Nonlinear Behavior of Reinforced Concrete T-Beams Strengthened With CFRP Subjected To Shear

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Abstract

This research is devoted to investigate the behavior and performance of RC beams subjected to shear and strengthened with CFRP sheets. The experimental program consisted of eight tested beams; two control beams, the first was without any shear reinforcement and the second was reinforced by steel stirrups, five beams was strengthened by CFRP sheets and the eighth beam was repaired after it had been loaded up to service load. The variables investigated in this work included the CFRP amount, orientation, distribution and anchorage of the end of the CFRP sheets. All beams had been tested as a simply supported span and subjected to two-point loading.

The experimental test results showed that the ultimate load capacity of the strengthened beams ranged between 30% to 45% over the ultimate load capacity of the reference (unstrengthened) beam. The ultimate load of repairing beam reached 96% of the ultimate load of similarly strengthened beam. Drilling the flange of the RC beams to wrap the CFRP strips was the most effective and significant factor for strengthening RC beams.

The use of ANSYS computer program (version 9.0, 2004) to create the finite element model is adopted. Interface element (CONTAC52) was used in the finite element model of RC beam that was strengthened by U-wrap CFRP strips to simulate the bond between the CFRP strips and concrete surface, while the strengthened RC beams by full wrap CFRP strips, full bond was assumed. The comparison between the numerical and the experimental results asserted the validity of the numerical analysis, the average difference of central deflection and ultimate load between experimental and numerical results were 4.3% and 4.7% respectively for all the tested and analyzed beams.

الخلاصة

ان الغرض من هذا البحث هو التحري عن تصرف و اداء العتبات الخرسانية المعرضة للقص المسلحة و المقوات بالشرائح البوليمرية المسلحة بالياف الكاربون و ذلك من خلال تقديم دراسة تحليلية و مخبرية للموضوع. تألف البرنامج العملي من ثمانية اعتاب، عتبان مصدران، الاول لم يحتوي على اي تسليح قص اما الثاني فسُلِحَ باترية حديدية. خمسة منها تم تقويتها بالياف الكاربون البوليمرية اما العتب الثامن فقد تم اعادة تصليحه بعد ان تم تحميلة الى حد الاحمال الخدمية. المتغيرات التي تم التحري عنها في هذة الدراسة تضمنت كمية الياف الكاربون البوليمرية و توجيهها و توزيعها و التثبيت الميكانيكي الموجودة في نهاية الياف الكاربون البوليمرية. جميع الاعتاب تم فحصها تحت ظروف الأسناد البسيط و المعرضة للتحميل ذي النقطتين.

تؤكد النتائج المختبرية ان سعة الحمل الاقصى للاعتاب المقواة تراوحت بين 30% الى 45% اعلى من الحمل الاقصى للعتب المصدري(غير المقوى). الحمل الاقصى للعتب المعاد تصليحه وصل الى 96% من الحمل الاقصى لمثيله من العتب المقوى. تثقيب شفة العتب الخرساني لتمرير شرائح الياف الكاربون البوليمرية كان عاملا مؤثرا و مهما في تقوية الاعتاب الخرسانية.

استعمل البرنامج الحاسوبي ANSYS (النسخة 9 ، 2004). تم افتراض ربط تام بين شرائح الياف الكاربون البوليمرية و السطح تماس الخرسانة للعتبات التي تم تقويتها باللف الكامل لشرائح الياف الكاربون البوليمرية، ما عدا واحدا و الذي تم تقويته بها على شكل U، تم استخدام الربط الجزئي. المقارنة بين النتائج النظرية والعملية اكدت صلاحية التحليل العددي حيث كانت أكبر نسبة فرق في المقاومة القصوى و اكبر هطول أقل من 4.3%، 4.7% بالتتابع لكل العتبات المفحوصة والمحللة.

1. Introduction

There have been techniques for strengthening almost as long as structures have existed. At ancient times when there was very limited structural knowledge structures were strengthened by insertion of extra members, supports or increased dimensions, methods that still are used today. As building knowledge has advanced, the strengthening techniques have become more sophisticated (Carolin A., 1999). It is important to stress that it is often more complicated to strengthen a structure than erecting a new one.

Concerns must be taken to existing materials, often in deteriorated condition, loads during strengthening and to existing geometry. In some cases it can also 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 can a flexure strengthening lead to a shear failure instead of giving the desired bearing capacity (Sharif A., et al., 1994). It should also 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. Many structures have, for instance, to withstand heavy loads and de-icing salts as well as large and many changes in humidity and temperature over a long time. These demands must be kept in mind when the structure is upgraded.

The strengthening should be designed with consideration to minimize the maintenance and repair needs. Due to the different advantages and drawbacks of existing methods, designers must closely evaluate all of the alternatives including the possibility that strengthening may not be the best choice. Finally, it is not only the economical and structural aspects that should form the basis for decisions of strengthening and choice of strengthening method, but environmental and aesthetic aspects must also be considered (Carolin A.,1999). Another subject that must be considered when a strengthening of a structure is designed is the consequences from loss of strengthening effectiveness by fire, vandalism, collision etc.

The most efficient technique for improving the shear strength of deteriorated RC members is to externally bond fiber-reinforced polymer (FRP) plates or sheets. Steel plates have traditionally been the most common strengthening material, but its use in concrete structures has gradually declined because of its poor workability in-situ and mechanical characteristics, including corrosion and heavy weight, even though its material properties are similar to steel bars. For the last 20 years, researchers have concentrated on developing FRP as an alternative strengthening material to steel plates. External bonding techniques for RC beams are useful for rehabilitating flexural and/or shear strength (Gyuseon Kim, et al., 2007).

2. Experimental Work

2.1 Specimens

Eight shear tests are reported. Of these tests, seven are without steel stirrups and one with stirrups. The beams were designed to have extra strength in flexure to ensure shear failure even after strengthening. Table 1 presents characteristics of the beams tested and parameters investigated. Detail of the beam without shear reinforcement **BC1** is presented in Figure 1. Detail of the beam with shear reinforcement **BC2** is presented in Figure 2.

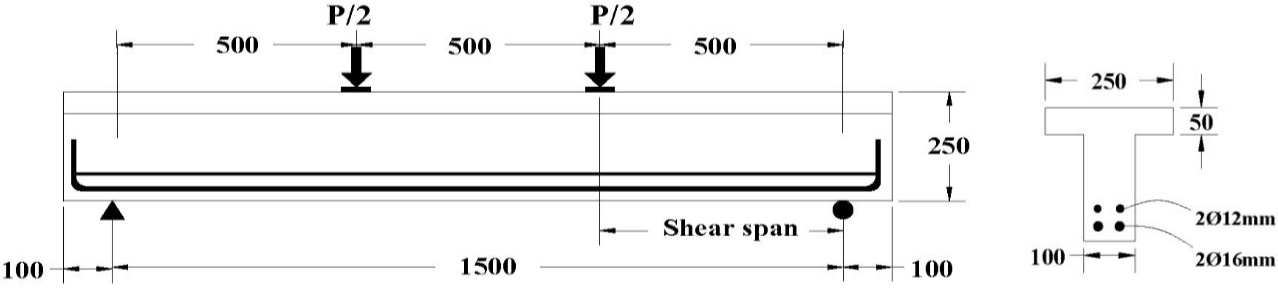


Figure 1: *Details of the beam without shear reinforcement (All dimensions in mm)*

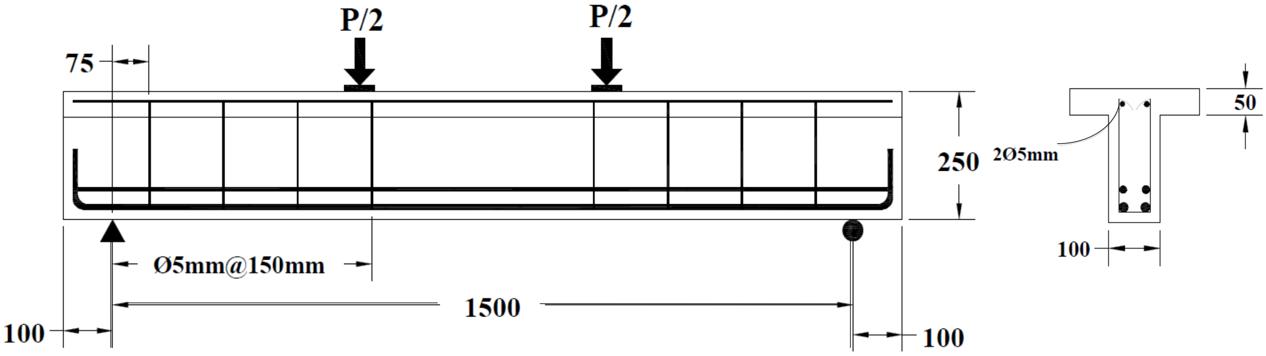


Figure 2: *Details of the beam with shear reinforcement (All dimensions in mm)*

Table 1: *Characteristics of Beams Tested and Parameters Investigated*

No.	Specimen designation	Orientation of CFRP strips(°)	Width of CFRP strips(mm)	Spacing of CFRP strips(mm)
1	BC1	- - - - -	- - - - -	- - - - -
2	BC2	- - - - -	- - - - -	- - - - -
3	BS1*	90°	21.6	150
4	BS2	90°	21.6	150
5	BS3	45°	21.6	150
6	BS4	90°	32.4	150
7	BS5	90°	21.6	100
8	BR**	90°	21.6	150

*Beam specimen **BS1** was strengthened by U-wrap strips and all the other beams were strengthened by full wrap strips.

**Repaired beam specimen.

2.2 Materials

Ordinary Portland cement (Type I) was used in casting all the specimens. Gravel of (19mm) maximum size was used. The average of compressive strengths for all tested beams was 20.2 MPa, while the average of tensile strengths was 2.06 MPa. Three tension bars of each 16mm, 12mm and 5mm diameter deformed steel bars were tested to acquire the yield stress and ultimate strength of the reinforcement, see table 2. Sikadur-330 was used as a bonding material.

Table 2: *Material Properties for Steel Reinforcement*

Properties	D-16mm	D-12mm	D-5mm
Yield stress, MPa	500	500	520
Ultimate strength, MPa	650	650	650

The mechanical properties of CFRP sheets used here were taken from manufacturing specifications (Sika, 2005) and are given in Table 3.

Table 3: *Technical Properties of CFRP Sheet (Sika, 2005)*

Properties	Tensile strength (MPa)	E-modulus (GPa)	Elongation at break (%)	Width (mm)	Thickness (mm)
Sika Wrap Hex 230C	4300	238	1.8	600	0.131

2.3 Surface Preparation

Before the CFRP was applied to the soffit of the beams, the surface of the concrete is grinded using an electrical hand grinder to expose the aggregate and to obtain a clean sound surface, free of all contaminants such as cement laitance, and dirt (see Figure 3). Wrapped corners rounded to be a minimum radius of 20 mm.

2.4 Drilling the Flange

Holes were made in the flange with approximately CFRP strip width by using hand drill machine with drill bit of the size (6 mm in diameter) to wrap the beam specimen with the CFRP strips, see Figure 4.

Figure 3: *Concrete surface preparation by electrical grinder*

Figure 4: *Process of the drilling*

2.5 Application of CFRP Composites

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

1. A two-part adhesive (black and white) was mixed in required proportion, until the color was a uniform grey, and then applied to the concrete surface to a thickness of 1.5 mm approximately. The adhesive was also applied to the sheet using the same thickness.
2. The sheet was then placed on the concrete, epoxy to epoxy, and a rubber roller was used to properly seat the sheet by exerting enough pressure so the epoxy was forced out on both sides of the sheet, see Figure 5. Removing the excess epoxy from sides of sheets be done.
3. After the CFRP installation was complete, holes that made in the flange were filled with epoxy, the CFRP strips were cured at ambient temperature for at least 7 days before testing.

Figure 5: *Application of CFRP sheets*

2.6 Testing Machine

Figure 6 shows the hydraulic machine which used to test all beam specimens.

Figure 6: *Test set-up of beam specimen*

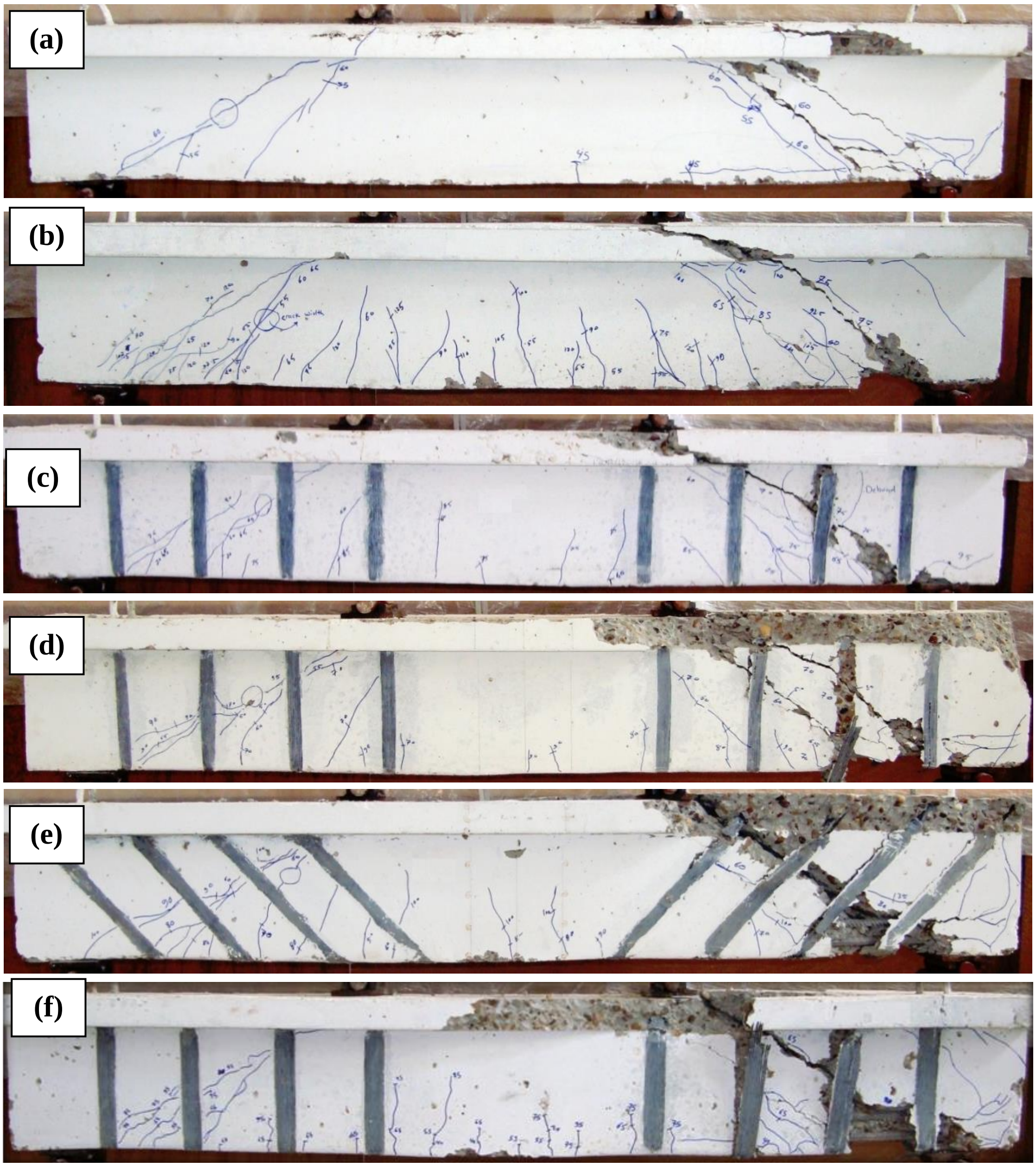
3. Experimental Results

3.1 Ultimate Loads and Failure Modes

All the beam specimens were tested up to failure. The recorded ultimate loads and failure modes for these beam specimens are presented in Table 4. Figure 7 and Figure 8 show the beam specimens after failure. Figure 9 shows the debonding of CFRP strip for beam **BS1**.

***Table 4**: Ultimate Loads and Failure Modes of the Beam Specimens*

Specimen	Ultimate Load , kN	Increase in Ultimate Load over BC1,%	Failure Mode
BC1	99	――――	Diagonal shear failure
BC2	120	20	Diagonal shear failure
BS1	95	――――	Deboning of CFRP
BS2	130	30	Diagonal shear failure
BS3	140	40	Diagonal shear failure
BS4	138	38	Diagonal shear failure
BS5	145	45	Flexural failure
BR	125	25	Diagonal shear failure



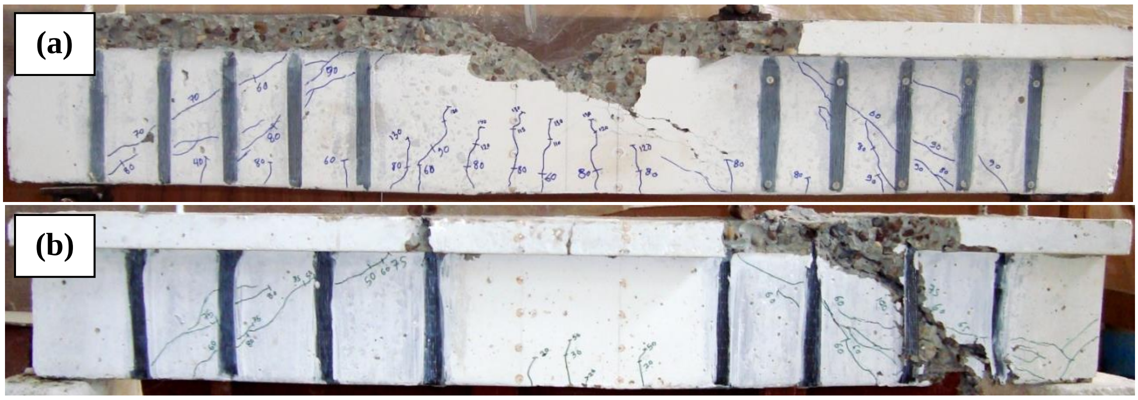


Figure 8: *Crack pattern after failure for Beam specimen: (a) **BS5** (b) **BR***

Figure 9: *Debonding failure*

3.2 Load Deflection Curves

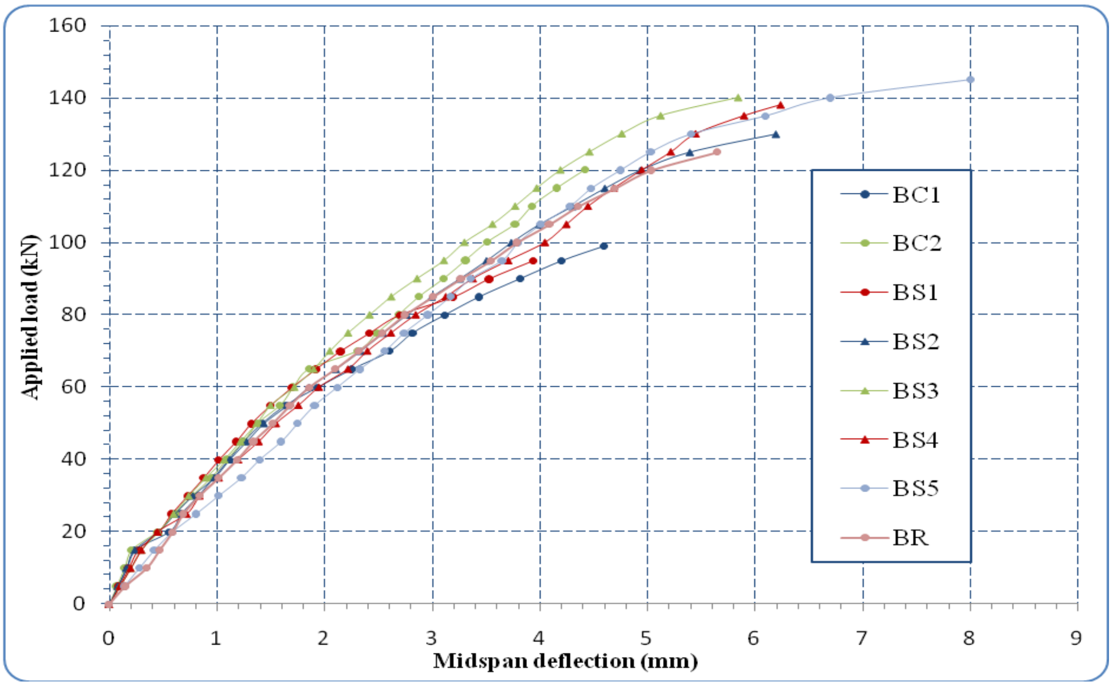


Figure 10: *Experimental load-deflection for all beam specimens*

3.3 Concrete Cracking

In the present study, the diagonal shear cracks were measured by using the crack-meter. Figure 11 shows the development of inclined crack width in control, strengthened and repaired beams. The repaired beam specimen developed from load 30kN because it was already cracked.

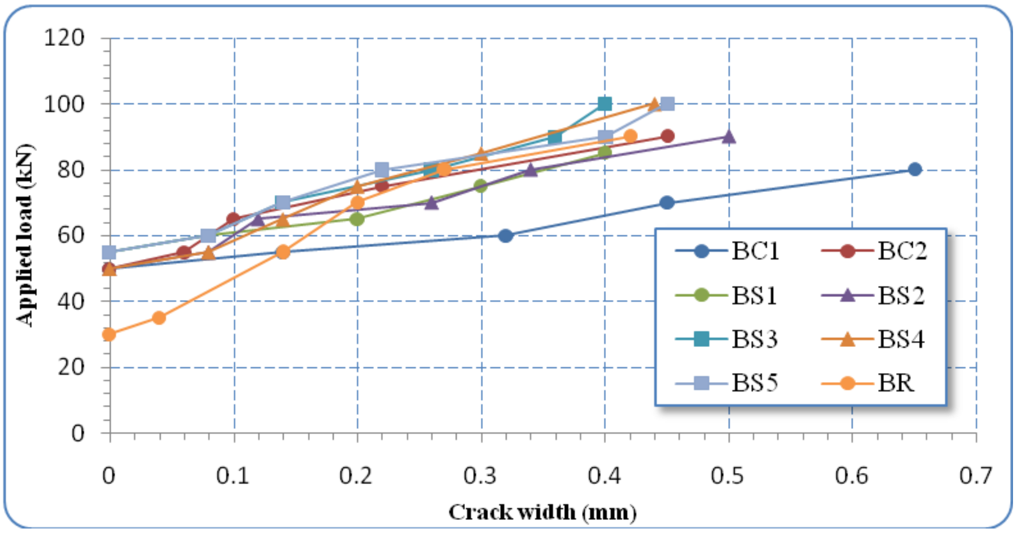


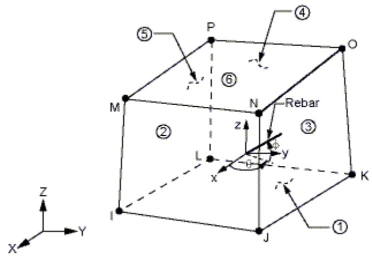
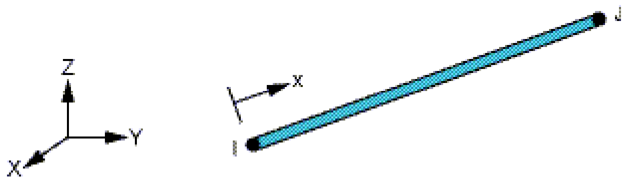
Figure 11: *Crack width versus applied load for all beam specimens*

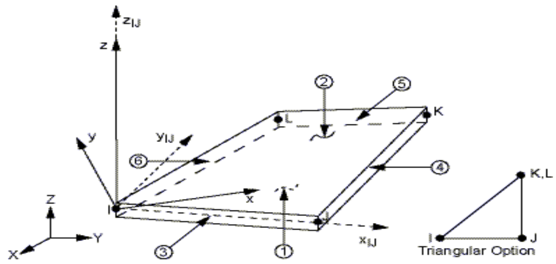
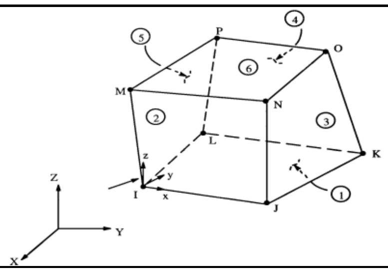
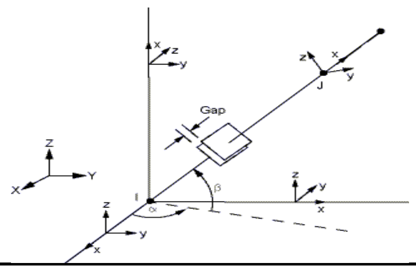
4. Finite Element Modeling

The finite element method has become a powerful tool for the numerical solution of a wide range of engineering problems, so the use of ANSYS-9 to create the finite element model is adopted.

4.1 Element Types

Table 5: *Element Types and their Geometry*

Material	Element Type	Geometry
Concrete	**SOLID65**	
Steel Reinforcement	**LINK8**	

CFRP Sheets	**SHELL41**	
Steel Plates	**SOLID45**	
Interface Element	**CONTAC52**	

By use the bilinear model (X.Z. Lu, at el., 2004), $\tau_{max} = 3.687\,\frac{\mathbf{N}}{\mathbf{m□^2}}$ and $s_0 = \mathbf{0.04□9\ mm}$, so the sticking stiffness will be:

$$\mathbf{KS} = \frac{\text{max.}\,\mathbf{bond\ stress}\ *\ \mathbf{area\ of\ CFRP\ el□ment}/_{\mathbf{2}}}{\mathbf{slip}} = \mathbf{14340}\,\frac{\mathbf{N}}{\mathbf{mm}}$$

The normal stiffness KN=1 was assumed. CONTAC52 was used just in model of beam **BS1** to simulate the bond between the CFRP strips and concrete surface because it was very effective factor and it was clear in the results.

4.2 FEM Input Data

4.2.1 Concrete

The elastic modulus of concrete for each beam model was calculated according to (ACI Committee 318, 2008) by using Equation $E_c = 4700\sqrt{f_c'}$. Poisson's ratio for concrete was assumed to be 0.16 for all reinforced concrete beams.

The shear transfer coefficients used in this study were 0.15 for an open crack and 0.5 for a closed crack for all strengthened beam specimens except the control beam specimen (without shear reinforcement), the shear transfer coefficients $\beta_□$ and $\beta_□$ were 0.1 and 0.5 respectively.

4.2.1.1 Compressive Uniaxial Stress-Strain Relationship for Concrete

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 (Willam, K.J., and Warnke, E.P. 1974).

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

$$\varepsilon_{\circ} = \frac{2 f_c'}{E_c} \text{-----------------------} \quad \text{(Eq. 2)}$$

$$\mathbf{E_c} = \frac{\sigma}{\varepsilon} \quad \text{---} \quad \text{(Eq. 3)}$$

σ = stress at any strain ⍰

ε = strain at stress ⍰

$\varepsilon_{⍰}$ = strain at ultimate compressive strength

The multilinear isotropic stress-strain implemented requires the first point of the curve to be satisfying Hook law.

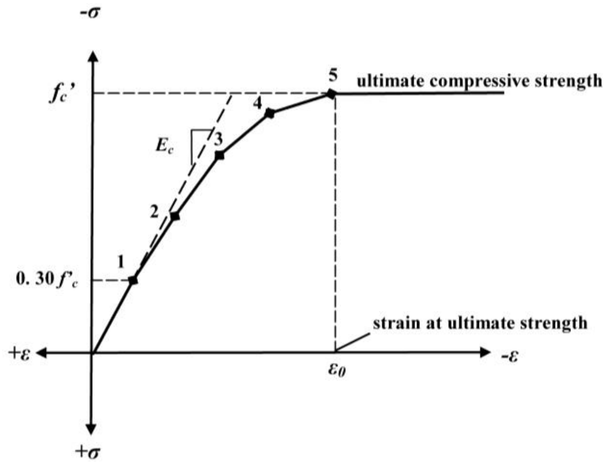


Figure 12: *Uniaxial Stress-Strain Curve (Willam, K.J., and Warnke, E.P. 1974)*

4.3 Modeling of RC Control and Strengthened Beams

A quarter of the full beam was used for modeling by taking advantage of the symmetry of the beam and loadings. At a plane of symmetry, the displacement in the direction perpendicular to that plane was held at zero.

A convergence study on quarter model of the beam was carried out to determine an appropriate mesh density. The beams of same material properties were modeled with an increasing number of elements 900, 1186, 2040, 2876 and 5332. The midspan deflection for all beams was observed for the same applied load of 5kN, see Figure 13.

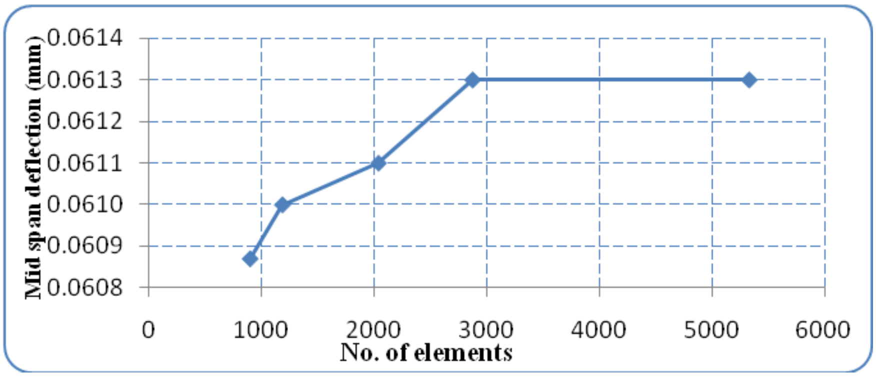


Figure 13: *Plot for convergence study*

From the graph it was found that model with number of elements more than 2876 had negligible effect on mid span deflection. So it was used for this entire study. Figure 14 shows a typical quarter symmetry finite element model for the beam specimen.

The strengthened beam specimen **BS1** failed by debonding of CFRP strips, therefore Interface Element (CONTAC52) was used, it wasn't used in the other strengthened beams, but full bond between CFRP strips and concrete surface was assumed. Figure 15 shows the finite element model of strengthened beam **BS1**.

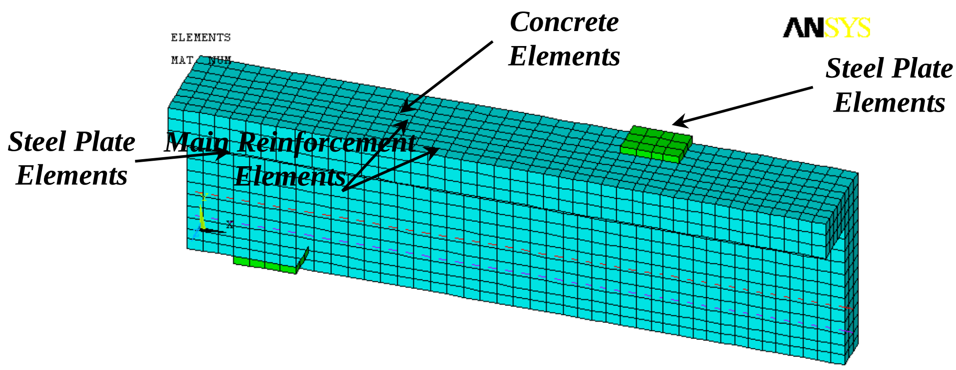


Figure 14: *Typical quarter symmetry finite element model*

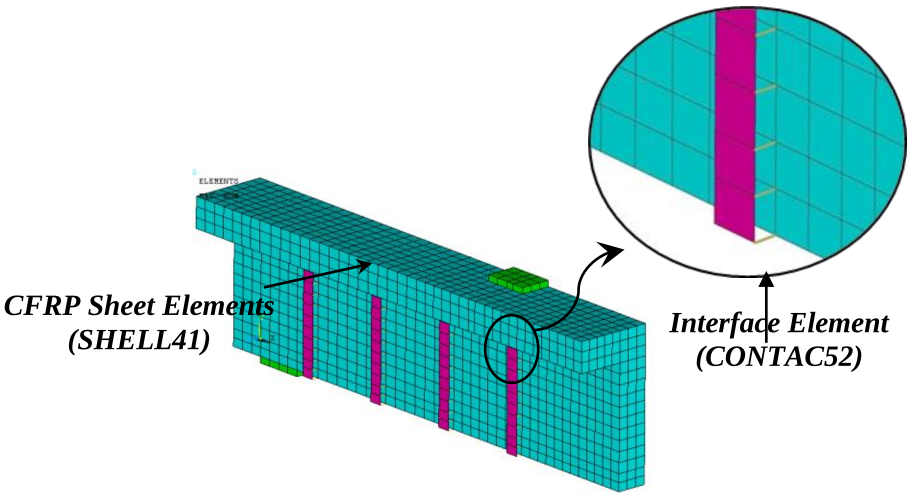
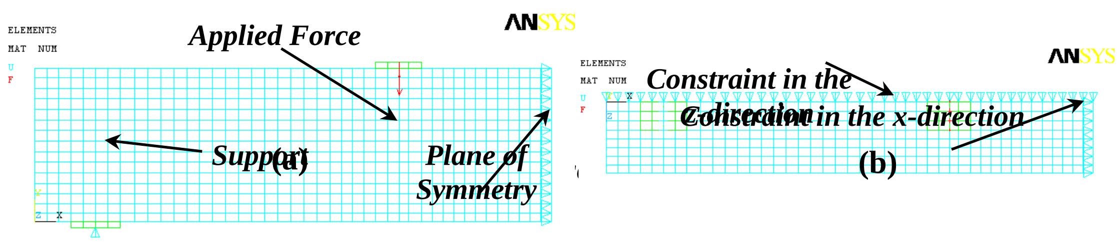


Figure 15: *Finite element model of strengthened beam* ***BS1***

4.4 Loading and Boundary Conditions

In the experiment, the loading and support dimensions were (100mm x 80mm). A one-cm thick steel plate, modeled using Solid45 elements, was added at the support and point of loading locations in order to avoid stress concentration problems. This provided a more even stress distribution over the support and point of loading areas. Moreover, a single line support was placed under the centerline of the steel plate to allow rotation of the plate. Figure 16 shows the applied load and boundary conditions. Due to a quarter of the full beam was modeled the applied load on it will be equal to ¼ from the total applied load. Figure 17 shows the distribution of applied load at nodes.



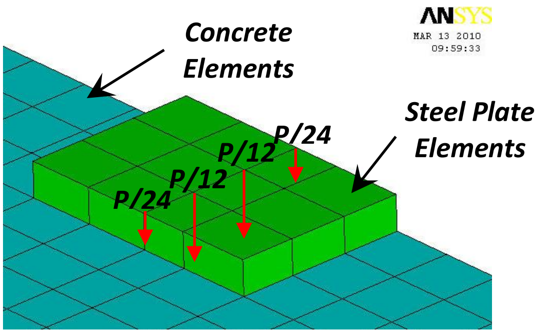
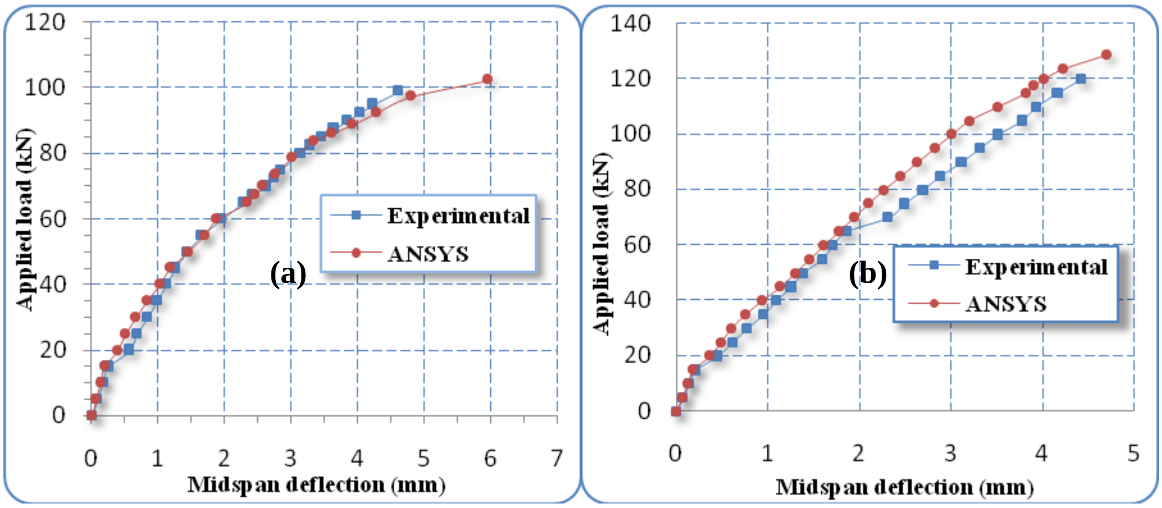


Figure 17: *Distribution of applied load at nodes*

4.5 Results from Finite Element Analysis

The results from the ANSYS-9 finite element analyses were compared with the experimental data.



(a) ***BC1*** *(b)* ***BC2***

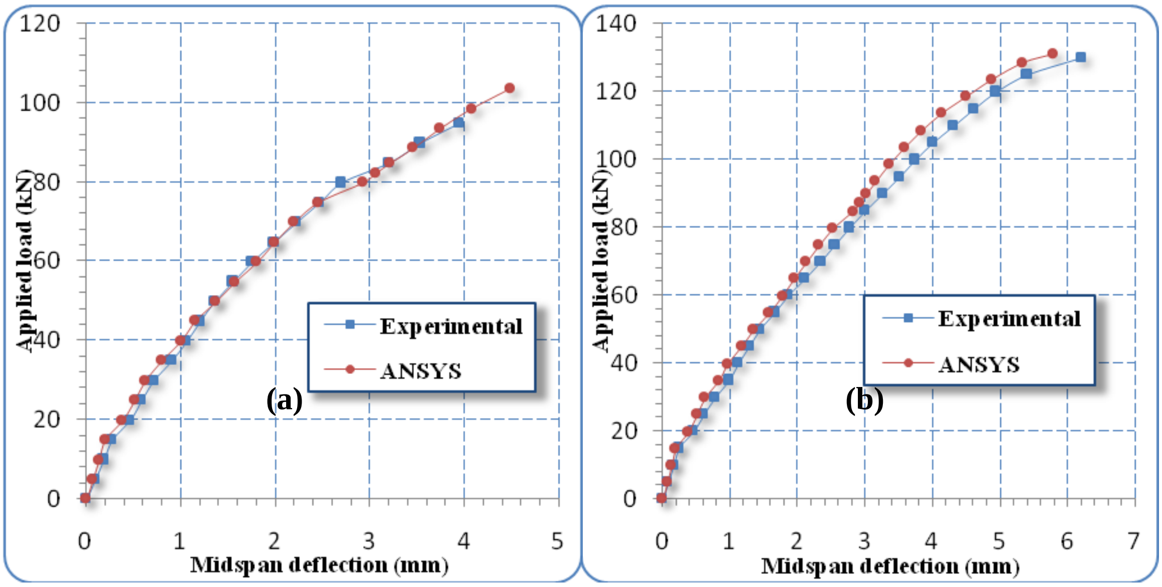


Figure 19: *Numerical and experimental load deflection relationship for strengthened beam (a)* ***BS1*** *(b)* ***BS2***

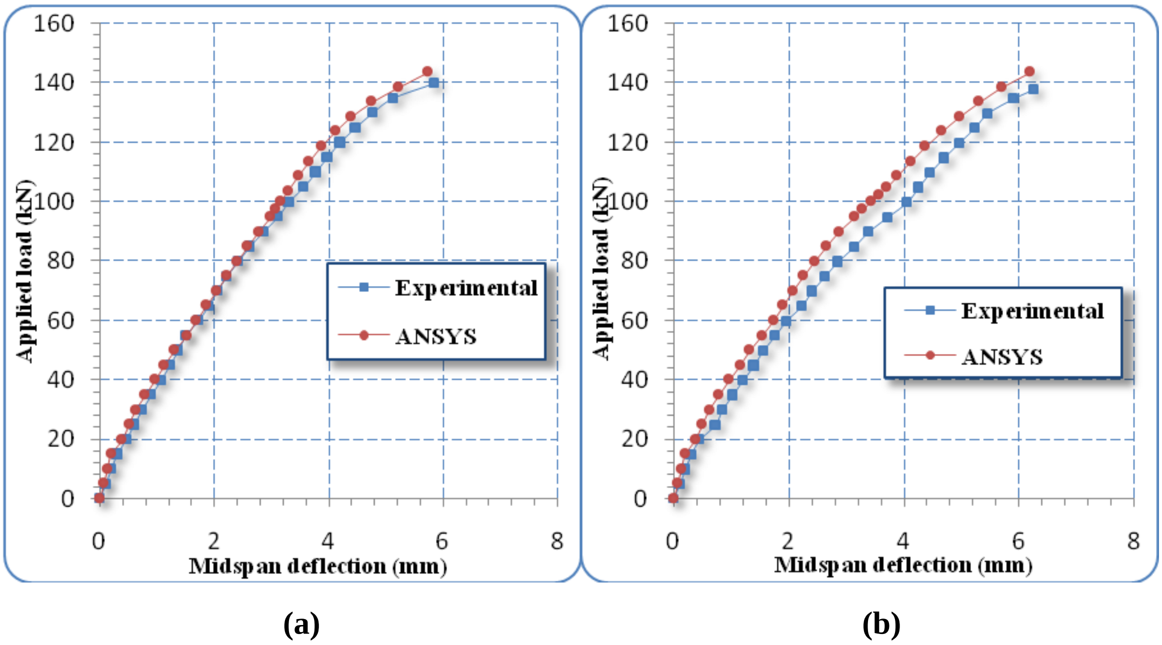


Figure 20: *Numerical and experimental load deflection relationship for strengthened beam (a)* ***BS3*** *(b)* ***BS4***

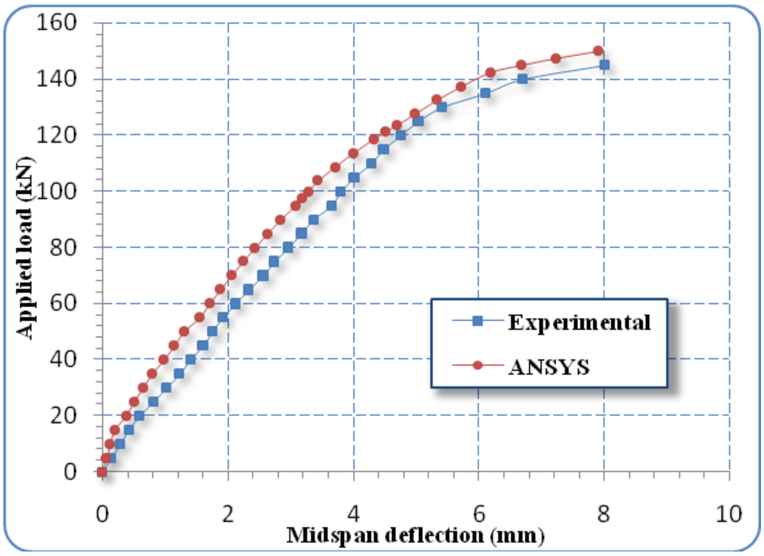


Figure 21: *Numerical and experimental load deflection relationship for strengthened beam* ***BS5***

Table 6 shows the failure load and its central deflection for each beam. The average difference of central deflection and ultimate load between experimental and ANSYS9 program was 4.3% and 4.7% respectively for all the tested and analyzed beams.

There are several factors that may cause the higher stiffness in the finite element models. Microcracks produced by drying shrinkage and handling are presenting in the concrete to some degrees. These would reduce the stiffness of the actual beams, while the finite element models do not include microcracks. Perfect bond between the concrete and steel reinforcing are assumed in the finite element analyses, but this assumption would not be accurate for the actual beams.

Table 6: *The Experimental and ANSYS Deflection Values of the Beams at the Failure Load.*

Beam Specimen	Failure Load (kN)		Central Deflection (mm)		Exp./ANSYS	
	Experimental	ANSYS	Experimental	ANSYS	Failure Load	Central Deflection
BC1	99	102.5	4.6	5.95	91	77
BC2	120	128.75	4.42	4.69	93	94
BS1	95	97.5	3.94	4.65	97	85
BS2	130	131.25	6.2	5.78	99	107
BS3	140	143.75	5.84	5.71	97	102
BS4	138	143.75	6.24	6.17	96	101
BS5	145	150	8	7.9	97	101

4.6 Maximum Stresses in CFRP Composites

For the actual strengthened Beams, there was no evidence that the CFRP reinforcing failed before overall failure of the beams. This was confirmed by the finite element analyses. The maximum tensile stress was 2592 MPa, while the ultimate tensile strength for the CFRP sheets is 4300 MPa.

5. Conclusions

1. The experimental test results confirm that the strengthening technique of CFRP system is applicable and can increase the shear capacity for strengthened and repaired of RC beams. In this study, the ultimate load capacity of the strengthened beams ranged between 30% to 45% over the ultimate load capacity of the reference (unstrengthened) beam.
2. Same behavior for strengthened as well as repaired beam was noticed except that the ultimate load in repaired beam reached (96%) of the ultimate load of similarly strengthened beam.
3. The strengthening by U-wrap CFRP strips did not increase the ultimate bearing capacity over the reference beam due to debonding of CFRP strips before the beam collapse.
4. Drilling the flange of the T-beam to wrap the CFRP strips was the most effective and significant factor for strengthening RC beams.
5. The inclined CFRP strips gave better enhancement than the vertical CFRP strips in ultimate load, deflection and crack width.
6. For the same amount of shear reinforcement ratio (by CFRP), decreasing the spacing between the CFRP strips was more effective than increasing the width of CFRP strips and the mode of failure was ductile flexural failure instead of be sudden shear failure.
7. The finite element model used in the present study is able to simulate the strengthened reinforced concrete beams with CFRP sheets. The cracking loads, crack patterns and ultimate loads predicted are very close to that measured during the experimental testing.
8. The finite element model showed that the CFRP strips did not reach to their ultimate tensile strength before overall failure of the beams.

6. References

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