma\} = [D] \cdot \{\varepsilon\} \dots \dots \dots (5.3)$$

Where $[D]$ is the constitutive matrix

After substituting Equation (5.3) into (5.2), the virtual internal work can be written as:

$$W_{int} = \int_v \{\partial \varepsilon\}^T [D] \{\varepsilon\} dv \quad \dots\dots\dots (5.4)$$

The displacements $\{U\}$ within the element are related by interpolation to nodal displacements $\{a\}$ by:

$$\{U\} = [N] \cdot \{a\} \quad \dots\dots\dots (5.5)$$

Where:

$[N]$ = shape function matrix

$\{a\}$ = unknown nodal displacements vector (local displacements)

$\{U\}$ = body displacements vector (global displacements).

By differentiating Equation (5.5), the strains for an element can be related to its nodal displacements by:

$$\{\varepsilon\} = [B] \{a\} \quad \dots\dots\dots (5.6)$$

Where

$[B]$ = strain-nodal displacement relation matrix, based on the element shape functions

Assuming that all effects are in the global Cartesian system, and then combining Equation (5.6) with Equation (5.4) yields [53]:

$$W_{int} = \{\partial a\}^T \int_v [B]^T [D] [B] dv \{a\} \quad \dots\dots\dots (5.7)$$

The external work, which is caused by the nodal forces applied to the element, can be accounted for by:

$$W_{ext.} = \{a\}^T \cdot \{F\} \quad \dots\dots\dots (5.8)$$

Where:

$\{F\}$ = nodal forces applied to the element

Finally, Equations (5-1), (5-7) and (5-8) may be combined to give:

$$\{\partial a\}^T \int_v [B]^T [D][B] dv \{a\} = \{\partial a\}^T \{F\} \dots\dots\dots (5.9)$$

Noting that $\{\partial a\}^T$ vector is a set of arbitrary virtual displacements, the condition required to satisfy Equation (5.9) can be reduced to:

$$[K^e] \cdot \{a\} = \{F\} \dots\dots\dots (5.10)$$

Where:

$$[K^e] = \int_v [B]^T [D][B] dv \dots\dots\dots (5.11)$$

$[K^e]$ = element stiffness matrix

The elemental volume in the global coordinate (dx, dy and dz) can be written in term of local coordinates (dξ, dη, and dζ). So, the equation 5.12 represents the element stiffness matrix.

$$[K^e] = \int_{-1}^{+1} \int_{-1}^{+1} \int_{-1}^{+1} [B]^T [D][B] |J| d\xi d\eta d\zeta \dots\dots\dots (5.12)$$

The simple integration limits should be noted. These integrals may be evaluated by the numerical integration technique described in Sec. 5.4.

Equation (5.10) represents the equilibrium equation on a one-element basis. For all elements, the overall stiffness matrix $[K]$ of the structure is built up by adding the element stiffness matrices (adding one element at a time), after transforming the local to the (overall) global coordinates, this equation can be written as:

$$[K]\{a\} = \{F^a\} \dots\dots\dots (5.13)$$

Where:

$[K] = \sum_n [K^e]$ = overall structural stiffness matrix

$\{F^a\} = \{F^{nd}\}$ = vector of applied loads (total external force vector)

n = total number of elements

5.3 Material Idealization

5.3.1 Concrete Idealization

SOLID65 was used to model the concrete. The solid element has eight nodes with three degrees of freedom at each node translation in x, y and z directions. The SOLID65 is capable of cracking in tension and crushing in compression.

The most important aspect of this element is the treatment of nonlinear material properties. The concrete is capable of cracking (in three orthogonal directions), crushing, plastic deformation, and creep. The rebars are capable of tension and compression, but not shear. They are also capable of plastic deformation and creep. This 8-node brick element is used, in this study, to simulate the behavior of concrete. The element is defined by eight nodes and by the isotropic material properties [53]. The geometry, node locations, and the coordinate system for this element are shown in Figure 5.1.

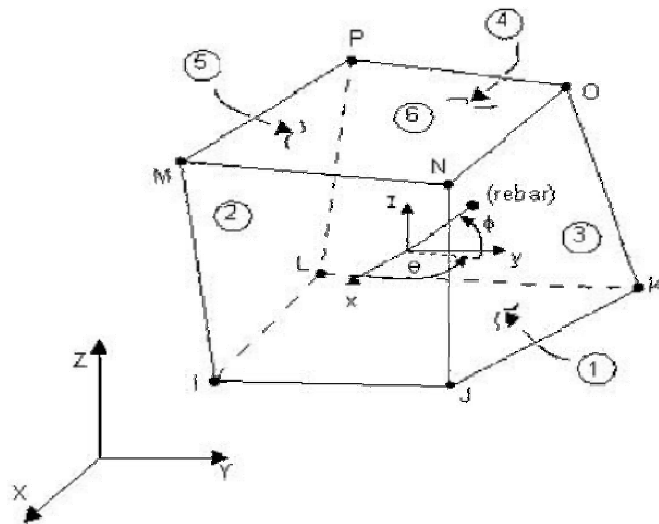


Figure 5.1 SOLID65 Geometry [53]

5.3.2 Reinforcement Idealization

LINK8 is a spar (or truss) element which may be used in a variety of engineering applications. This element can be used to model trusses, sagging cables, links, springs, etc. The 3-D spar element is a uniaxial tension-compression element with three degrees of freedom at each node translations of the nodes in x, y, and z-directions. As in a pin-jointed structure, no bending of the element is considered. Plasticity, creep, swelling, stress stiffening, and large deflection capabilities are included. This element is used, in this study, to simulate the behavior of steel reinforcement that works as a main steel reinforcement in resisting the flexural stresses [53]. The geometry, node locations, and the coordinate system for this element are shown in Figure 5.2.

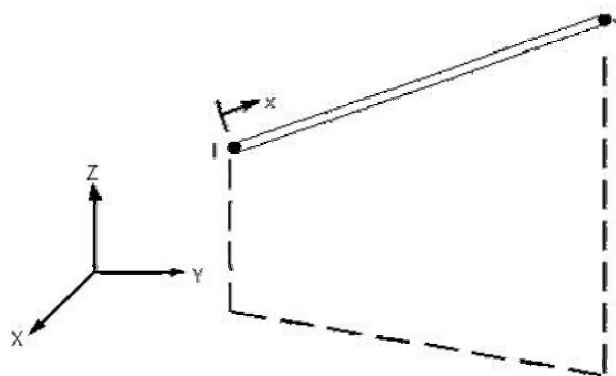


Figure 5.2 LINK8 Geometry [53]

5.3.3 Steel Plate Idealization (Supports)

An eight node solid element, SOLID45, was used for steel supports in the beam models. The element is defined with eight nodes having three degrees of freedom at each node translations in x, y and z directions [53]. The geometry and node location for this element type are shown in Figure 5.3.

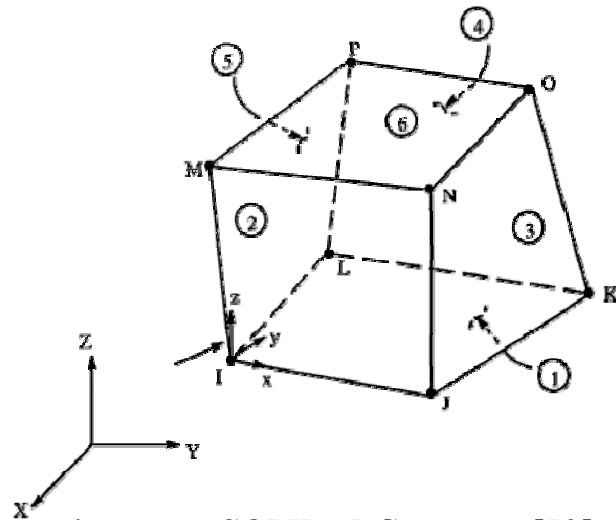


Figure 5.3 *SOLID45 Geometry [53]*

5.3.4 CFRP Idealization

A four node SHELL 41 element was used to model CFRP strips and. SHELL41 is a 3-D element having membrane (in-plane) stiffness but no bending (out-of-plane) stiffness. It is intended for shell structures where bending of the elements is of secondary importance. The element has three degrees of freedom at each node translations in the nodal x , y , and z directions. The element can have variable thickness, stress stiffening and large deflection [53]. The geometry and node location of this element type are shown in Figure 5.4.

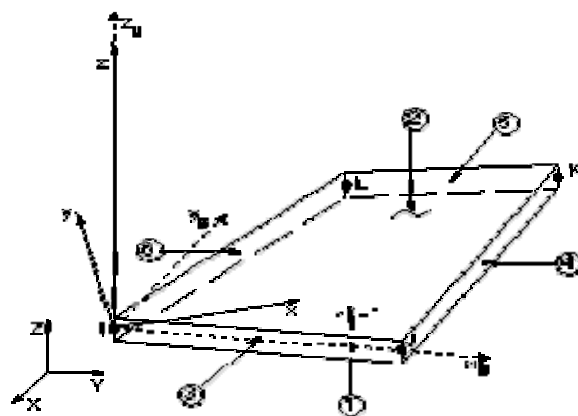


Figure 5.4 *SHELL41 Geometry [53]*

5.4 Numerical Integration

In most finite element analyses, the element stiffness matrix $[K^e]$ cannot be obtained analytically. Thus, to perform the integration required to evaluate the element stiffness matrix, a suitable scheme of numerical integration has to be used.

The application of the three-dimensional finite element analysis in connection with the nonlinear behavior of structures needs a large amount of computation time due to frequent evaluation of the stiffness matrix. Therefore, it is necessary to choose a suitable integration rule that minimizes the computation time but with sufficient accuracy.

In the current study, the “Gauss-Legendre quadrature scheme” is used and it has been found to be accurate and efficient [54]. Hence, the equations of the stiffness matrix element in two dimensions can be expressed as:

$$I = \int_{-1}^{+1} \int_{-1}^{+1} F(\xi, \eta) \cdot d\xi \cdot d\eta \approx \sum_{i=1}^n \sum_{j=1}^m W_i W_j F(\xi_i, \eta_j) \quad \dots\dots\dots (5.14)$$

When extended to three-dimensional cases, the equations of the stiffness matrix element are expressed as [53][55].

$$I = \int_{-1}^{+1} \int_{-1}^{+1} \int_{-1}^{+1} F(\xi, \eta, \zeta) \cdot d\xi \cdot d\eta \cdot d\zeta \approx \sum_{i=1}^n \sum_{j=1}^m \sum_{k=1}^l W_i W_j W_k F(\xi_i, \eta_j, \zeta_k) \quad \dots\dots (5.15)$$

Where: n , m , and l are the number of Gaussian points in the ξ , η and ζ - directions respectively.

In general, " n , m , and l " are taken to be equal in three directions. $F(\xi, \eta, \zeta)$ is a function which represents the matrix multiplication “ $[B]^T [D] [B] |J|$ ”.

W_i , W_j and W_k are weighting factors, see Table 5.1.

In a similar manner the reinforcing bar element stiffness matrix can be written as:

$$I = \int_{-1}^{+1} F(\xi) \cdot d\xi \approx \sum_{i=1}^n W_i \cdot F(\xi_i) \quad \dots\dots\dots (5.16)$$

The integration rule, which has been used in present work, is $2 \times 2 = 4$ points for the shell element, and $2 \times 2 \times 2 = 8$ point Gauss quadrature for the brick element and $1 \times 2 = 2$ for reinforced bar. Figure 5.5 show the distribution of the sampling points over the volume of the 8 node and their weighting factors are given in Table 5.1. The integration points are also the sampling points for stresses and material state determination [53][56].

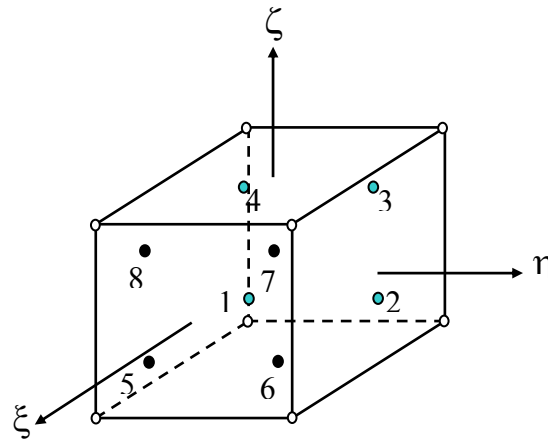


Figure 5.5 Distribution of integration points[53]

Table 5.1 Sampling Points Locations and Weighting Factors for (2x2x2), (2x2) and (2x1) Gauss Quadrature [53], [56]

Number of Sampling Points	Local Coordinate			Weight
	ξ	η	ζ	
1-8	± 0.57734	± 0.57734	± 0.57734	1.0
1-4	± 0.57734	± 0.57734		1.0
1-2	± 0.57734			1.0

5.5 Nonlinear Solution Techniques

The use of finite element discretization in nonlinear structural problems results in a set of nonlinear algebraic equations of the form:

$$[k] \{a\} = \{F^a\} \quad \dots\dots\dots (5.13)$$

The solution for the equation of a linear elastic structural problem can be obtained directly. But in nonlinear problems, a direct solution is no longer possible since the stiffness matrix $[K]$ depends on the displacement level [53]. Therefore, it cannot be exactly calculated before the determination of the unknown nodal displacements $\{a\}$. For the solution of a nonlinear structural problem, the state of equilibrium of the structural system corresponding to the applied load must be found, and the equilibrium equations can be expressed as:

$$\{R\} = \{F^{in}\} - \{F^a\} \quad \dots\dots\dots (5.17)$$

Where: $\{R\}$ = out of balance forces vector, $\{F^{in}\}$ = internal nodal forces vector

$$\text{and} \quad \{F^{in}\} = \int_v [B]^T \{\sigma\} \cdot dv \quad \dots\dots\dots (5.18)$$

The internal forces also depend on the displacement level, and they have to be approximated in successive steps until Equation (5.17) is satisfied within acceptable tolerance. Several techniques exist for solving the nonlinear Equation (5.13) and they can be classified into:

- Iterative Techniques.
- Incremental Techniques.
- Incremental-Iterative Techniques.

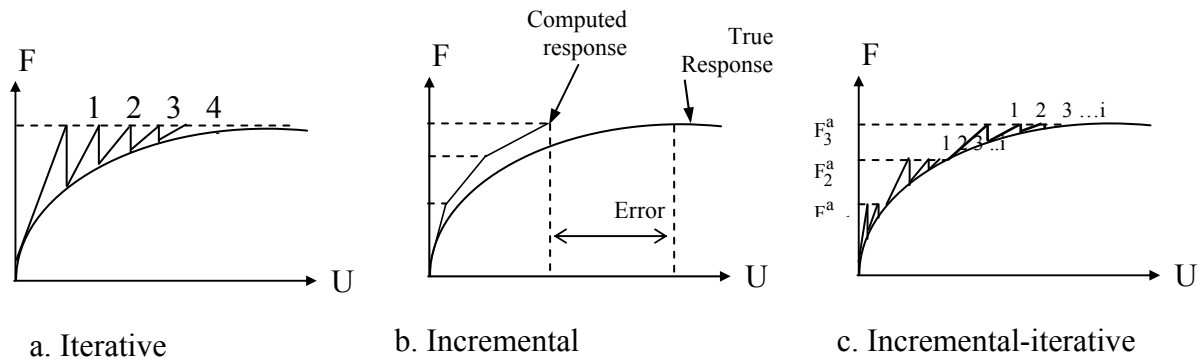


Figure 5.6 Basic Techniques for Solving Nonlinear Equations [52]

► Iterative Techniques

The purely iterative techniques imply the application of the total load in a single increment, as shown in Figure 5.6(a). The out of balance force is used as an additional load. The total displacement is taken as the sum of the accumulated displacements from each iteration. The iterative corrections continue until the out of balance forces become negligibly small [58]. This type of technique is not suitable for tracing the entire nonlinear equilibrium path because it fails to produce information about the intermediate stages of loading [53]. However, this requirement can be achieved by using the combined incremental-iterative method.

► Incremental Techniques

In this method, the nonlinear behavior is determined by solving a sequence of linear problems, and the load is applied in a several small increments, Figure 5.6(b). The structure is assumed to respond linearly within each increment with recomputed (or updated) stiffness. This is a simple procedure, which requires no iterations within each load increment, but errors are accumulated after several increments. So, it is necessary to use small load increments, which in turn will increase the computation cost if compared with other techniques [57].

► Incremental-Iterative Techniques

This technique, Figure 5.6 (c), is usually carried out by applying the external loads as a sequence of sufficiently small increments, and within each increment of loading, iterations are performed until equilibrium is satisfied according to some selected convergence criterion. This mixed method is widely used in the analysis of reinforced concrete structures because of its accuracy and capability to provide information throughout the loading history.

ANSYS employs the "Newton-Raphson" approach to solve nonlinear problems. In this approach, the load is subdivided into a series of load increments. The load increments can be applied over several load steps. Figure 5.7 illustrates the use of Newton-Raphson equilibrium iterations in a single DOF nonlinear analysis[53].

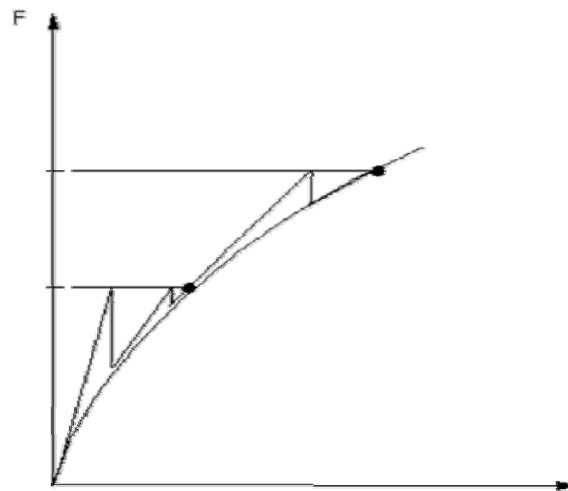


Figure 5.7 Newton-Raphson approach [53]

5.6 Convergence Criterion

For every incremental load, the iteration continues until convergence is achieved. The convergence criteria for the nonlinear analysis of structural problems can be classified as:

1. Force criterion.
2. Displacement criterion.
3. Stress criterion.

Only the displacement criterion is adopted in the present study. In the displacement criterion, the incremental displacements at iteration i and the total displacement are determined. The solution is considered to be converged when the norms of the incremental displacements are within a given tolerance of the norm of the total displacements; the (Euclidean norm) is used and takes the form [53].

$$\|(\Delta U_i)\| = (\sum \Delta U_i^2)^{0.5} \leq T_n (\sum U_i^2)^{0.5} \dots\dots\dots (5.19)$$

Where: U may equal u, v, w

5.7 Analysis Termination Criterion

In the physical test under force control, collapse of a structure takes place when no further loading can be sustained. This is usually indicated in the numerical tests by successively increasing iterative displacements and a continuous growth in the dissipated energy. Hence, the convergence of the iterative process cannot be achieved. A maximum number of iterations for each increment are specified to stop the nonlinear solution if the convergence limit has not been achieved for this study. This maximum number of iterations depends on the type of the problem, extent of nonlinearities, and on the specified tolerance [59]. In this study the maximum number of iteration is 50.

5.8 Material Modeling

In the present section, and as required within the frame of this study, a description of the constitutive relationships for the materials comprising the reinforced concrete beams with CFRP is given.

5.8.1 Mechanical Behavior of Concrete

Concrete is a material with a grossly heterogeneous internal structure. It consists of inert aggregate particles embedded within a binding paste made of cement and water. The presence of bond micro-cracks at interfaces between the cement paste (and mortar) and especially the coarse aggregate prior to any load application can be viewed as a source of weakness in the structure of concrete [60]. Segregation, shrinkage and thermal movements in the mortar cause many of these microcracks. Some micro-cracks may be developed during loading because of the difference in stiffness between the aggregate and the mortar. The gradual growth of these micro-cracks with further loading contributes to the nonlinear behavior of concrete [60]. Under additional load, these behaviors are more complicated by occurrence of cracks, resulting in a redistribution of stresses and causing increase in steel stresses, bond stress, and some bond slip. A much more complex situation due to the formations of cracks are the effect of aggregate interlock, and dowel action. These nonlinear responses are highly increased by the yielding of reinforcement and the nonlinear behavior of concrete in compression.

Many experimental studies of the behavior of concrete under uniaxial and multiaxial loading conditions have been performed. The aims of such investigations made to understand the complex response of concrete for various imposed stress conditions and to provide the necessary data required to develop accurate numerical models for use in nonlinear finite element analysis of concrete structures.

5.8.1.1 Uniaxial Behavior of Concrete

Typical plots of compressive stress versus strains in the uniaxial compression test are shown in Figure 5.8. The main observations and the characteristics of these curves can be summarized, as follows [60]:

1. The axial stress-strain curve as shown in Figure 5.8 has a nearly linear elastic behavior up to 30% of its maximum compressive strength, f_c' . This stress level is called the point of onset of localized cracking [61]. For stress above $0.3 f_c'$, concrete begins to soften and the curve shows a gradual increase in curvature up to about $0.75 f_c'$ to $0.9 f_c'$, after which the curve bends more sharply until it approaches the peak point at f_c' . Beyond the peak stress level, concrete shows a softening response, which is presented by the descending portion of the stress-strain curve.
2. The modulus of elasticity of concrete E_c is generally taken to be a function of the compressive strength f_c' , with a slightly higher value of the modulus corresponding to a higher compressive strength. In lieu of actual test data, the modulus of elasticity, E_c can be calculated with a reasonable accuracy from the empirical formula [1].

$$E_c = 0.043(w_c)^{1.5}(f_c')^{0.5} \quad \dots\dots\dots (5.19)$$

In which, w_c is the air-dry unit weight of concrete in kg/m^3 , f_c' is the cylinder compressive strength of concrete in N/mm^2 and E_c is the modulus of elasticity of concrete in N/mm^2 .

For the normal weight concrete based on a dry unit weight (2200-2500 kg/m^3), E_c can be permitted to be taken as [1]:

$$E_c = 4700 \sqrt{f_c'} \quad (\text{MPa}) \quad \dots\dots\dots (5.20)$$

3. Poisson's ratio (ν) of concrete has been observed to remain approximately constant and ranges from about 0.15 to 0.22 up to a stress level of $80\%f_c'$. Beyond this level, Poisson's ratio increases rapidly and values in excess of 1.0 have been measured [63].

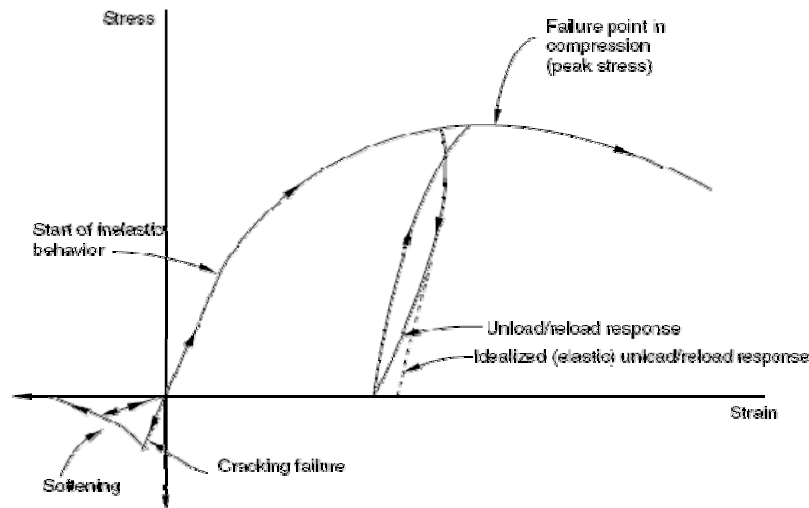


Figure 5.8 Typical stress-strain curves for concrete in uniaxial compression test [62].

4. The direct tensile strength of concrete is difficult to measure and is normally taken as $0.3 \text{ to } 0.4\sqrt{f_c'}$. Many times, either the modulus of rupture, f_r or the split cylinder strength (f_t) is used to approximate the tensile strength of concrete. The value of the modulus of rupture of concrete varies quite widely but is normally taken as in Equation 5.21[1]. The split cylinder tensile strength is usually somewhat lower, at approximately $0.45 \text{ to } 0.55\sqrt{f_c'}$ in N/mm^2 [60].

$$0.6\lambda \sqrt{f_c'} \dots\dots\dots(5.21)$$

Where: f_r and f_c' in MPa

$\lambda = 0.75$ for all lightweight concrete

$\lambda = 0.85$ for sand-lightweight concrete

$\lambda = 1$ for normal weight concrete

5.8.1.2 Multiaxial Behavior of Concrete

The behavior of concrete under multiaxial stress condition is very complex if compared with that under uniaxial stress condition. A typical biaxial strength envelope is shown in Figure 5.9a. When subjected to biaxial compressive, the maximum strength increases approximately 25% at a stress ratio of ($\sigma_2/\sigma_1=0.5$) and 16% at an equal biaxial compression state ($\sigma_2/\sigma_1=1$). Under biaxial tension-compression, the tensile strength decreased almost linearly as applied compression increase. For biaxial tension states, concrete exhibits nearly constant tensile strength [64].

Under triaxial compressive stress, concrete exhibits strength that increases with the increasing confining pressures. Experimental studies indicate that the three-dimensional failure envelope is a function of the three principal stresses as shown in Figure 5.9b. This failure surface can be represented by three-stress invariants. These invariants are the first invariant of the stress tensor (I_1), the second and the third invariant of the stress deviatoric tensor (J_2) and (J_3) [60].

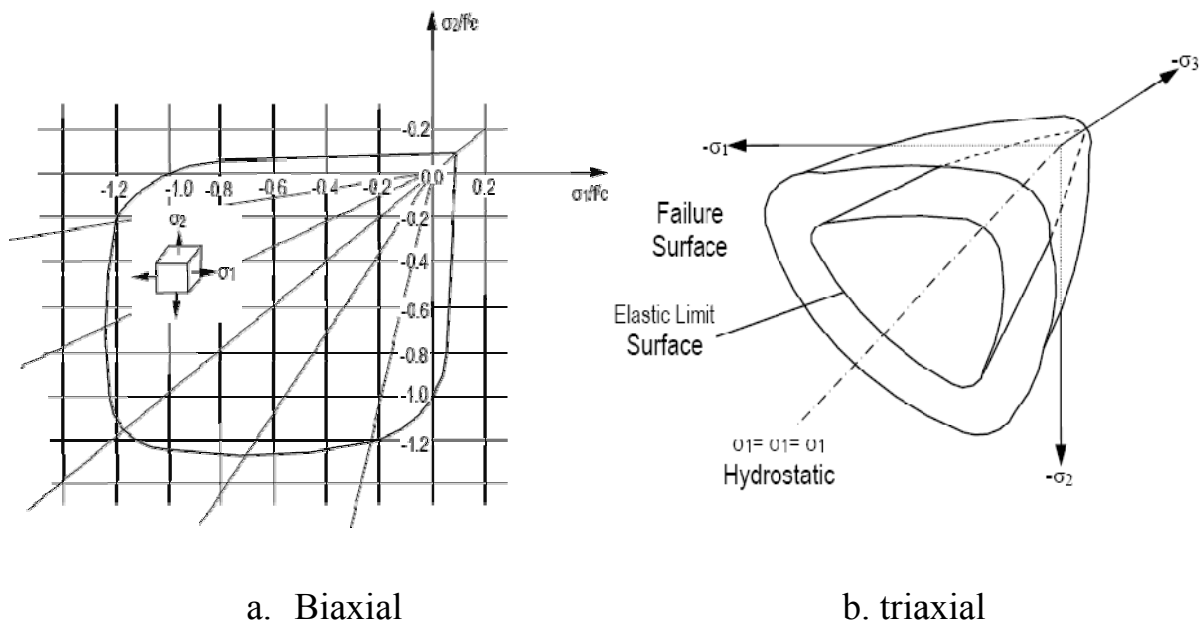


Figure 5.9 Typical multiaxial stress-strain curve for concrete [60]

5.8.2 Concrete Modeling

In the present study, concrete behavior is considered to be elasto-plastic followed by perfect plastic brittle fracture model. This model should include the following:

1. Stress-strain relationship
2. Failure criterion to simulate cracking and crushing types of fractures
3. Cracking modeling
4. Crushing modeling

5.8.2.1 Stress-Strain Relationship

The adopted compressive uniaxial stress-strain relationship for the concrete model was obtained using the following equations to complete the multilinear isotropic stress-strain curve for the concrete [65].

$$f = \frac{E_c \varepsilon}{1 + \left(\frac{\varepsilon}{\varepsilon_o}\right)^2} \dots\dots\dots (5.22)$$

$$\varepsilon_o = \frac{2\hat{f}_c}{E_c} \dots\dots\dots (5.23)$$

$$E_c = \frac{f}{\varepsilon} \dots\dots\dots (5.24)$$

f = stress at any strain ε

ε = strain at stress f

ε_o = strain at ultimate compressive strength

The multilinear isotropic stress-strain implemented requires the first point of the curve to be satisfying Hook law. As shown in Figure 5.10 the simplified stress-strain relationship for this study is constructed from six points connected by straight lines. The first point is defined as $0.3\hat{f}_c$, is calculated in the linear range (equation 5.24). Points 2, 3, 4 and 5 are calculated from equation 5.22, while point 6 is at ε_o and $\hat{f}_c$ is calculated from equation 5.23.

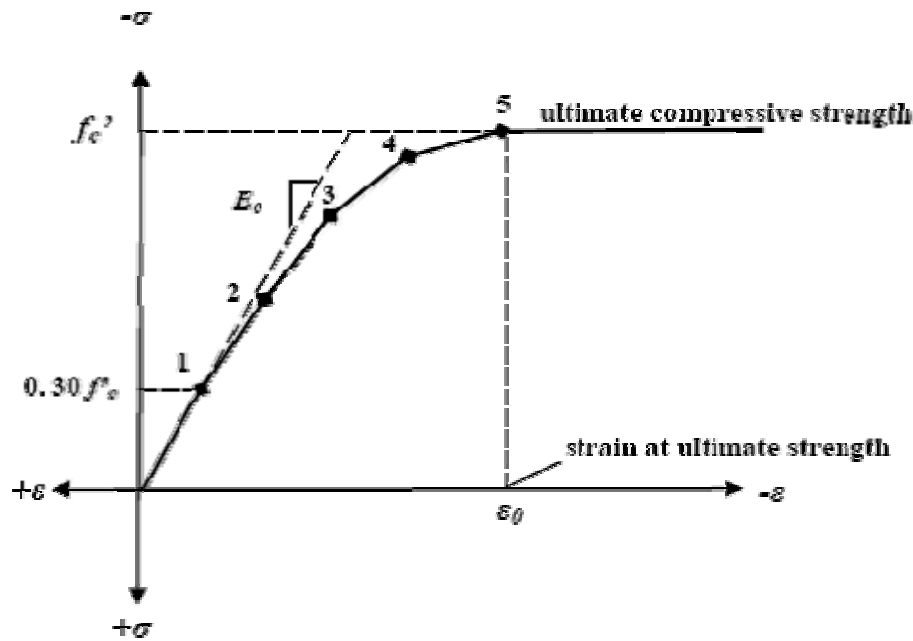


Figure 5.10 Uniaxial Stress-Strain Curve [65]

5.8.2.2 Failure Criterion for Concrete

Willam and Warnke in 1974 [65], developed a mathematical model capable of predicting failure for concrete materials under multiaxial stress state. Both cracking and crushing failure modes are accounted for. This model is represented by the following equation:

$$\frac{F}{f'_c} - S \geq 0 \dots\dots\dots (5.25)$$

Where

F = function of principal stress state ($\sigma_{xp}, \sigma_{yp}, \sigma_{zp}$).

S = failure surface expressed in terms of principal stresses and five input parameters (f_t, f'_c, f_{cb}, f_1 and f_2).

$\sigma_{xp}, \sigma_{yp}, \sigma_{zp}$ = principal stresses in principal directions

If equation (5.25) is satisfied, the material cracks or crushes.

Five input strength parameters are needed to define the failure surface as well as an ambient hydrostatic stress state (σ_h^a). These are:

f_t = ultimate uniaxial tensile strength.

f'_c = ultimate uniaxial compressive strength.

f_{cb} = ultimate biaxial compressive strength

f_1 = ultimate compressive strength for a state of biaxial compression superimposed on hydrostatic stress state (σ_h^a)

f_2 = ultimate compressive strength for a state of uniaxial compression superimposed on hydrostatic stress state (σ_h^a)

σ_h^a = ambient hydrostatic stress state.

The failure surface can be specified with a minimum of two constants, f_t and f'_c . The other three constants can be determined from Willam and Warnke:

$$f_{cb} = 1.2 f'_c \quad \dots\dots\dots (5.26)$$

$$f_1 = 1.45 f'_c \quad \dots\dots\dots (5.27)$$

$$f_2 = 1.725 f'_c \quad \dots\dots\dots (5.28)$$

However, these values are valid only for stress states where the condition stated below is satisfied:

$$|\sigma_h| \leq \sqrt{3} f'_c \quad \dots\dots\dots (5.29)$$

Where:

$$\sigma_h = \text{hydrostatic stress state} = \frac{1}{3}(\sigma_{xp} + \sigma_{yp} + \sigma_{zp}) \quad \dots\dots\dots (5.30)$$

Both the function F and the failure surface S are expressed in terms of principal stresses denoted as σ_1 , σ_2 , and σ_3 where:

$$\sigma_1 = \max (\sigma_{xp}, \sigma_{yp}, \sigma_{zp})$$

$$\sigma_3 = \min (\sigma_{xp}, \sigma_{yp}, \sigma_{zp})$$

and $\sigma_1 \geq \sigma_2 \geq \sigma_3$.

The failure of concrete is categorized into four domains:

- 1- $0 \geq \sigma_1 \geq \sigma_2 \geq \sigma_3$ (compression – compression – compression)
- 2- $\sigma_1 \geq 0 \geq \sigma_2 \geq \sigma_3$ (tensile – compression – compression)
- 3- $\sigma_1 \geq \sigma_2 \geq 0 \geq \sigma_3$ (tensile – tensile – compression)
- 4- $\sigma_1 \geq \sigma_2 \geq \sigma_3 \geq 0$ (tensile – tensile – tensile).

The concrete will crack if any principal stress is a tensile stress, while crushing will occur if all principal stresses are compressive.

A three-dimensional failure surface for concrete is shown in Figure 5.11. The most significant nonzero principal stresses are in the x and y directions, represented by σ_{xp} and σ_{yp} , respectively. Three failure surfaces are shown as projections on the σ_{xp} - σ_{yp} plane. The mode of failure is a function of the sign of σ_{zp} (principal stress in the z direction). For example, if σ_{xp} and σ_{yp} are both negative (compressive) and σ_{zp} is slightly positive (tensile), cracking would be predicted in a direction perpendicular to σ_{zp} . However, if σ_{zp} is zero or slightly negative so the material is assumed to crush [53].

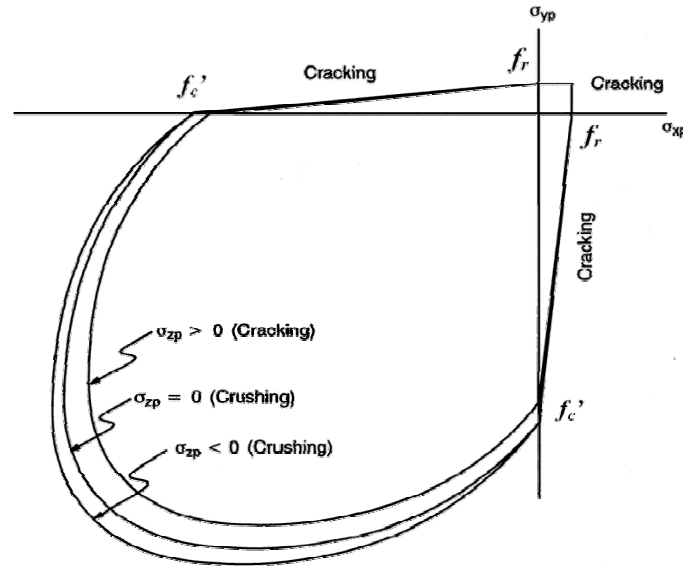


Figure 5.11 3D-failure surface for concrete [53]

In a concrete element, cracking occurs when the principal tensile stress in any direction lies outside the failure surface. After cracking, the elastic modulus of the concrete element is set to zero in the direction parallel to the principal tensile stress direction. Crushing occurs when all principal stresses are compressive and lies outside the failure surface; subsequently, the elastic modulus is set to zero in all directions, and the element effectively disappears [53]. A pure “compression” failure of concrete is unlikely. In a compression test, the specimen is subjected to a uniaxial compressive load. Secondary tensile strains induced by Poisson’s effect occur perpendicular to the load. Because concrete is relatively weak in tension, these actually cause cracking and the eventual failure [66] [67].

Through the study in this filed, it was found that if the crushing capability of the concrete is turned on, the finite element beam models fail prematurely, also, the solution showed divergence. Therefore, in this study, the crushing capability was turned off and cracking of the concrete was assumed to control the failure of the finite element models as recommended in many researches [37, 43 and 68].

5.8.2.3 Cracking Mode

Three different approaches for crack modeling have been employed in the analytical studies of concrete structures using the numerical technique of the finite element method. These are (1) smeared cracking mode, Figure 5.12(a), (2) discrete cracking mode Figure 5.12(b), and (3) fracture mechanics mode. The particular cracking model to be selected from the three alternatives depends on the purpose of the analysis. In general, if overall load-deflection behavior is desired, the smeared crack modeling is probably the best choice [60]. If detailed local behavior is of interest, adaptations of the discrete cracking modeling are

useful. For the special class of problems in which fracture mechanics is the appropriate tool, a specialized fracture model may prove to be necessary [60].

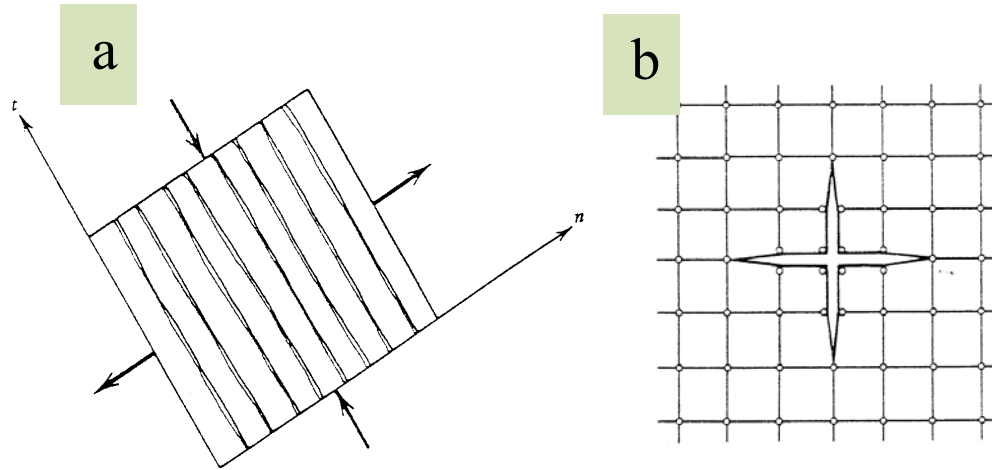


Figure 5.12 (a) *The smeared cracking modeling approach [69]* (b) *two dimensional cracking representation in discrete cracking modeling approach [69]*

For ANSYS computer program [53], crack modeling of concrete, depends on smeared cracks; therefore, general description for this modeling is given below.

The smeared crack model was first used by Rashid (1968) [70]. In this approach, the cracked concrete is assumed to remain as a continuum; that is, the cracks are "smeared out" in a continuous fashion. Rather than representing a single discrete crack, the present approach has the effect of representing an infinite number of parallel fissures (tiny cracks) across the cracked concrete element Figure 5.12a. These parallel fissures are assumed to form in the planes (or surfaces for axisymmetric problems) perpendicular to the maximum principal stress (or strain) direction. After the first cracking has occurred, it is assumed that the cracked concrete becomes orthotropic or transversely isotropic, with one

of the material principal axes being oriented along the direction of cracking.

Before cracking, the uncracked concrete is assumed an isotropic linear elastic material. However, the onset of cracking introduces orthotropic material properties and new incremental constitutive relationships must be derived. This is accomplished by modifying the tangent material stiffness (or the tangent elasticity matrix) [60].

In ANSYS computer program [53], the presence of a crack at an integration point is represented through modification of the stress-strain relations by introducing a plane of weakness in a direction normal to the crack face. Also, a shear transfer coefficient is introduced which represents a shear strength reduction factor for those subsequent loads which induce sliding (shear) across the crack face.

2.8.2.3.1 Shear Transfer Model

Concrete is assumed to behave linearly in tension up to the onset of cracking. When concrete cracks, its shear stiffness is reduced. However, cracked concrete can partially transmit shear across the crack due to aggregate interlock and dowel action of the reinforcement. The shear transfer mechanism depends on the reinforcement ratio, bar size, bar arrangement, the amount of concrete cover, the type of concrete and aggregate size [71]. To estimate such an effect, a shear coefficient β is introduced which represents a shear strength reduction factor for concrete across the crack face [53]. When the crack is formed, only a constant value of a shear transfer coefficient β_o for the opened crack is introduced, and if the crack is closed, the shear transfer coefficient β_c is used. The values of β_o and β_c are always in the range $1 > \beta_o > \beta_c > 0$. These values depend on the texture of the cracked surface.

The shear transfer coefficient, β , represents conditions of the crack face. The value of β ranges from 0.0 to 1.0, with 0.0 representing a smooth crack (complete loss of shear transfer) and 1.0 representing a rough crack (no loss of shear transfer) [53]. The value of β_o used in many studies of reinforced concrete structures, however, varied between 0.05 and 0.2 [43]. For the beam without shear reinforcement the shear transfer coefficient for the open cracks used was 0.05, and the transfer coefficient for the closed cracks was 0.4, because there was no shear reinforcement to restrict the cracking growth if it was formed and the closed cracks were not capable to be performed as closed cracks for a long time of loading [17]. In the present study a 0.05 and 0.4 coefficients for the open and close crack are adopted in this study respectively.

5.8.2.4 Crushing Modeling

In ANSYS computer program [53], the material is assumed to crush at an integration point if the material at that point fails in uniaxial, biaxial, or triaxial compression. In concrete element, crushing is defined as the complete deterioration of the structural integrity of the material (e.g. material spalling). Under conditions where crushing has occurred, material strength is assumed to have degraded to an extent such that the contribution to the stiffness of an element at the integration point in question can be ignored.

5.8.3 STEEL

Compared to concrete, steel is a much simple material to represent. Its strain-stress behavior can be assumed identical in tension and in compression. A typical uniaxial stress-strain curve for a steel specimen loaded monotonically in tension is shown in Figure 5.13.

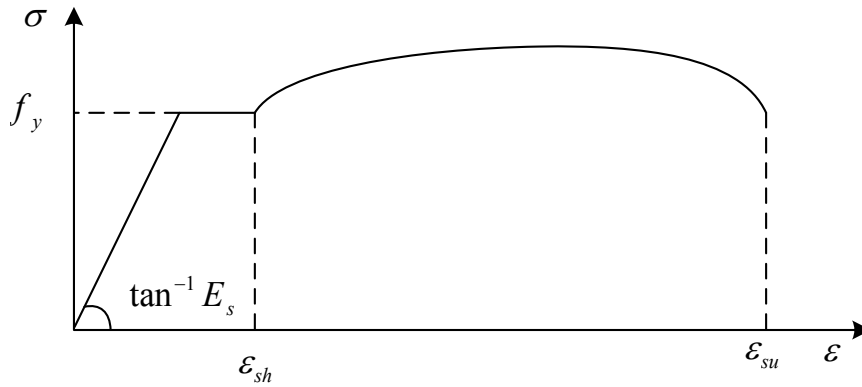


Figure 5.13 Typical stress-strain curve for steel [72]

The stress-strain diagram may for simplicity consists of two branches: the first branch starts from the origin with a slope equal to E_s , up to f_y . A second branch is horizontal or, for practical use of computers, is assumed to have a very small slope such as $10^{-2}E_s$ and this last case is limited to the strain 0.01 according to [73] and it is adopted in the present study.

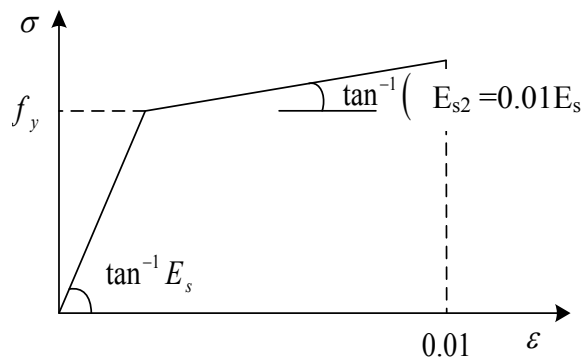


Figure 5.14 Idealization for computer calculations [73]

Tavarez (2001) [74] discusses three techniques that exist to model steel reinforcement in finite element models for reinforced concrete. Figure 5.15 shows the discrete model, the embedded model, and the smeared model.

The reinforcement in the discrete model Figure 5.15a uses bar or beam elements that are connected to concrete mesh nodes; therefore, the concrete and the reinforcement mesh share the same nodes and concrete occupies the same regions occupied by the reinforcement [74]. Also, it has a capability to adequately account for the bond-slip and dowel action phenomena [75].

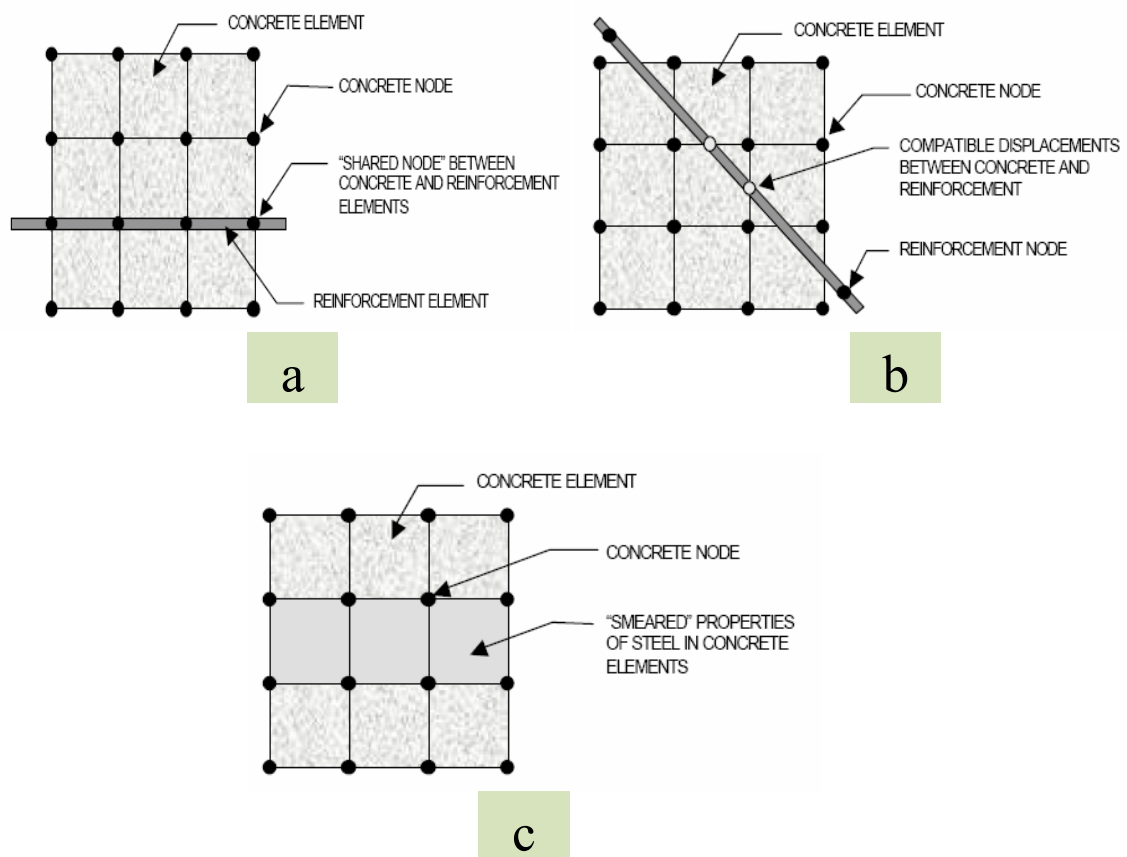


Figure 5.15 Models for Reinforcement in Reinforced Concrete [74]:

(a) discrete; (b) embedded; and (c) smeared

The embedded model Figure 5.15b overcomes the concrete mesh restriction because the stiffness of the reinforcing steel is evaluated separately from the concrete elements. The model is built in a way that keeps reinforcing steel displacements compatible with the surrounding concrete elements. When reinforcement is complex, this model is very advantageous. However, this model increases the number of nodes and

degrees of freedom in the model; therefore, increasing the run time and computational.

The smeared model Figure 5.15c assumes that reinforcement is uniformly spread throughout the concrete elements in a defined region of the FE mesh. This approach is used for large-scale models where the reinforcement does not significantly contribute to the overall response of the structure.

Fanning [76] modeled the response of the reinforcement using the discrete model and the smeared model for reinforced concrete beams. It was found that the best modeling strategy was to use the discrete model when modeling reinforcement. So, it is adopted in the present study.

5.8.4 Behavior of CFRP Composites

FRP composites are materials that consist of two constituents. The constituents are combined at a macroscopic level and are not soluble in each other. One constituent is the reinforcement, which is embedded in the second constituent; a continuous polymer called the matrix. The reinforcing material is in the form of fibers, which are typically stiffer and stronger than the matrix. The FRP composites are anisotropic materials, that is, their properties are not the same in all directions. Figure 5.16 shows a schematic of FRP composites.

Linear elastic orthotropic properties of the FRP composite are assumed thought this study as shown in Figure 5.17. In addition, full bond between the concrete and CFRPs is assumed.

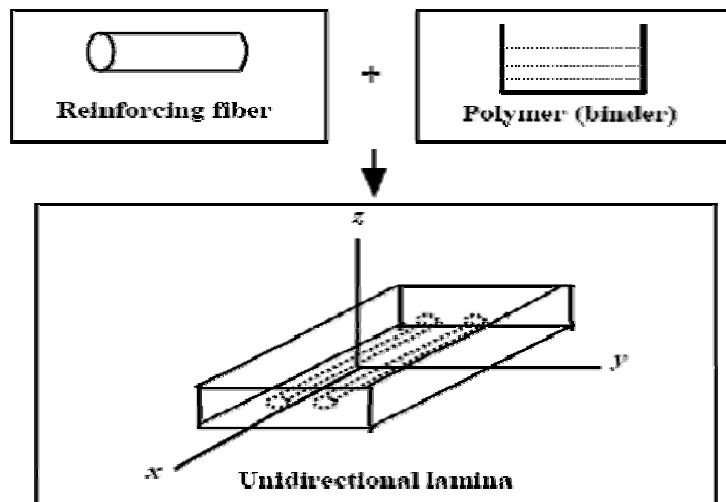


Figure 5.16 FRP composite [77]

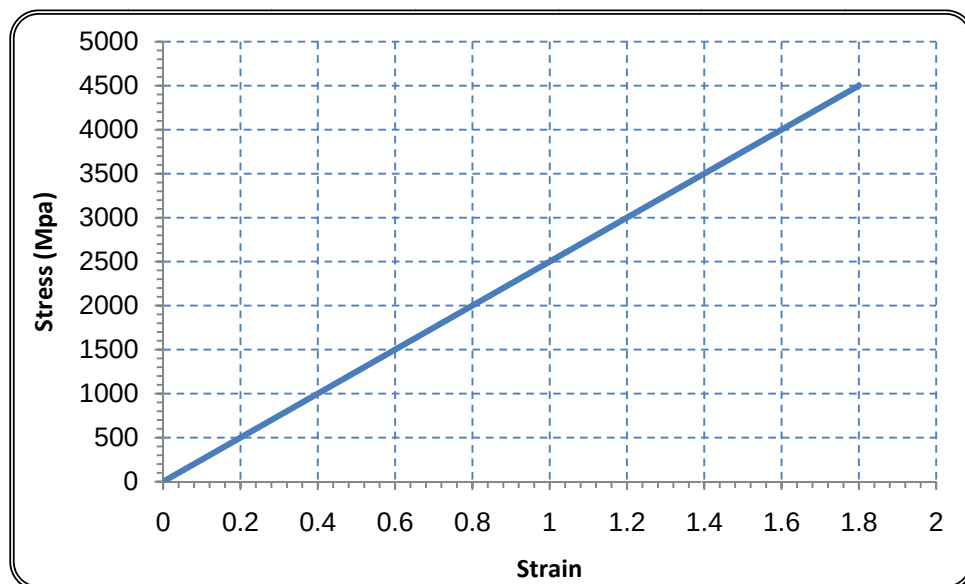


Figure 5.17 stress-strain relationship for FRP composite [50]

CHAPTER SIX

FINITE ELEMENT ANALYSIS

6.1 Introduction

The aim of the present chapter is to make a comparison between the F.E. model results and the experimental results that explore the adequacy of elements type, material modeling, real constants and convergence criteria are adequate to model the response of the reinforced concrete deep beam with opening strengthening by CFRP.

This chapter includes the analysis of beams tested in chapter four and some important parametric studies by using a powerful nonlinear finite element method package (ANSYS software).

6.2 Model Verification

In order to verify the used model, three examples in literature are analyzed to check the accuracy of material modeling, real constant and boundary condition. The results are compared with the experimental results.

6.2.1 Zarnic et al. strengthened beam

Figure 6.1 shows the geometry and loading of Zranic simply supported strengthened beam. The beam flexurally strengthened by CFRP plate. The properties of the strengthened beam are presented in the Table 6.1[78].

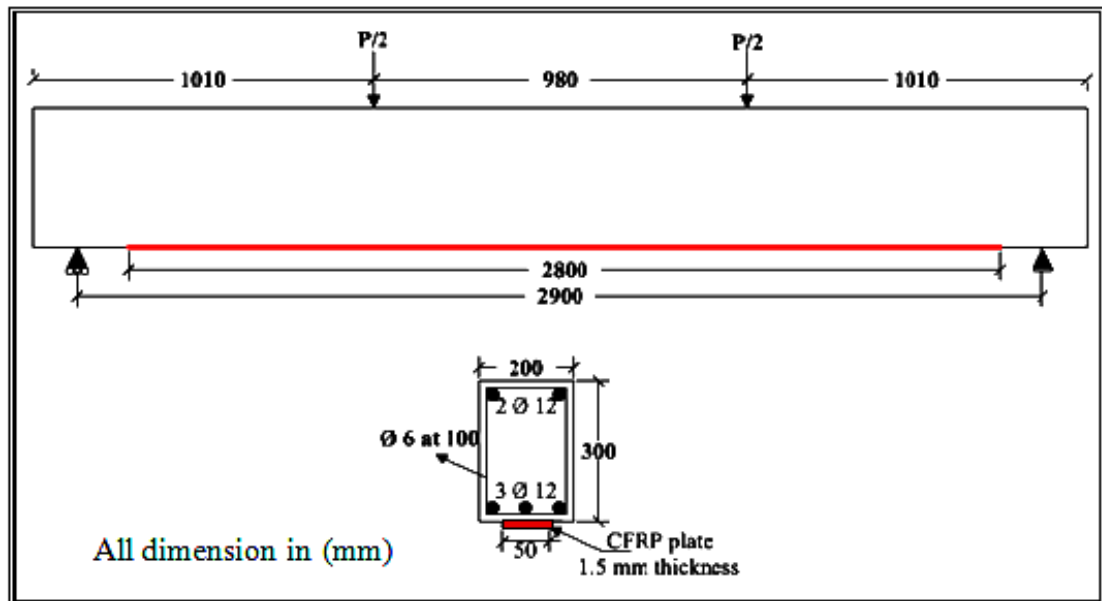


Figure 6.1 Detail of Zarnic et al. strengthened beam [78].

Table 6.1 Material properties of Zarnic et al. strengthened beam [78]

Type of material	Material Properties	Symbol	Value
Concrete	Young's modulus	E_c (MPa)	27000
	Compressive strength	$\hat{f}_c$ (MPa)	25
	Tensile strength	f_t (MPa)	3
	Poisson's ratio	ν	0.2
	Shear transfer coefficients	$Shr-op$	0.2
		$Shr-cl$	0.85
Steel	Young's modulus	E_s (MPa)	210000
	Yield stresses	f_y (MPa)	460
	Poisson's ratio	ν	0.3
CFRP	Young's modulus	E_{CFRP} (MPa)	150000
	Thickness	t (mm)	1.2

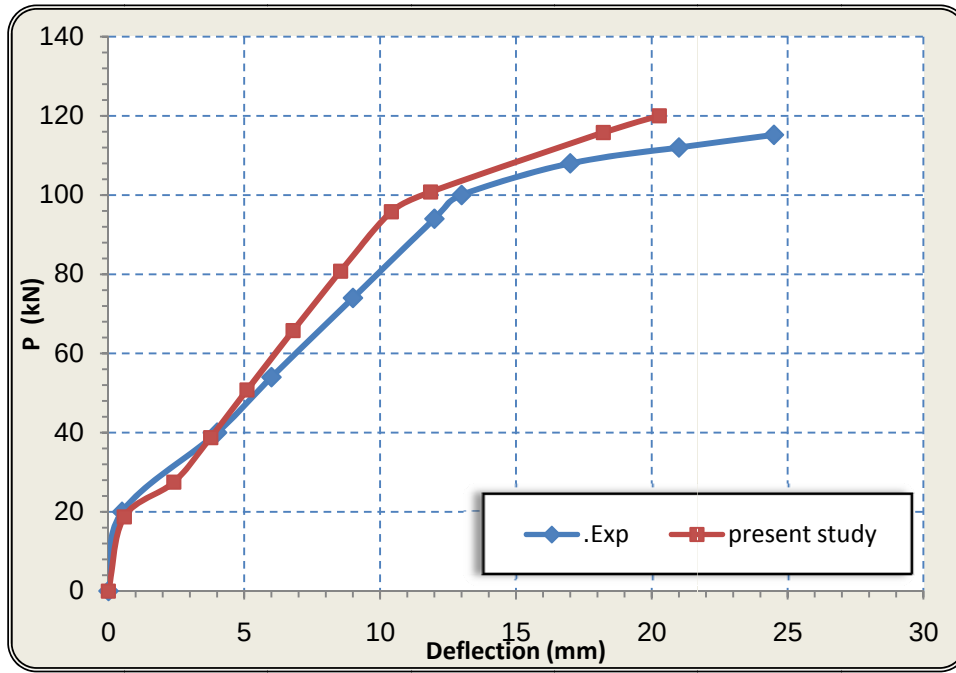


Figure 6.2 Numerical and experimental Load-Deflection curve for Zarnic et al. beam compared with the present model

As shown in Figure 6.2 the results show reasonable agreement between the experimental and numerical results. Where the difference in the ultimate load are about 4 %.

6.2.2 Pham and Al-Mahaidi's CFRP simple beam

The loading and boundary conditions are shown in Figure 6.3. The beam flexurally strengthened by a CFRP sheet had a thickness of (0.26 mm) and width of (100 mm) [79].

Table 6.2 contained the material properties and the parameters of the strengthened beam. Figure 6.4 shows a comparison between the experimental data and the numerical results.

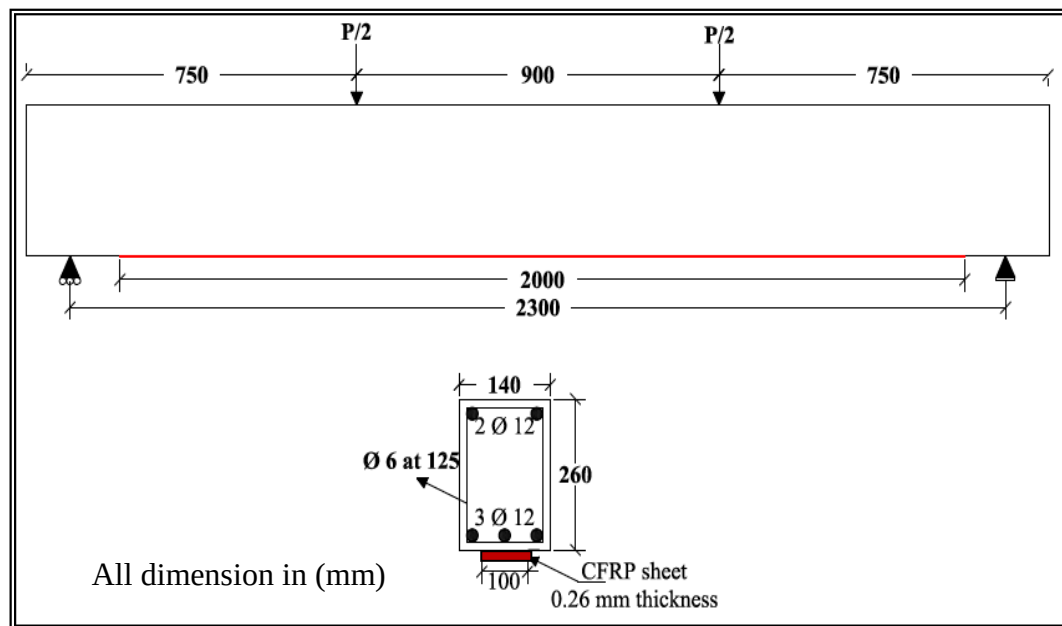


Figure 6.3 Detail of Pham and Al-Mahaidi composite beam [79]

Table 6.2 Material properties of Pham and Al-Mahaidi's composite beam [79].

Type of material	Material Properties	Symbol	Value
Concrete	Young's modulus	E_c (MPa)	32460
	Compressive strength	$\hat{f}_c$ (MPa)	47.7
	Tensile strength	f_t (MPa)	3
	Poisson's ratio	ν	0.2
	Shear transfer coefficients	$Shr-op$	0.2
		$Shr-cl$	0.85
Steel	Young's modulus	E_s (MPa)	192000
	Yield stresses	f_v (MPa)	504
	Poisson's ratio	ν	0.3
CFRP	Young's modulus	E_{CFRP} (MPa)	213500
	Thickness	t (mm)	0.26

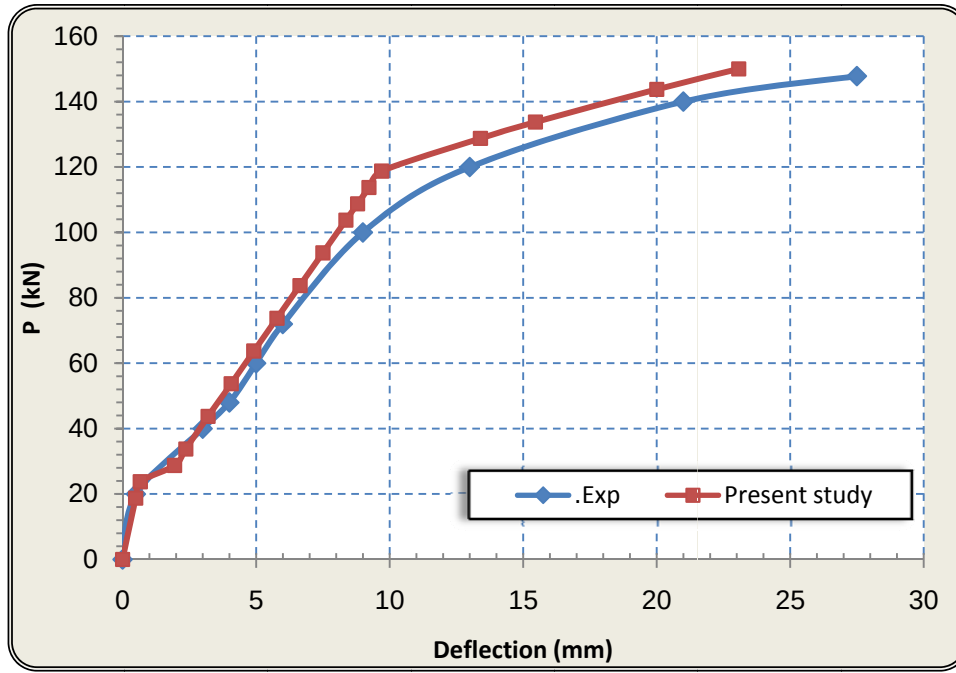


Figure 6.4 Numerical and experimental Load-Deflection curve for Pham and Al-mahaidi beam

As noted in Figure 6.4 the results show a good agreement between the experimental results and the F.E. model, where the difference in the ultimate load is about 1.9 %.

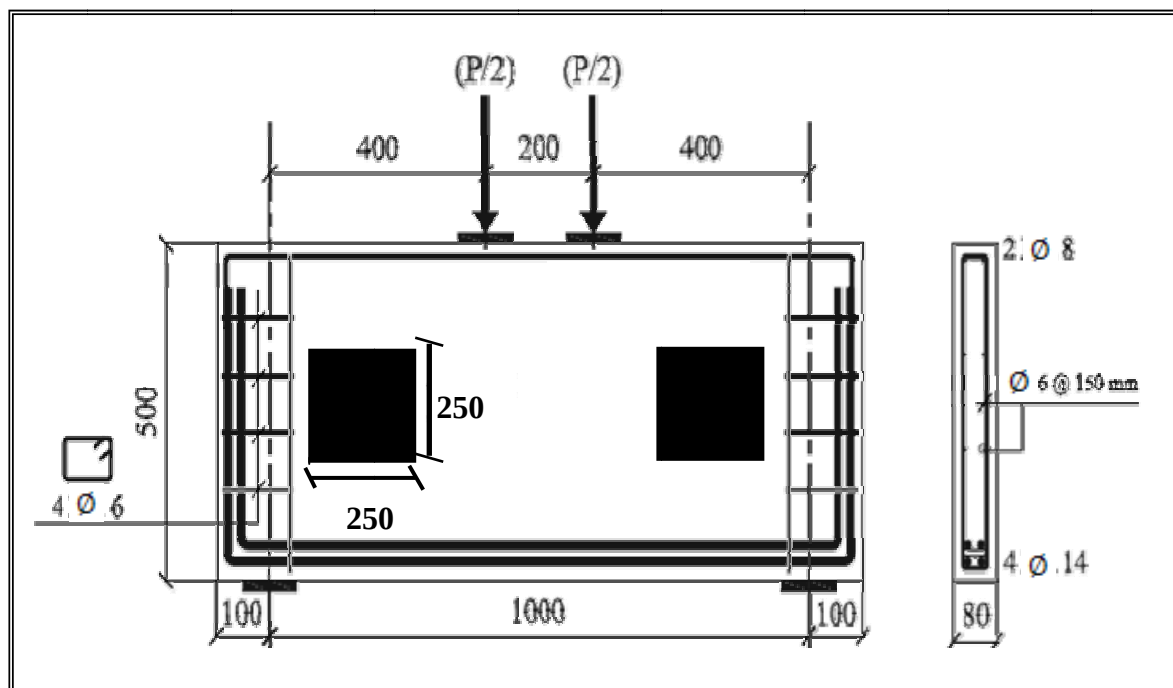
6.2.3 El Maaddawy and Sherif deep beam

A simply supported deep beam with openings tested by El Maaddawy and Sherif [3] is selected to verify the used model. Figure 6.5 shows the geometry and the dimension of the specimen, where the opening located at the center of shear span.

A summary of material properties and parameters for the tested beam are given in Table 6.3. Also, Figure 6.6 shows the comparison between the experimental and the numerical results.

Table 6.3 Material properties of El Maaddawy and Sherif deep beam [3]

Type of material	Material Properties	Symbol	Value
Concrete	Young's modulus	E_c (MPa)	32460
	Compressive strength	$\hat{f}_c$ (MPa)	21
	Tensile strength	f_t (MPa)	2
	Poisson's ratio	ν	0.2
	Shear transfer coefficients	$Shr-op$	0.2
		$Shr-cl$	0.85
Steel (main reinforcement)	Young's modulus	E_s (MPa)	200000
	Yield stresses	f_y (MPa)	420
	Poisson's ratio	ν	0.3
Steel (stirrup)	Young's modulus	E_{CFRP} (MPa)	200000
	Yield stresses	f_y (MPa)	350
	Poisson's ratio	ν	0.3

**Figure 6.5** Detail El Maaddawy and Sherif deep beam [3]

(All dimension in mm)

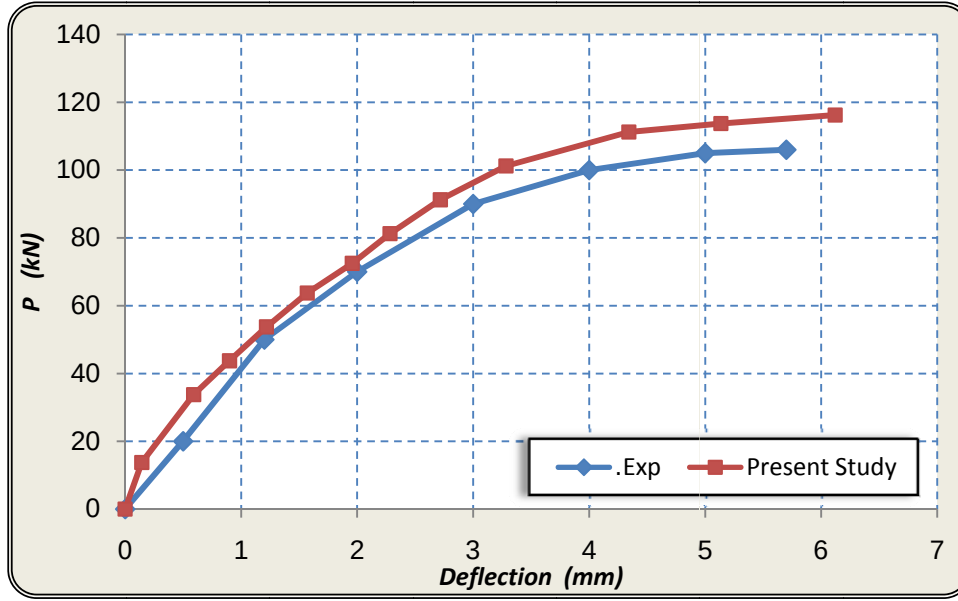


Figure 6.6 Numerical and experimental results for El Maaddawy and Sherif deep beam

As shown in Figure 6.6 the results show a good agreement between the present study and experimental results, where the difference in ultimate load is about 9% and 7 % in the ultimate deflection.

In general, the used model that is described in the previous chapter shows the validity of material properties, real constant and boundary condition though the comparison of the three examples above; therefore, this model will be used in the present study.

6.3 Specimen Description in Finite Element

By benefit from the symmetry, a quarter of the beam is being model as shown in Figure 6.7. This approach reduces computational time and the computer disk space requirements significantly. The section below describes the mesh density, boundary condition and applying load techniques that adopted in the present study.

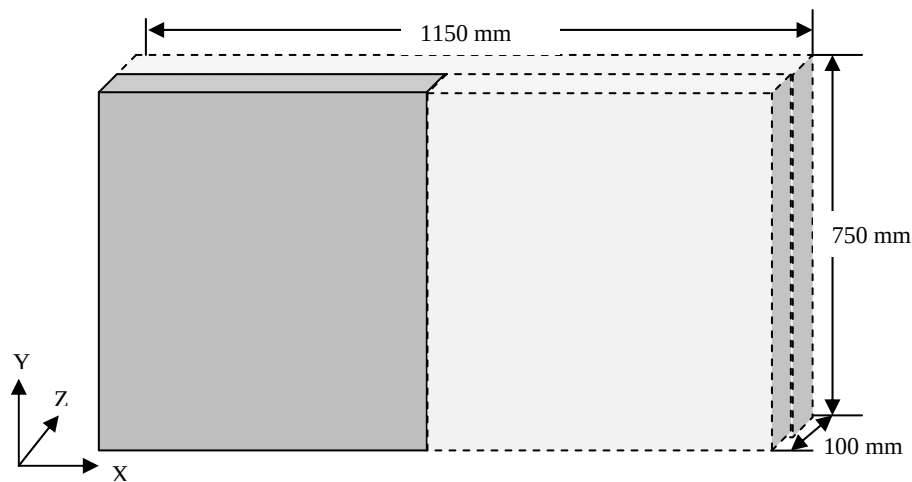


Figure 6.7 Sketch for quarter of the beam

6.3.1 Mesh Refinement

An important step in finite element modeling is the selection of the mesh density. A convergence of results is obtained when an adequate number of elements are used in a model. This is practically achieved when an increase in the mesh density has a negligible effect on the results; therefore, a convergence study was carried out for three cases, **C-1**(solid deep beam), **C-2** (deep beam with web opening) and **S-1**(deep beam with web opening strengthened with full wrap CFRP). Three types of mesh are used to find the best mesh size for three cases as shown in Figure 6.8.

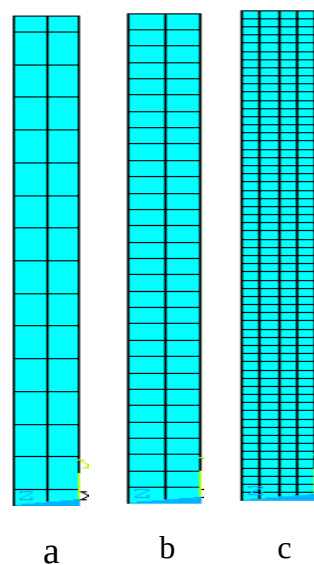


Figure 6.8 mesh density

Figure 6.9 shows the relation between the number of elements and mid-span deflection. It can be noted from the Figure below that the difference can be neglected when the number of elements increased from (1666) to (11040) for the solid deep beam, from (1602) to (10528) for the beam with opening and from (1878) to (11440); therefore, the (1666, 1602 and 1878) models select for model the C-1, C-2 and S-1 respectively.

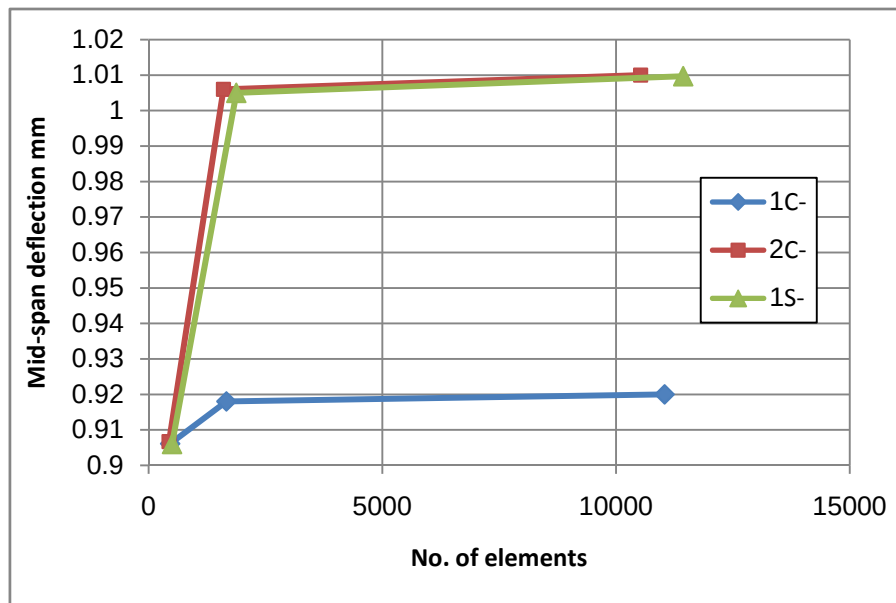


Figure 6.9 Result of convergence study

6.3.2 Loading and Boundary Condition

Displacement boundary conditions are needed to constrain the model to get unique representation for the actual beam. Boundary condition need to apply at points of symmetry and where the supports and loads exist. The symmetry boundary conditions are set first. The model being used is symmetric about two planes. To model the symmetry, nodes on these planes must be constrained in the perpendicular directions. Therefore, the nodes in U_x and U_z have a degree of freedom equal to zero as shown in Figure 6.10. The support was modeled in such a way as a roller. A single

line of nodes on the plate is given constraint in the UY direction. By doing this, the beam will be allowed to rotate at the support, as shown in Figure 6.10.

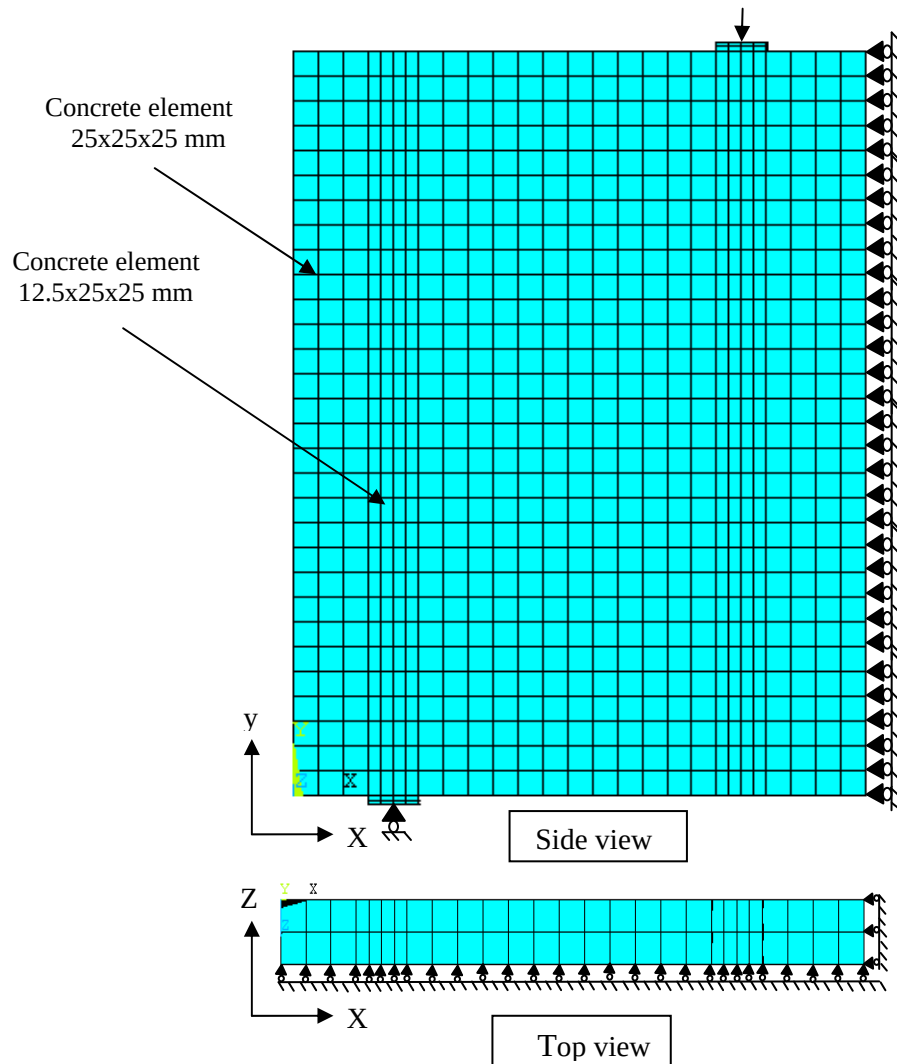


Figure 6.10 Boundary conditions for the quarter of the beam

A steel plate of (50x100x10 mm) is modeled by using SOLID45 elements, which is used at the support and loading locations to avoid the situation of stress concentration. A line of load is applied to the center of steel plate at the location of the load as illustrated in Figure 6.11.

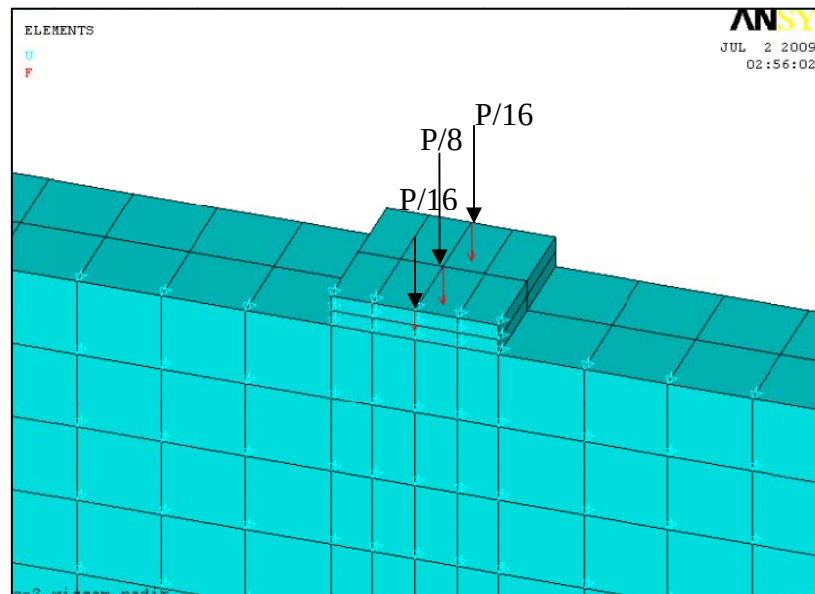


Figure 6.11 Boundary conditions at the loading plate

6.4 Results of Finite Element Analysis

► Specimen C-1

The numerical load-deflection curves are compared with the experimental results, shown in the Figures 6.12.

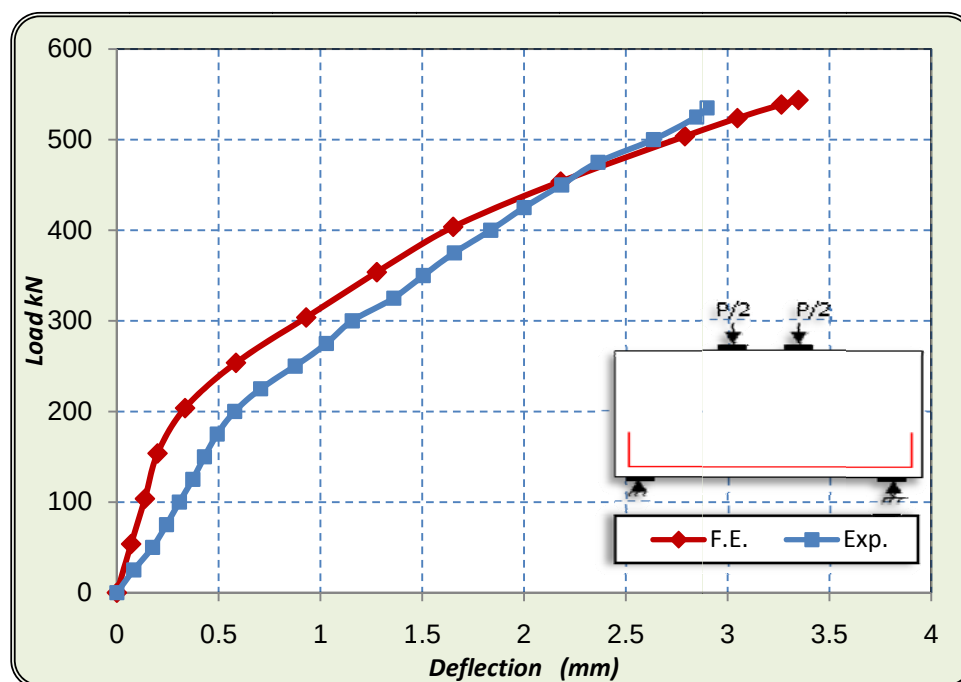


Figure 6.12 Load-Deflection curve for the Control Beam (C-1)

As shown in Figure 6.12 the FE result is stiffer at first stage of loading than the actual beam. The first crack obtained from the F.E. is a flexural crack at 148.75 kN, which is lower than the experimental (180 kN) about 17% that may be attributed to that cracks in experimental are measured when they visible. Then the shear cracks appear at 198.75 kN that lower for the experimental (240 kN) about 17%. As the applied load increased the inclined crack propagate upward to the loading point. The ultimate load for the F.E. (543.75 kN) is more than experimental (535kN) by about 1.64%. The ultimate deflection from F.E. is higher than experimental in about 15.5%.

Figure 6.13 shows a crack pattern of control beam (C-1) at the ultimate load. It can be noted in region (a), the flexural cracks sign appear perpendicular to the X-direction.

Figure 6.13 region (b) explains the development of shear stresses in XY- plane, which leads to the inclined crack as shown in region (b)

For the concrete structure subjected to the uniaxial compression, crack propagate parallel to the direction of the applied compressive load, since the crack result from the tensile strain development are due to Poisson's effect [67,68]; therefore, the sign of crack in the region (c) appear perpendicular on the Z-direction, these cracks are called compressive crack.

Figure 6.14 shows the destroy of the load path (arch action) which due to shear stresses in XY-plane, that support the validity of F.E. method.

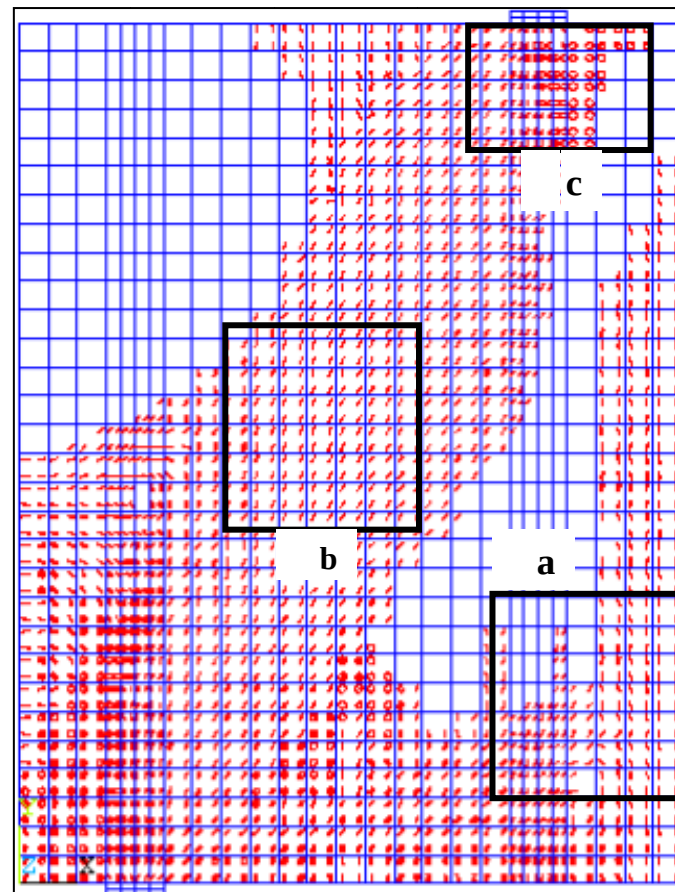


Figure 6.13 Crack pattern of C-1 at the ultimate load

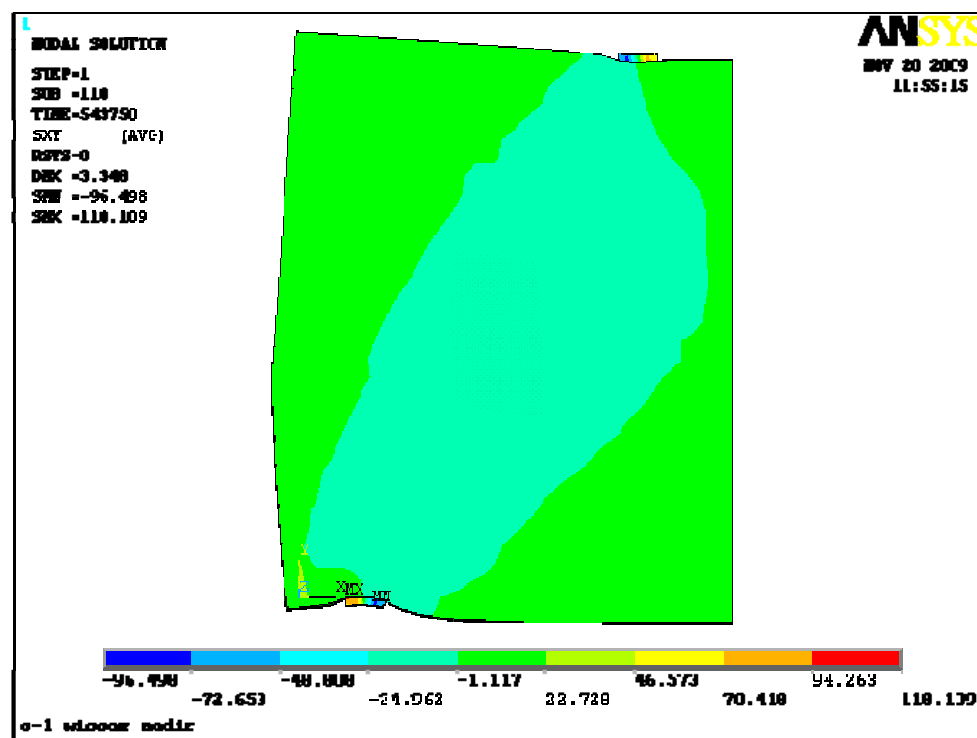


Figure 6.14 Shear stresses in XY-plane for control beam (C-1) at ultimate load

► Specimen C-2

Figure 6.15 shows a comparison between the numerical and experimental results.

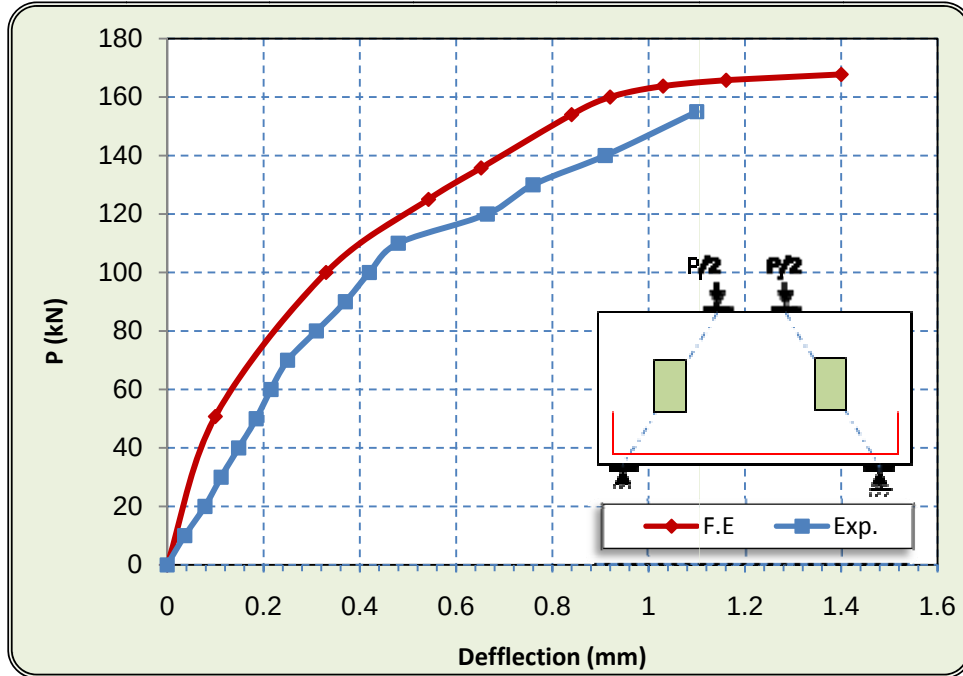


Figure 6.15 Load-Deflection curve for the Control Beam (C-2)

As shown above in Figure 6.15, the FE curve gives a stiffer response at first stage of loading than the experimental results. The first crack is inclined crack it appears at the corners of the opening at 67.75 kN, which is lower than the experimental (110 kN) in about 38%. These cracks development from the opposite corners upward to the supporting and loading points as shown in Figure 6.16. Flexural cracks appear at 85 kN and have limited propagate. With process the applied load cracks propagate from the end of the beam at the same level of top corner to the corner, that the same behavior in the experimental work. The ultimate load for the FE (167.75 kN) is higher than experimental results (155 kN) in about 8.2%.

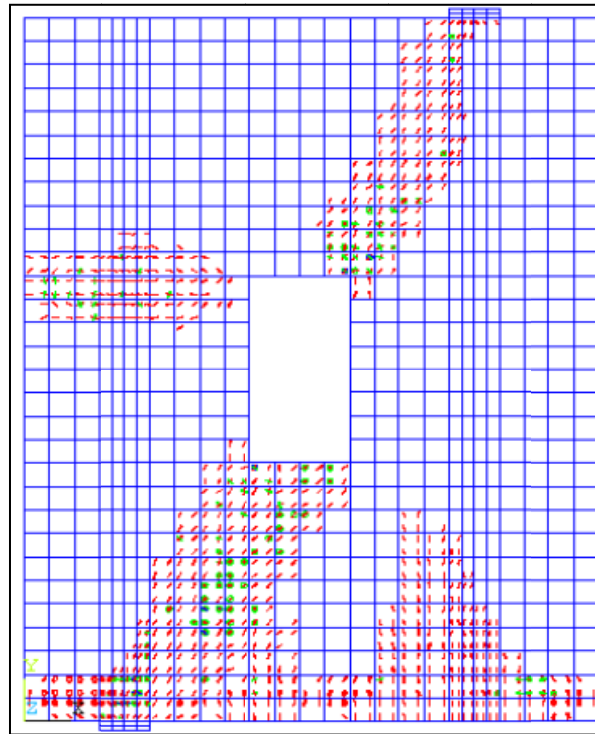


Figure 6.16 Crack pattern of control (C-2) at 0.9P

Figure 6.17 shows how the shear stresses transfer in XY-plane through the opening zone.

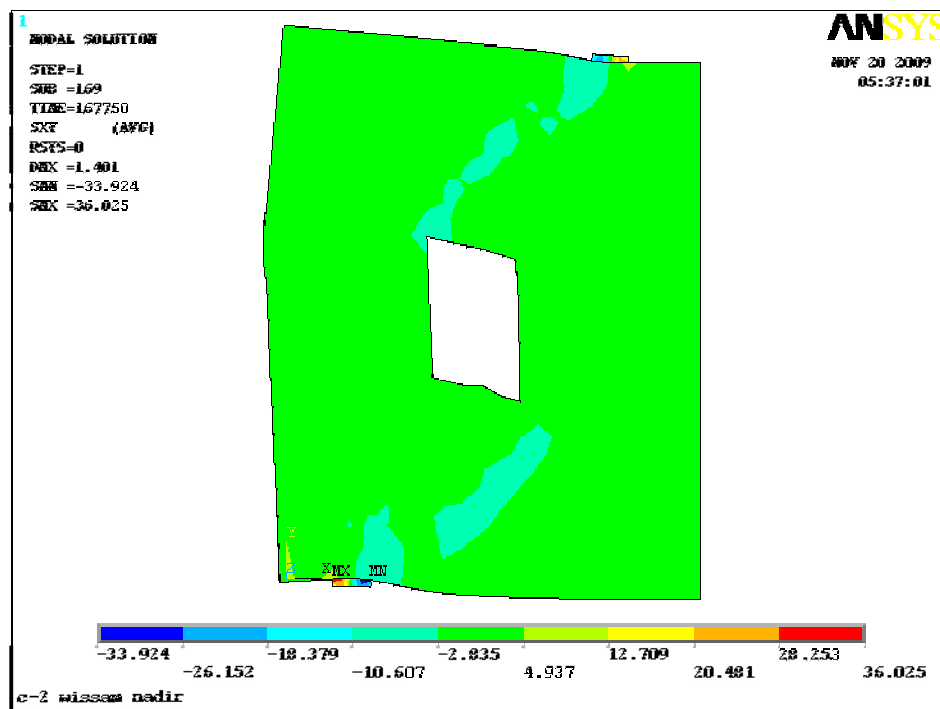


Figure 6.17 Shear stresses in XY-plane for control beam (C-2) at the ultimate load

► **Specimen S-1, S-2 and S-3 (strengthened beam by full wrap, U-wrap and two face wrap CFRP strip respectively)**

The finite element results show same behavior for three strengthened beams (S-1, S-2 and S-3), that support the experimental work. Figures 6.18 to 6.20 show the comparison between the experimental and F.E. results.

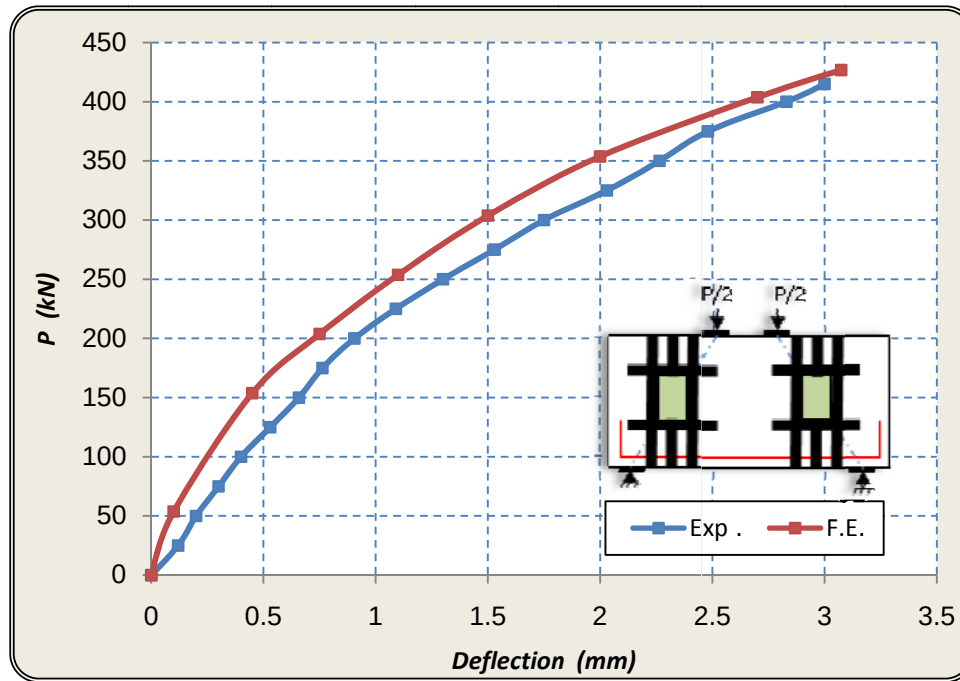


Figure 6.18 Load-Deflection curve for the strengthened Beam (S-1)

The load-deflection curve for FE shows a good agreement with the experimental results. The first crack appears at the opposite corners at 83.75 kN, which is higher than the control beam C-2 (67.75 kN) by about 24%, that is consistent with the experimental results. The crack pattern is similar to the control beam C-2, but, it's noted the flexural cracks increase propagate up to the third depth of the beam as shown in Figure 6.21. The ultimate load for the F.E. (426.75 kN) is higher than experimental results in about 2.83% for S-1, 1.6% for S-2 and 8% for S-3.

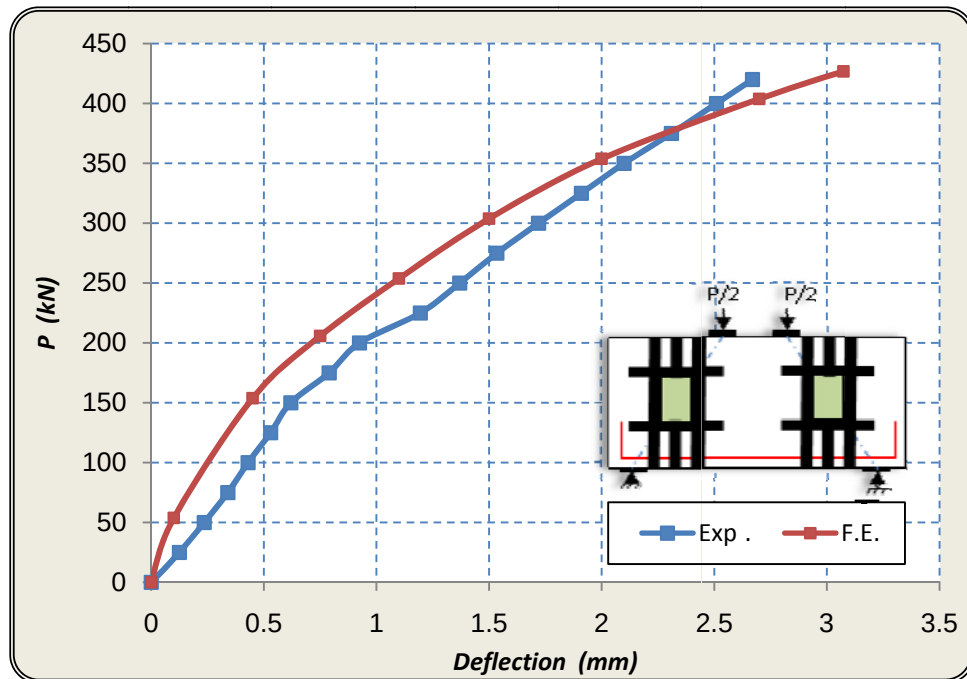


Figure 6.19 Load-Deflection curve for the strengthened Beam (S-2 U-wrap)

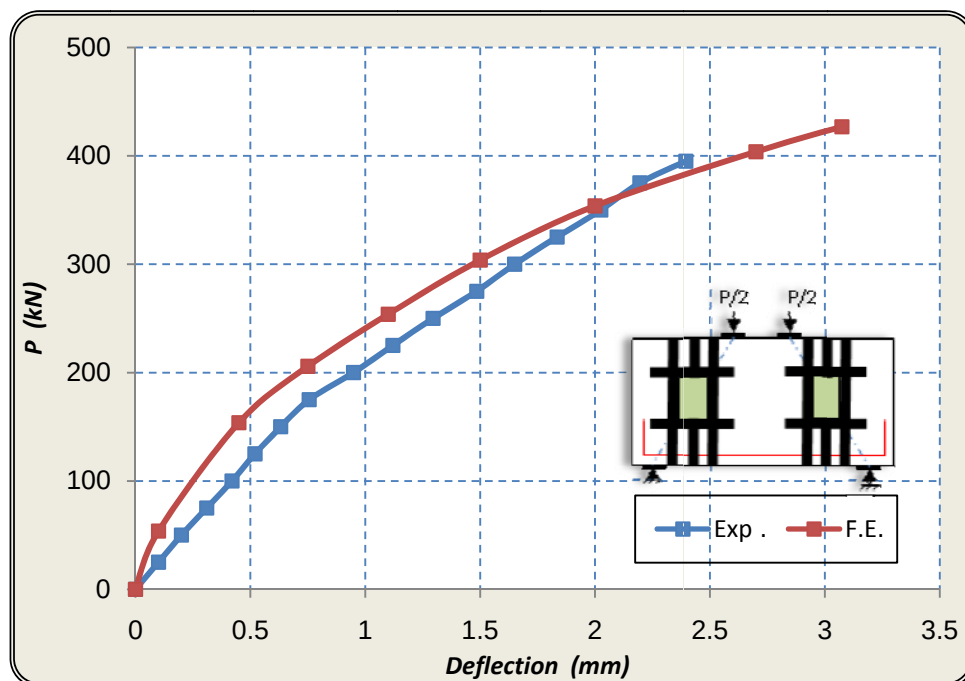


Figure 6.20 Load-Deflection curve for the strengthened Beam (S-3 two face wrap)

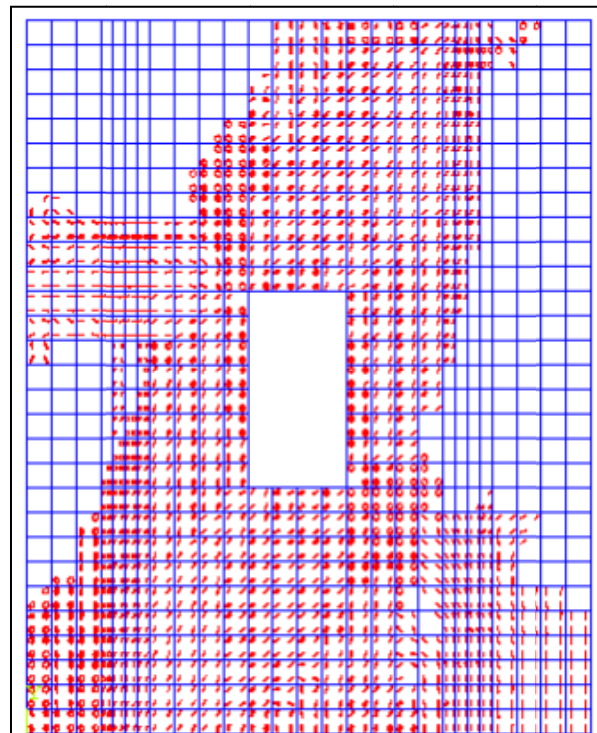


Figure 6.21 crack pattern for S-1 at the ultimate load

Figure 6.22 show the shear stresses in XY-plane development in strengthened beam S-1.

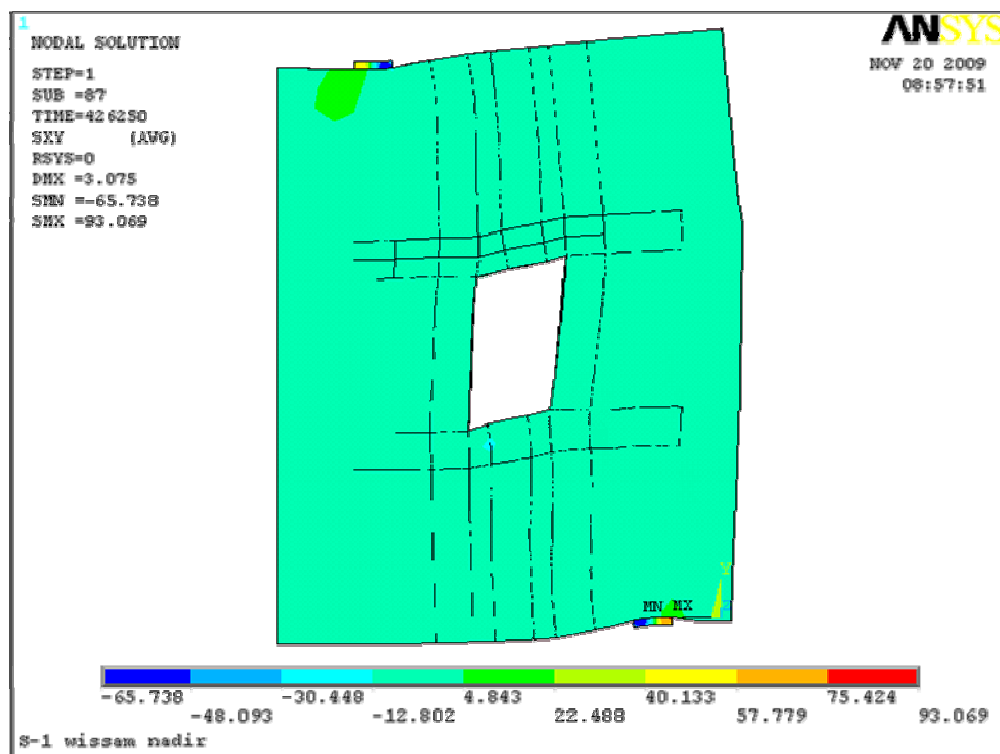


Figure 6.22 Shear stresses in XY-plane at the ultimate load

► Specimen S-4

Figure 6.23 show the comparison between FE and experimental results for specimen S-4.

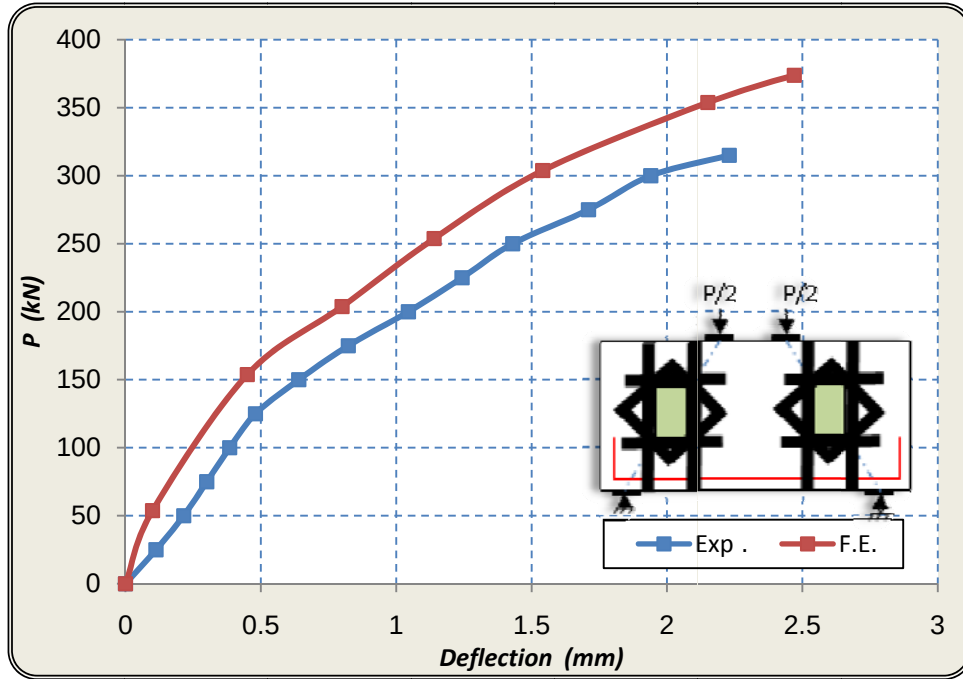


Figure 6.23 Load-Deflection curve for the strengthened Beam (S-4)

Again, load numerical result of specimen S-4 shows stiffer behavior with respect to experimental results at the first stage of loading. This may be attributed to the assumption of full bond is not true in this case because the strips applied in 45° do not have enough length to resist the stresses. The first crack is inclined crack at the corner of the opening in 73.75 kN. Then the cracks are development upward to the supporting and loading points. The ultimate load from the F.E. (373.75 kN) is higher than the experimental results by about 19%.

It can be noticed that the models of strengthening can be delay the crack growth, but, cannot change the formation of the crack; therefore, the crack in all beams with opening strengthening has the same crack pattern.

► Specimen S-5

Figure 6.24 shows the comparison between F.E. and the experimental results for the specimen S-5.

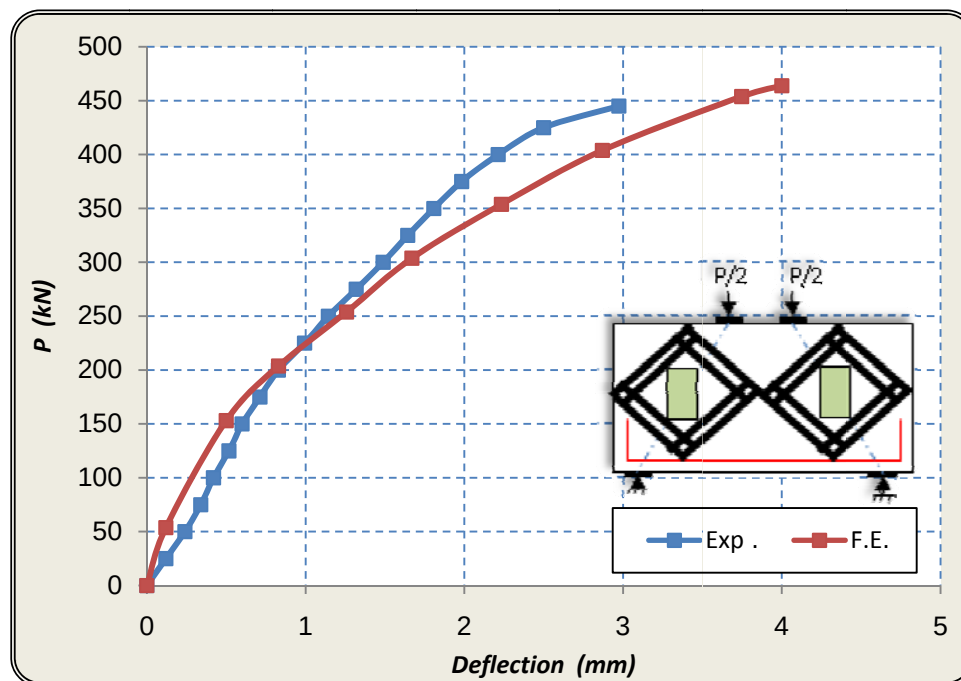


Figure 6.24 Load-Deflection curve for the strengthened Beam (S-5)

The first cracks are appearing at 88.75 kN. In this model the experimental response shows a stiffer response than FE result, where at ultimate load the FE result (463.75 kN) is higher than experimental (445 kN) about 4.2%.

In general, the load-deflection curves for all beams from the F.E. analysis show a good agreement with the experimental data as explained in Table 6.4. Figure 6.25 shows summary of load-deflection curves from F.E. analysis.

Table 6.4 Experimental and Theoretical Ultimate Load and Deflection

Beam Symbol	Failure Load (kN)		Exp./ANSYS (%)	Central Deflection (mm)		Exp./ANSYS (%)
	Exp.	ANSYS		Exp.	ANSYS	
C-1	535	543.75	98.4	2.898	3.348	86.6
C-2	155	167.75	92.4	1.05	1.4	78.6
S-1	415	426.75	97.2	2.95	3.074	96.0
S-2	420	426.75	98.4	2.67	3.074	86.85
S-3	395	426.75	92.6	2.395	3.074	77.9
S-4	315	373.75	84.3	2.38	2.47	96.35
S-5	445	463.75	96.0	3	4	75.0
Average			94.2			85.32

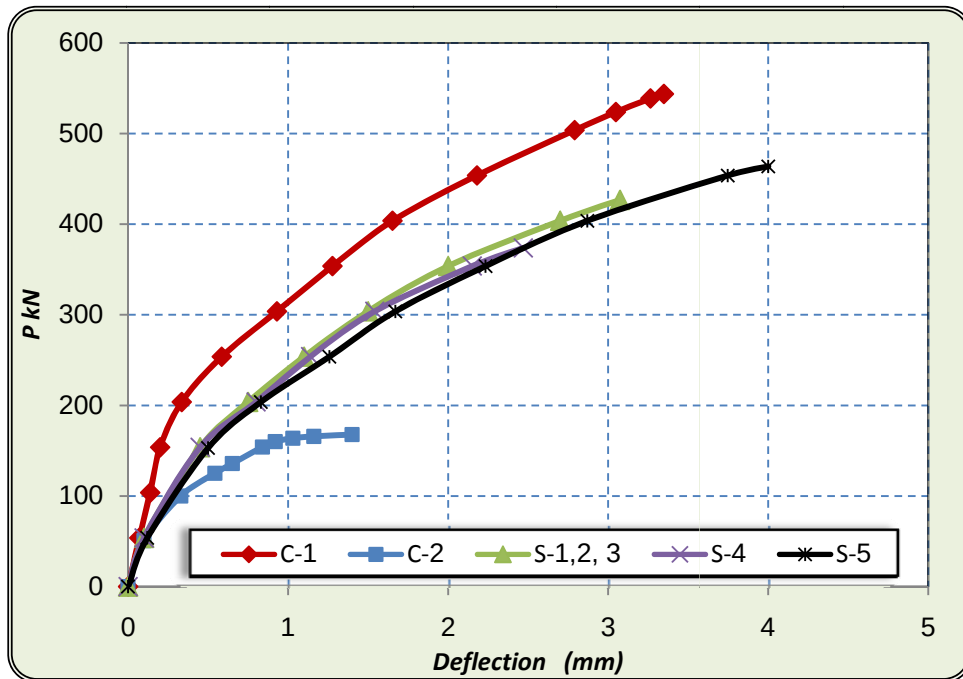


Figure 6.25 summary of load- deflection for all analyzed beams

The finite element load-deflection curve for most beams as explained above showing a stiffer response rather than the experimental results. Microcracks produced by drying shrinkage and handling are presenting in the concrete, these would reduce the stiffness of the actual beam, while the F.E. does not include the effect of microcracks. The F.E. analyses assume that concrete is a homogenous material but, the true it is a heterogeneous material. Also, a perfect bond between the concrete and steel is assumed in the F.E. analysis. However, the assumption would not be true in the actual beam. As the bond slip accrue, the composite action between the concrete and steel reinforcing lost. Therefore, the overall stiffness of the actual beam could be lower than F.E. analyses.

The crack pattern for all beams consistent approximately with the experimental work. This support the activity of the finite element model adopted in the present study.

6.5 Parametric Study

From the reasonable agreement of the finite element model as explained above in 6.4, some selected parameters on the strengthening deep beam with web opening are decided to be studied, include:

- Openings location though zone of shear span.
- Openings Size.
- Openings Shape.

All the beams in parametric study strengthened with model (S-3).

6.5.1 Openings Location

Figure 6.26 shows the distribution of the openings though the shear span with a distance $0.2D$ from the top and bottom chord as recommended by [26] in literature review.

However, this study shows the effect of CFRP sheet on the behavior of deep beam under openings condition with various positions of the openings though the shear span. As shown in Figure 6.27, if the openings interrupt the load path (T_2 , C, B_1) the strength of the beam will be lower than which act out the load path (T_1 and B_2), which is approach from the behavior of the solid deep beams

In addition, it is found that the critical positions of the openings are near the support and at the center of the shear span that deals with the previous study.

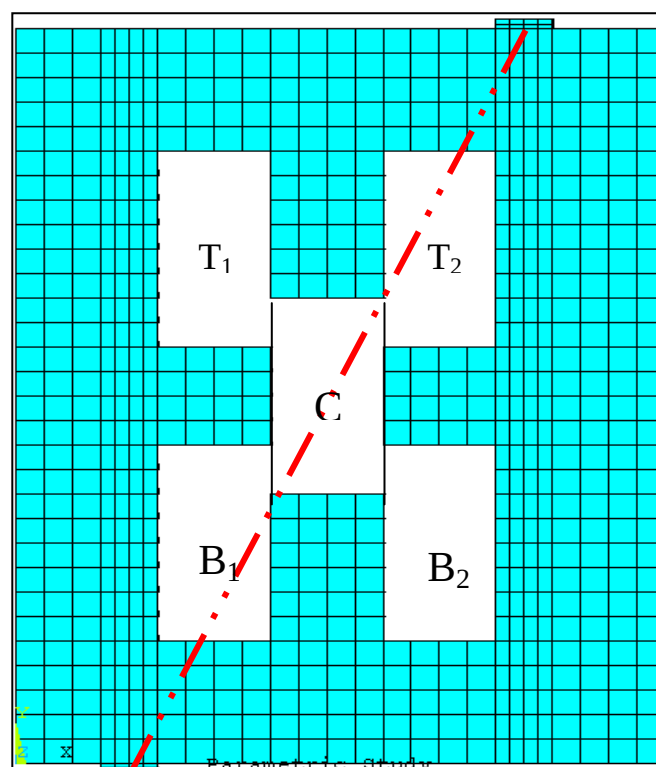


Figure 6.26 Distribution the openings though the shear span (parametric study)

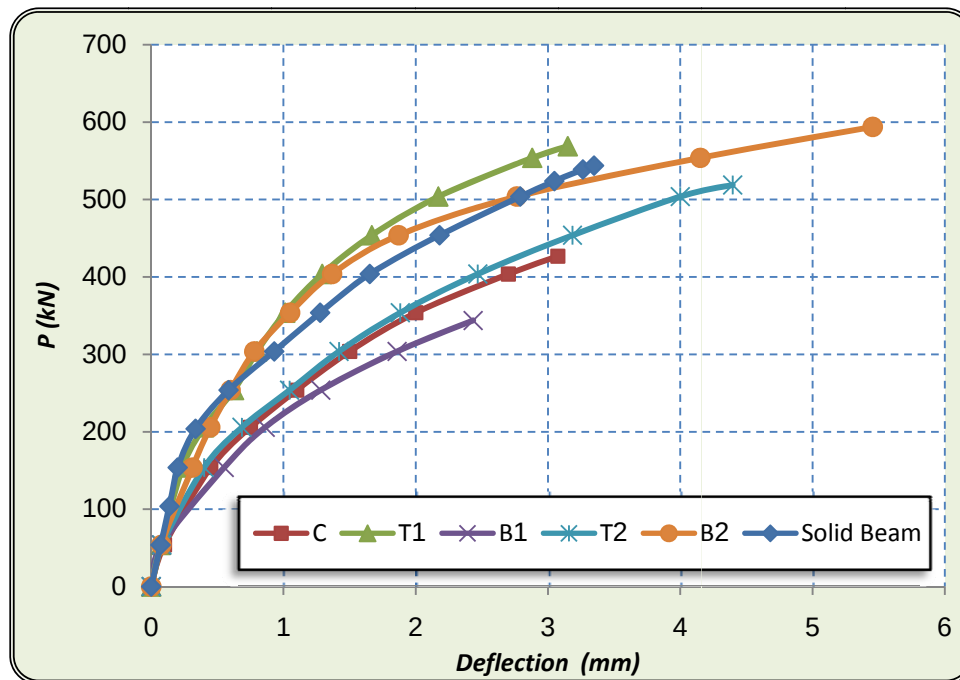


Figure 6.27 Load-Deflection curves for openings location

Table 6.5 shows the comparison of ultimate load between solid deep beam and other cases.

Table 6.5 The ultimate load for the study of opening location

Beam symbol	Numerical failure load kN	Decrease in ultimate load %
C-1	543.75	
C	426.75	21.5
T ₁	568.75	-4.6
T ₂	518.75	5.12
B ₁	343.75	36.78
B ₂	593.75	-9.2

6.5.2 Openings Size

To explain the effect of the opening size on the behavior of strengthened deep beam with web openings, four sizes of opening are provided (100x100, 100x200, 200x200 and 250x250) at the center of shear span.

From Figure 6.28 it can be shown the effect of the openings size on the behavior of the beam.

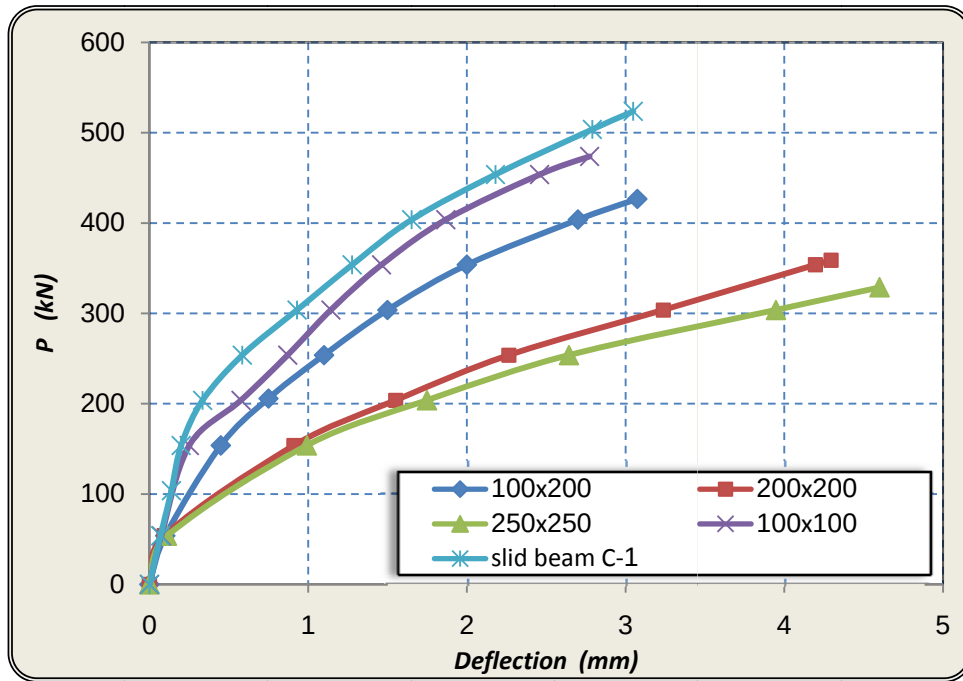


Figure 6.28 Effect of the openings size

From Figure 6.28 it is shown that the ultimate load and effect of CFRP decrease with increasing the opening size. When the area of the openings increased from (100x200) to (200x200) mm (duplicated) the ultimate load decrease about 16.3% while, the ultimate load increased about 10% when the area of the openings decreased to half (100x100)mm with respect to S-3 (100x200). That may be due to the increase interruption of the load path.

6.5.3 Openings shape

To explain the effect of openings shape three types of shapes (rectangular, square and circle) are used with the same position and area (20000 mm^2). As shown in Figure 6.29 the circle openings have the same behaviors at the first stage compared with the other shapes, then it gives a stiff response. That may be due to no corners and the shear stresses which can be transferred through the circumference of the opening. The ultimate load of circle openings (518.875 kN) is higher than square opening (488.75 kN) and rectangular (426.75 kN) by about 5.8% and 17.4 % respectively.

The rectangular openings gave lower ultimate load than other shapes of openings that may be due to the interrupted load path is which longer than other shapes.

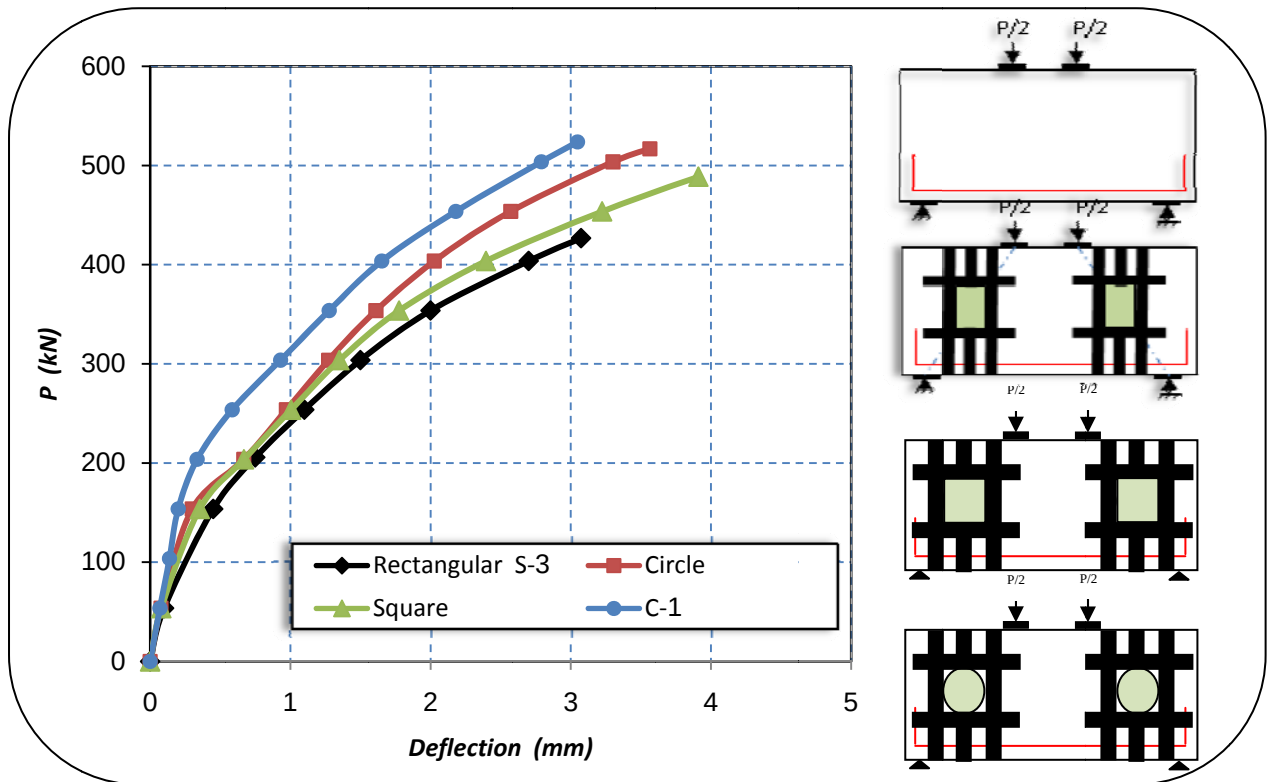


Figure 6.29 Effect of the openings shape

CHAPTER SEVEN

CONCLUSIONS AND RECOMMENDATIONS

7.1 Introduction

Based on the present experimental and theoretical results of R.C. deep beams with web openings strengthening by CFRP laminate made from normal concrete, the following conclusions can be drawn:

7.2 Conclusions

1. The use of CFRP sheets as external strengthening techniques has significant effect on the overall behavior of tested specimen. Where the ultimate load increased by (100-190) %.
2. The comparison between the F.E. analysis and the experimental results asserted the validity of the numerical analysis and the methodology developed. The maximum difference in the ultimate load was less than 10%.
3. The distribution and orientation of the CFRP straps play an important role in upgrading the deteriorated members; therefore, the strengthening specimen by inclined CFRP stirrup (S-5) gives the best result if it compared with the other strengthened models, that may be due to the orientation of the CFRP straps are perpendicular on the inclined crack. While the specimen S-4 is not giving the expected result

4. From the comparison between the strengthened beams S-1(full warp), S-2(U-warp) and S-3(two face warp), it can be observed that there is no significant effect for the vertical anchorage warp if it used for the deep beam.
5. It is noted that the strengthening of deep beams with web openings by CFRP decrease the crack width. Nevertheless, has no effect on the formation of inclined crack.
6. CFRP in most cases of strengthened beams are neither rupture nor debonding. But, a peel of concrete splitting with CFRP straps. It mean that all strengthened model used in the present study are successful in upgrading such situation.
7. The behavior and the ultimate load of the strengthened deep beam with web openings depend on the interruption of the load path; therefore, when the opening located away from the supports and the load path, the ultimate load increased by about (36.0, 21.0 and 24.0) % with respect to that located on the load path.
8. The circle shape of openings gives a higher ultimate load than square and rectangular strengthened openings by 5.8% and 17.4% respectively
9. Increasing the opening size leads to decrease the ultimate load and increase the deflection in spite of strengthened by CFRP straps. When the opening size increased from (100x200) mm to (200x200 and 250x250) mm the ultimate load decreased about (16.3 and 23.3) % respectively.
10. The crack patterns at the final load from the finite element model correspond well with the observed failure of the experimental results.

7.3 Recommendations for the Future work

1. Experimental investigation of the behavior of R.C. continuous deep beam with web openings strengthening by CFRP.
2. Experimental investigation full scale deep beams with web openings.
3. Examine the behavior of deep beam with web openings strengthening by CFRP sheet under dynamic load.
4. Experimentally investigation of the effect of CFRP sheet in upgrading the deep beams with web openings in the presence of steel shears reinforcement.
5. Theoretical investigations of the bond slip between the concrete and CFRP sheet.

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الخلاصة

يتم استحداث الفتحات في الأعتاب الخرسانية لتجهيز الخدمات مثل قنوات التبريد والأنابيب أو الأبواب و الشبائيك. على الرغم من ذلك , تجهيز الأعتاب الخرسانية العميقة بالفتحات يسبب تدهور مفاجئ في سعة القص. السبب في ذلك يعود إلى تركيز الإجهاد في زوايا الفتحات و انقطاع مسار الحمل عبر دعامة الضغط (compressive strut).

إن هذا البحث يتضمن دراسة عملية ونظرية لاستخدام ألياف الكربون البوليمرية لتقوية الأعتاب الخرسانية العميقة الحاوية على فتحات.

تم فحص ثمان أعتاب خرسانية عميقة ذات مقطع مقطع (750 x100) ملم و بطول (1150) ملم فحصت تحت تأثير اربع نقاط حمل. سبعة نماذج كانت تحتوي على فتحتان مستطيلة ذات مقطع (200x100) ملم, موزعة على شكل واحدة في وسط منطقة القص و متناظرة حول خط الوسط. النموذج الأول ترك بدون فتحات و النموذج الثاني احتوى على فتحات استخدم كمصدر للمقارنة النتائج. أما النماذج البقية تم تقويتها باستخدام ألياف الكربون البوليمرية. تضمنت المتغيرات شكل, اتجاه ونظام التثبيت العمودي.

النتائج العملية أشارة الى إن استخدام ألياف الكربون البوليمرية لتدعيم الأعتاب الخرسانية العميقة الحاوية على فتحات لها فعالية ايجابية على التصرف الكلي مثل قابلية التحمل القصوى, الشقوق و الهطول. نسبة الزيادة في قابلية التحمل القصوى كانت تتراوح بين (100-190)% و كذلك وجد إن نظام التثبيت العمودي المستخدم (full warp, U-warp and two face warp) لم يعزز مقاومة القص إذا ما قورنت مع بعضها البعض (S-1, S-2 and S-3). بينما اظهر نظام التقوية بشكل مائل (S-5) استجابة اصلب مقارنة مع موديلات التقوية الأخرى. حيث أعطى هذا الموديل زيادة 190% في قابلية التحمل الأقصى مقارنة بالمصدر (C-2) و 8.5 % مقارنة بالنموذج (S-1). كذلك استخدام ألياف الكربون البوليمرية كنظام تقوية خارجي يقلل من عرض الشق و لكنه لا يستطيع تغيير شكل الشق المائل. جميع النماذج المفحوصة فشلت من خلال الانقسام إلى جزئين خلال الشق المائل.

من جهة أخرى تم استخدام التحليل الأخطي بواسطة عناصر ثلاثية الأبعاد لتحري أداء العتبات الخرسانية المسلحة المقواة بألياف الكربون البوليمرية باستخدام البرنامج الحاسوبي (ANSYS V.9.0). تم افتراض وجود ربط تام بين الألياف الكربونية و سطح التماس مع الخرسانة وكذلك بين حديد التسليح و الخرسانة. العنصر الطابوقي (Solid65) استخدم لتمثيل الكونكريت بينما استخدم العنصر (Link8) و العنصر (Shell41) لتمثيل حديد التسليح و ألياف الكربون البوليمرية على التوالي. المقارنة بين النتائج العملية والنظرية أكدت صلاحية التحليل العددي بشكل واضح حيث كانت النتائج معقولة.



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

التحري العملي و النظري لتصرف الاعتبات الخراسانية العميقة الحاوية على فتحات و المقواة بألياف الكربون البوليمرية

رسالة

مقدمة الى كلية الهندسة في جامعة بابل
كجزء من متطلبات نيل درجة ماجستير في علوم الهندسة المدنية
(إنشاءات)

من قبل

وسام نادر نجم

بكالوريوس هندسة مدنية 2006

إشراف

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