Al-Nahrain University, College of Engineering Journal (NUCEJ) Vol.19 No.1, 2016 pp.10 - 26

# Experimental and Numerical Investigation on Shear Transfer of Concrete Specimens Strengthened with CFRP Sheets under Tensile Forces

**Abdullkader I. Al-Hadithi**
Dams & Water Recourses Engineering Department
College of Engineering
University of Anbar.
al_hadithi2000@yahoo.com

**Akram S. Mahmood**
Civil Engineering Department,
College of Engineering,
University of Anbar

**Yasmin S. Abdul-Hameed**
Civil Engineering Department,
of Engineering, College
University of Anbar

## Abstract

This study presents experimental and numerical investigations that concern the behavior of pull-off for normal and high strength concrete specimens strengthened with various configurations of carbon fiber reinforced polymer (CFRP) sheets. Three parameters have been investigated; the first is the strength of concrete by casting half of samples as normal strength concrete and the other as high strength concrete. The second parameter is the ratio of reinforcement crossing the shear plane. The third parameter is the CFRP strengthened. Three dimensional finite elements tools with eight-node elements are used to represent concrete, whereas embedded bar element type is used to represent the reinforcement. Nonlinear behavior of concrete in compression, tension and the reduction of the shear modulus due to cracking are be taken into account.

A finite element which was used to represent concrete is three dimensional with eight nodes elements and embedded bar elements to represent the reinforcement. Nonlinear behavior of concrete in compression, tension and the reduction of the shear modulus due to cracking are be taken into account.

**Keywords:** Shear Transfer, Carbon Fiber Reinforced Polymer, High Strength Concrete, Finite Element, Shear Strength, Shear Plane.

## 1. Introduction

A shear strength which is transmitted across a specific shear plane is denoted as shear transfer. Examples of such situations are precast concrete connections, brackets, corbels, members with shear span less than the effective depth where pure or direct shear is more likely to occur, column footing connections subjected to high shear forces and concrete cast at different ages.

**Mattock and Hawkins (1972)** studied the influence of direct stresses acting parallel and transverse to the shear plane on pull-off tests for normal concrete of initially uncracked and cracked specimens. The results showed that direct tension stresses parallel to the shear plane reduce the transfer strength of initially uncracked concrete ,but do not reduce the shear transfer strength of concrete initially cracked in the shear plane.

**Delorenzis and Nanni, (2001); Hassan And Rizkalla, (2002)** showed that significant increase in stiffness and strength can be achieved using FRP strengthening techniques.

**Karunasena et.al.,(2002), Mohammed A. and Faiud Y.(2010)** showed that the effects of CFRP in improving the moment capacity of deteriorated concrete beams, especially for the structural behavior of such elements under cyclic loading. Wrapped CFRP trend them the fatigue life of strengthened specimens improved the properties of strength and deformation, and considerably increased the ductility of RC beams.

**Mahmoud K.(2004), Al-Mahaidi and Taplin (2004) and Mostofinejad D., and Talaeitaba S.(2006), and Ridha A.(2008)** made comparisons between the finite element modeling of RC strengthened with CFRP and the experimental work. The results showed that good accuracies between the experimental and the finite element resulted. The results showed that good ductility and strength enhancement could be achieved by employing correctly configured FRP.

**Zhang Z. et al(2004), Abdul-Razaq A.(2010), and Lee H. et. al.(2011)** studied a series of experimental tests to investigate the shear behavior of RC strengthened with CFRP sheets in shear. From the results, the shear strengthening performance of CFRP sheets increases as the strengthening length increases with respect to ultimate load.

**Duthinh and Starnes(2001), ALI D.(2007) and Abdesselam Z.(2012**) showed that FRP is very effective for flexural strengthening of concrete beams reinforced with carbon FRP and steel.

**Kachlakev D. (2010)** concluded that the shear reinforcement increases the load carrying capacity by 45% for the experimental beam and by 15%

for the finite element model. This finite element model can be used in additional studies to develop design rules for strengthening reinforced concrete bridge members using FRP.

**Al-Hardan S.(2012)** reported field tests on the use of Carbon Fiber Reinforced Polymers (CFRP) retrofitting to restore the load capacity of the one–way slabs after subjected to different degrees of exposure fire.

Fiber Reinforced Polymer (FRP) sheets have been found to be successful for flexural and shear strengthening and for ductility enhancement of concrete structures. CFRP materials are distinguished by their extremely high strength and rigidity. Low density, excellent damping properties and high resistance to impact are combined with exactly modifiable thermal expansion to complement the complex characteristics profile.

This study presents experimental and numerical investigations which concern the behavior of pull-off for normal and high strength concrete specimens strengthened with various configurations of carbon fiber reinforced polymer (CFRP) sheets, as shown in fig.(1), where SikaWrap type C45 used for strengthening the concrete blocks (**SikaWrap-230(2009)**). Sikadur 330 resin was used for adhesion the CFRP sheets, all properties can be shown from **Sikadur-330 (2008)**. Many samples casted and tested according to the study program. The parameters investigated are, the strength of concrete, area of reinforcement crossing the shear plane, and the directions of the fibers CFRP sheets.

Three dimensional with eight nodes elements are to be used to repesent concrete, and embedded bar elements to represent the reinforcement. The nonlinear behavior of concrete and the reduction of the shear modulus due to cracking is to be taken into account. The finite element results are compared with the experimental results to illustrate the adequacy of this modeling.

**2. Experimental Program**

The experimental work consists of casting concrete shear test block samples reinforced with steel bars, as shown in fig.(2). It is divided into two groups, the first group consists of nine samples of normal strength concrete and the second one consists of nine samples of high strength concrete.

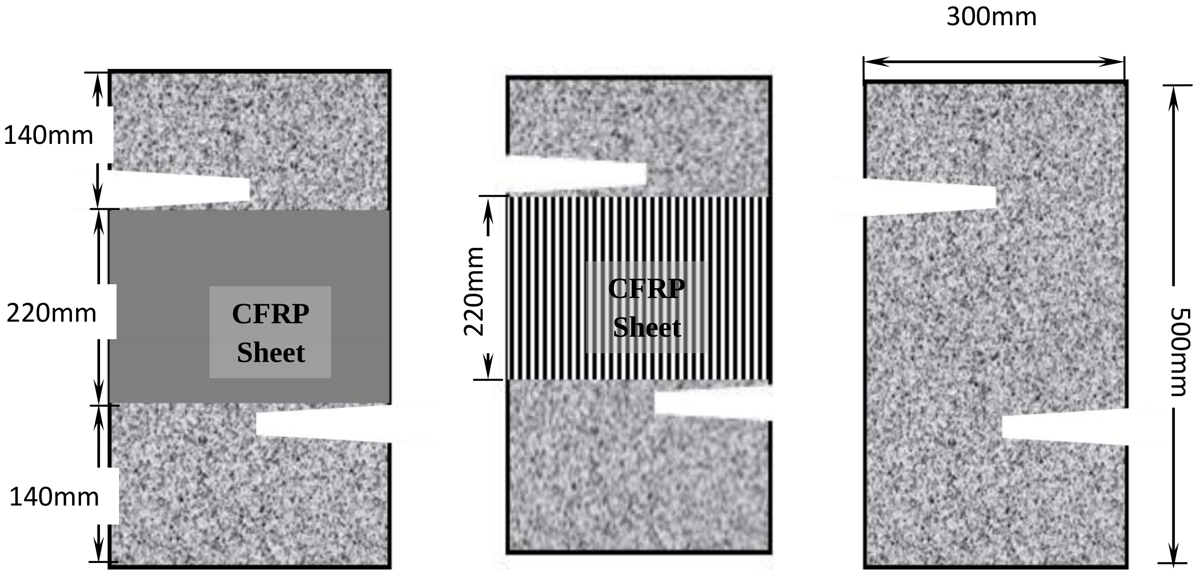
The group of normal concrete samples is divided into three subgroups. The first one is without steel reinforcement (plain concrete) and consists three samples. The first sample is (NWF0) strengthened with full CFRP at zero degree angles in the direction of the shear plane (shear crack). The second sample is strengthened with full CFRP at the right angle in the direction perpendicular to the shear plane (shear crack) direction. It is assigned as (NWF90). And the third sample is a reference one.

Each sample of the second subgroup is reinforced by two 6mm diameter steel reinforcing bars. And also consist of three samples, the first one is reinforced by two 6mm-diameter steel reinforcing bars and strengthened with full CFRP in the direction of the shear plane (shear crack) at zero-degree angles. It is assigned as (N6F0). The second sample designed as (N6F90) is reinforced by two 6mm-diameter reinforcing bars and strengthened with full CFRP in the direction perpendicular to the shear plane (shear crack) direction. The third sample is a reference.

Each sample of the third subgroup is reinforced by two 10mm-diameter reinforcing bars. It consists of three samples, the first sample is reinforced by two 10mm-diameter steel reinforcing bars and strengthened with full CFRP in the direction of the shear plane (shear crack) at zero-degree angles. It is assigned as (N0F0).

The second sample is reinforced by two 10mm-diameter steel reinforcing bars and strengthened with full CFRP in the direction perpendicular to the shear plan (shear crack) direction. It is assigned as (N0F90).The third sample is a reference.

The same manner and designation for the sample of high strength concrete as shown in the paramedical shape of the program. Finally, the test of the samples, has been executed in practical stage, as illustrated in photos in fig.(3).




(a)90 Degree Sample (b)Zero- Degree Sample (c)Reference Sample

**Figure 1:** CFRP configuration.

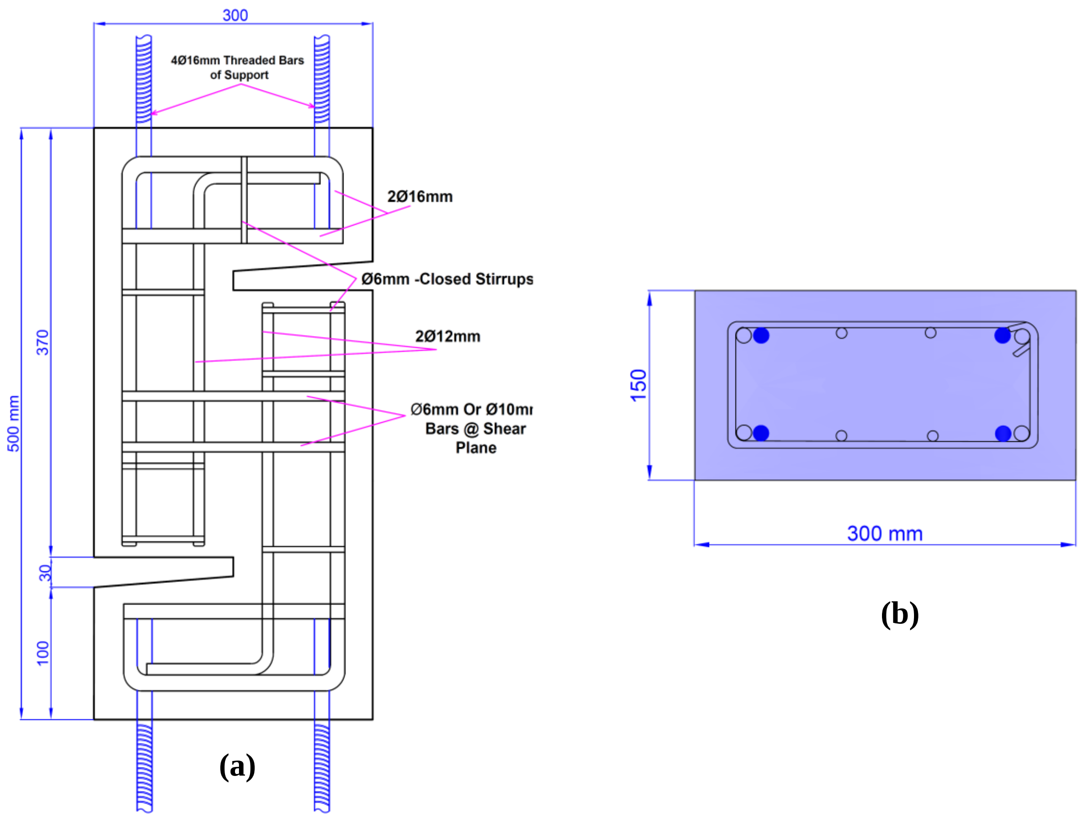



**Figure 2:** Test specimen (a): Dimensions and Steel Reinforcement; (b): Cross Section

**Figure 3:** Measurement of Deformations

## 3. Finite Element Modeling:

During the last three decades of the previous century, interest in non- linear analysis of concrete structures has increased steadily. This is because of the wide use of concrete as a structural materials, and because of the development of relevant powerful analysis techniques implemented on electronic digital computers. The most powerful technique that is already in wide use of the non-linear analysis of reinforced concrete structures is the finite element method **Wolanski B.S. and Anthony J.(2004)**.

Finite element method (FEM) was developed to predict a numerical model for the analysis of shear strength in normal and high strength concrete specimens strengthened with CFRP sheets, using the ANSYS package.

The finite element procedure implemented in this study is developed using the available element types from ANSYS element library. The concrete is modeled using SOLID65 element type. SOLID65 is selected because this concrete material model can predict the failure of brittle materials by adopting the constitutive model of concrete. Both cracking and crushing failure modes can be accounted for, whereas the steel for longitudinal and transverse reinforcements is modeled by LINK8 element type. LINK8 in ANSYS is selected because this steel material model can take into account the complete stress-strain relations of materials. Both yielding and strain-hardening failure modes can be accounted for. The steel plate is modeled with SOLID45 element type. And the fiber strengthening is modeled with SOLID46. The concepts used are directly applicable to 3D SOLID elements. By adopting and combining these four element types, the reinforced concrete model was developed. The model was subjected to an axial tensile loading on their top surface, while the bottom surface was restrained. The loading procedure can be elaborated by subjecting the model to a step-by-step incremental axial tension on its entire top surface.

The analytical models were constructed according to the actual specimens. The models had a typical cross section of 500 mm × 300 mm with the height of 150 mm. The concrete cover was 15 mm. The constitutive laws used in the proposed analytical model were developed for three materials of the specimens, namely concrete, steel and CFRP. Since the proposed procedure is intended as an alternative way for predicting the actual nonlinear behavior of both unconfined and confined reinforced concrete specimens prior to conducting the experimental program, the analytical model proposed by (weliam-wrank) to represent the stress-strain relationship of concrete was adopted in this study.

## 4. Results and Discussion

### 4.1 Comparison between the Finite Elements and the Experimental Results for the Effect of Compression Strength:

As in case of experimental work, samples similar to those in the experimental work with the same boundary conditions and the same constraints had been modeled. It is solved by ANSYS program and results were obtained and compared with the experimental ones. Table(1) shows the experimental and the finite elements results and the differences between them. The difference percentages between the experimental and the finite-element results range between 2.7% to 12.5%.

**Table 1:** Ultimate Shear Strength values obtained by the finite element method results and by the experimental work.

| Samples | Shear Strength (kN) (Experimental) | Shear Strength (kN) (Finite elements) | Difference (%) |
|---|---|---|---|
| NWR | 38 | 36.85 | 3% |
| NWF90 | 47.5 | 41.58 | 12.5% |
| NWF0 | 57 | 51.97 | 9% |
| N6R | 53.2 | 48.51 | 8.8% |
| N6F90 | 60.8 | 55.44 | 8.81% |
| N6F0 | 64.6 | 58.9 | 8.82% |
| N10R | 57 | 51.97 | 8.82% |
| N10F90 | 68.4 | 62.37 | 8.8% |
| N10F0 | 76 | 68.72 | 9.6% |
| HWR | 57 | 51.97 | 8.8% |
| HWF90 | 68.4 | 60.64 | 11% |
| HWF0 | 76 | 68.7 | 9.6% |
| H6R | 66.5 | 64.7 | 3% |
| H6F90 | 72.2 | 68.7 | 4.85% |
| H6F0 | 91.2 | 83.2 | 8.7% |
| H10R | 76 | 68.7 | 9.6% |
| H10F90 | 85.5 | 83.2 | 2.7% |
| H10F0 | 98.8 | 93.55 | 5.3% |

From the results above it is noticed that all the experimental results are larger than the finite element results because in case the finite element analysis the program stopped the solver when the concrete failed without taking into account the effect of the reinforcing bars which give more strength to the sample as the experimental work. This face also explains the stiffness reduction for the finite element models of the tested specimens.

The figures (4-12) show the difference, that the shear strength predicted by the finite element model is lesser than that given by the experimental work. Values of the results of the finite element model did not make an effect on the whole curves of the tests but only the first point of curvature which means that the results are a part of the whole outcome.

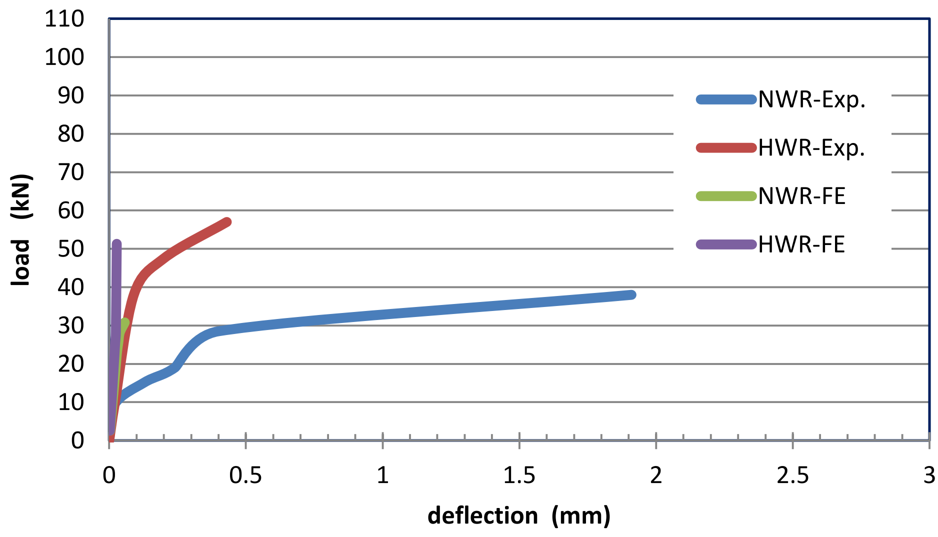



**Figure 4:** The load-deflection relations of sample (NWR) and (HWR) as given by the experimental work and by the finite element model.

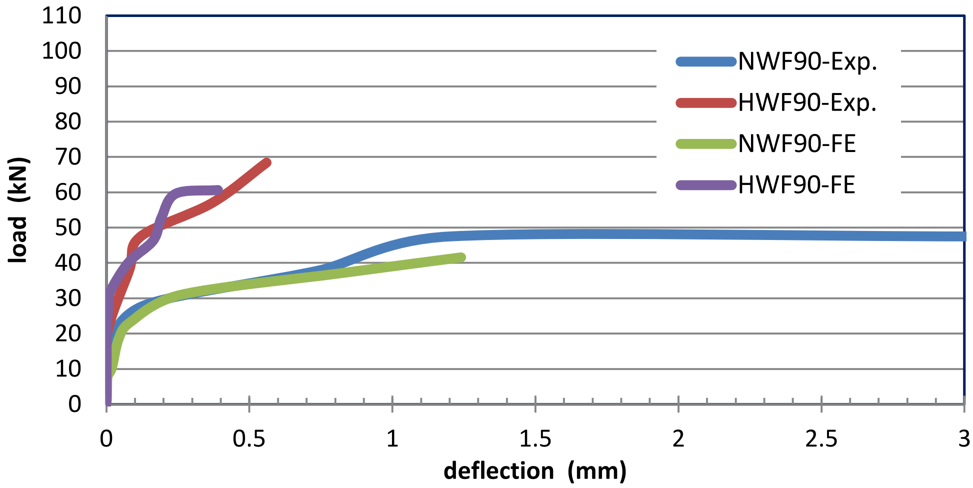



**Figure 5:** The load-deflection relations of sample (NWF90) and (HWF90) as given by the experimental work and by the finite element model.

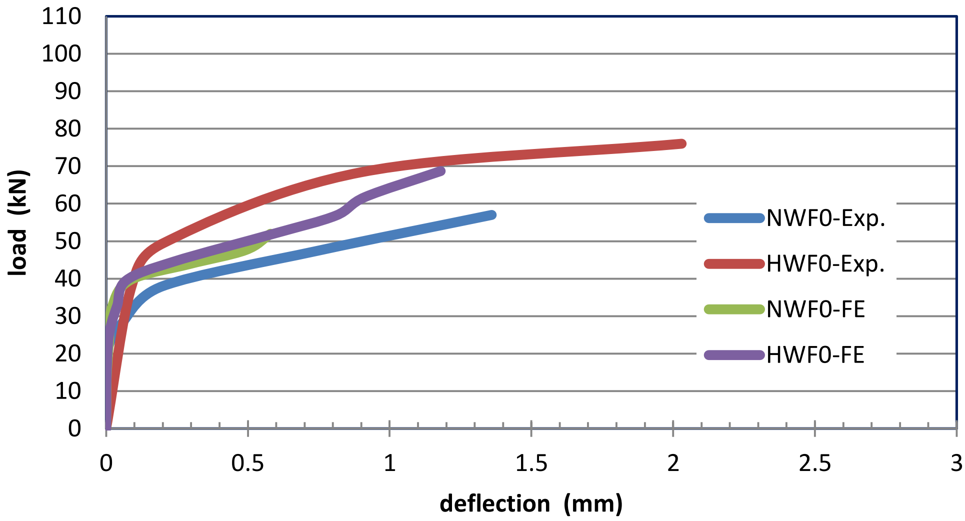



**Figure 6:** The load-deflection relations of sample (NWF0) and (HWF0) as given by the experimental work and by the finite element model.

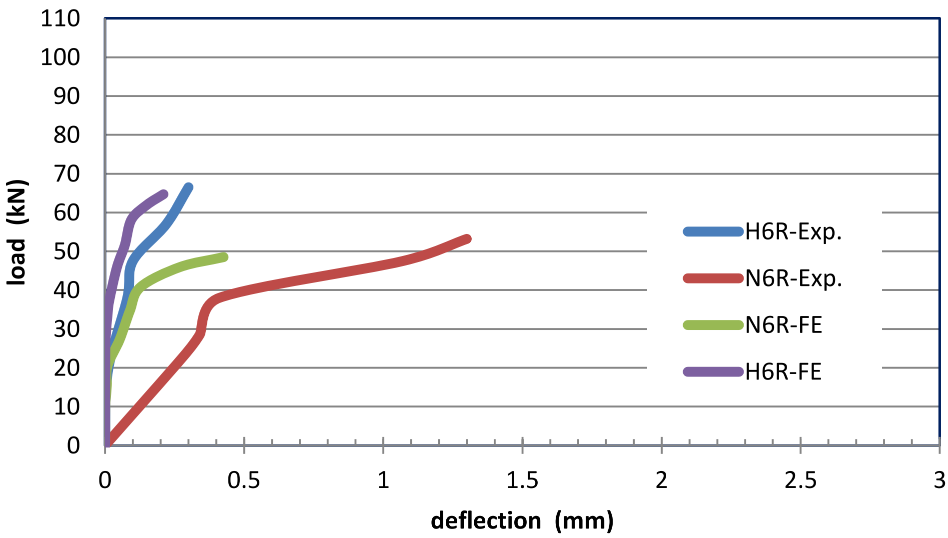



**Figure 7:** The load-deflection relations of sample (N6R) and (H6R) as given by the experimental work and by the finite element model.

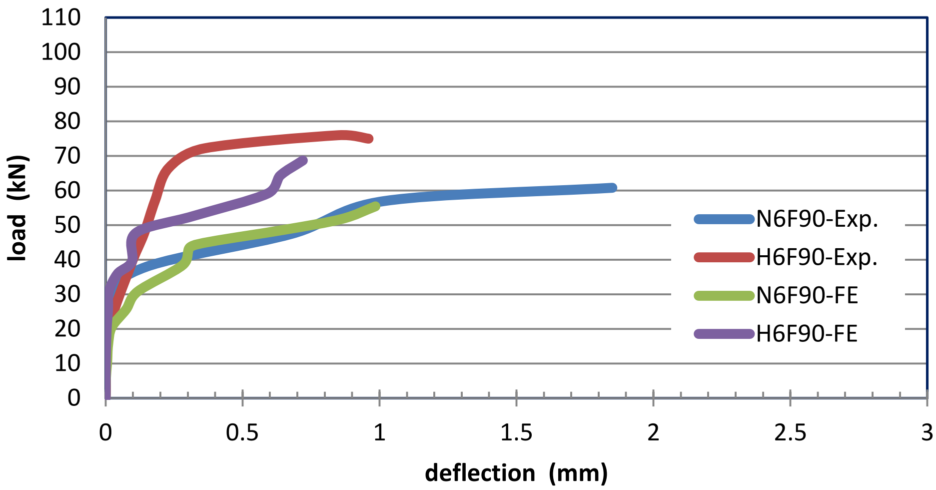



**Figure 8:** The load-deflection relations of sample (N6F90) and (H6F90) as given by the experimental work and by the finite element model.

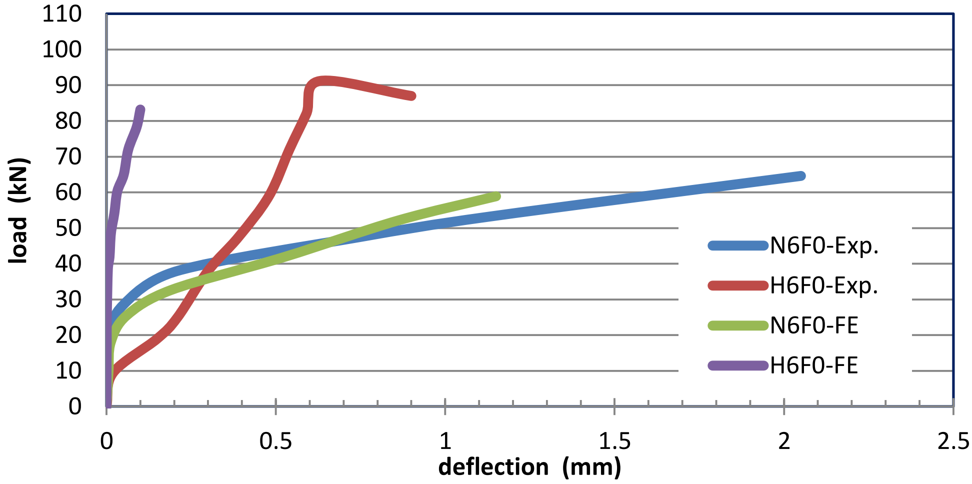



**Figure 9:** The load-deflection relations of sample (N6F0) and (H6F0) as given by the experimental work and by the finite element model.

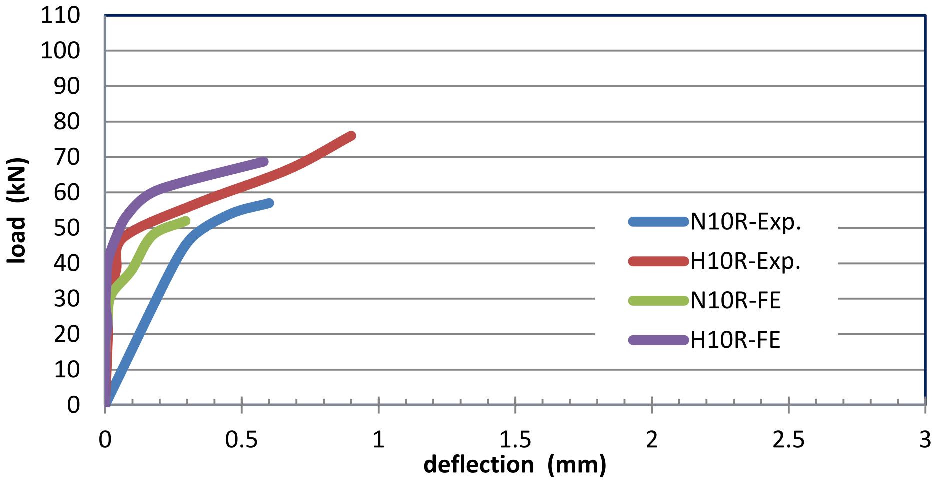



**Figure 10:** The load-deflection relations of sample (N10R) and (H10R) as given by the experimental work and by the finite element model.

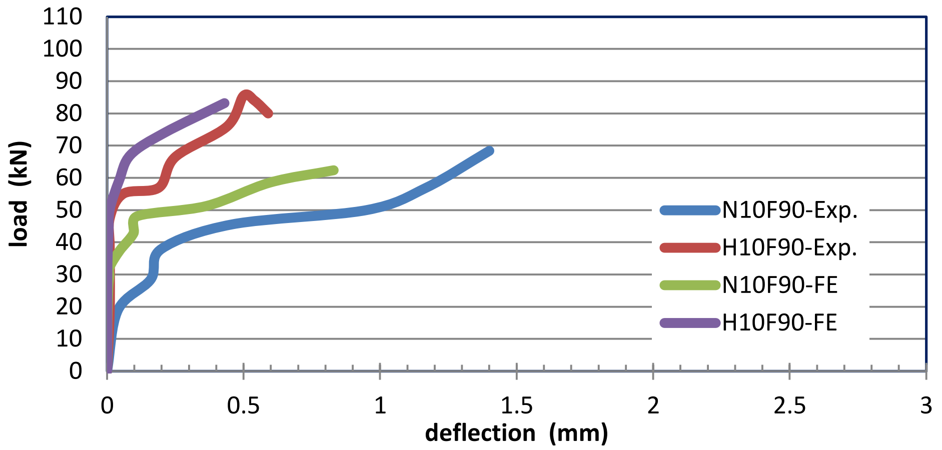



**Figure 11:** The load-deflection relations of sample (N10F90) and (H10F90) as given by the experimental work and by the finite element model.

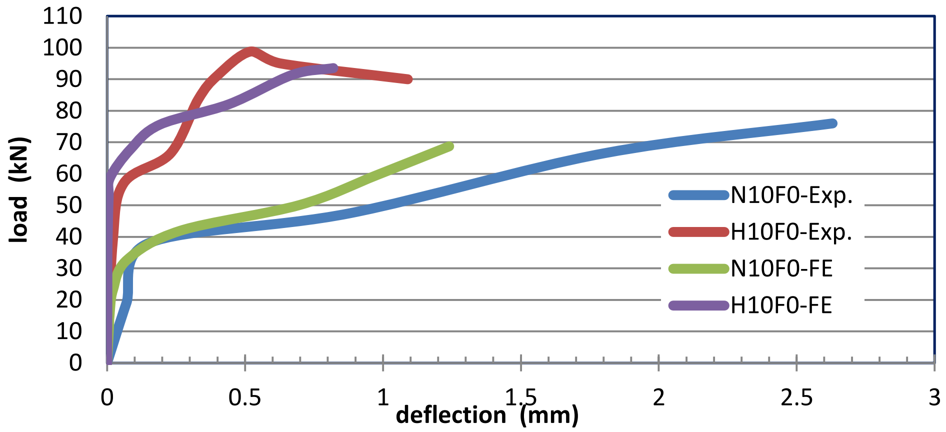



**Figure 12:** The load-deflection relations of sample (N10F0) and (H10F0) as given by the experimental work and by the finite element model.

### 4.2 Comparison between the Finite Element and the Experimental Results for the Effect of Strengthening

A comparison between the results obtained from the finite element model and those from the experimental work for the effect of strengthening the samples with CFRP sheets has also been made. Table (2) shows the difference

Between the percentages of increases in the ultimate shear strength due to strengthen by

CFRP sheets obtained by the experimental and the finite element results. Those differences do not exceed 9% anywhere.

**Table 2:** Percentages of increases in the ultimate shear strength due to strengthen by CFRP sheets obtained experimentally and numerically.

| Samples | Experimental results | Finite elements results | Difference |
|---|---|---|---|
| (NWF90),(NWR) | 20% | 11% | 9% |
| (NWF0),(NWR) | 33% | 29% | 4% |
| (N6F90),(N6R) | 12.5% | 12.5% | —— |
| (N6F0),(N6R) | 17.6% | 17% | 0.6% |
| (N10F90),(N10R) | 16.7% | 16% | 0.7% |
| (N10F0),(N10R) | 25% | 24.4% | 0.6% |
| (HWF90),(HWR) | 17% | 14% | 3% |
| (HWF0),(HWR) | 25% | 24% | 1% |
| (H6F90),(H6R) | 8% | 6% | 2% |
| (H6F0),(H6R) | 27% | 22% | 5% |
| (H10F90),(H10R) | 11% | 17% | -6% |
| (H10F0),(H10R) | 23% | 27% | -4% |

From the results in Table(2), that the difference between samples is sometimes greater in experimental and sometimes equal (which is only in case of H10F90), For (H10R) the finite element prediction is larger than that obtained the experimental work, because of the experimental tested specimens have many changes in preparing and carrying to test machine, that may created many small cracks or deformation are not taken in finite element simulation. The figures (13-18) show the difference between finite elements and experimental results.

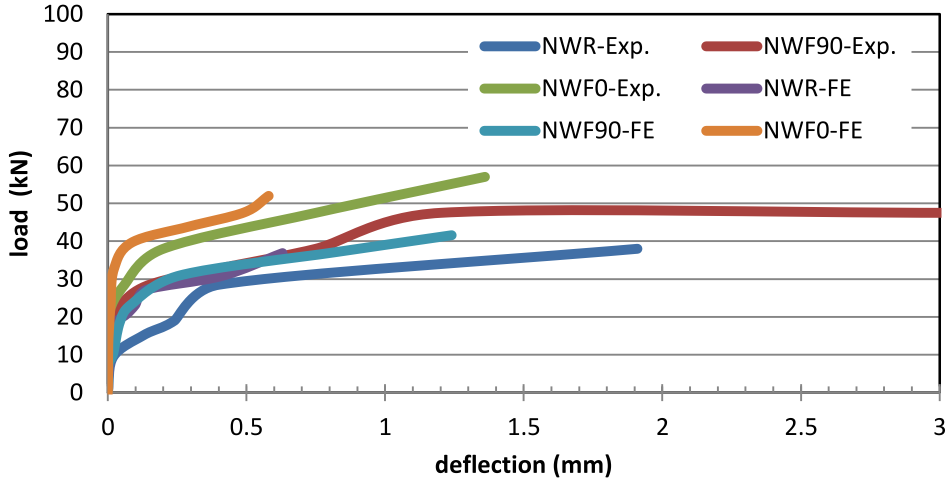



**Figure 13:** Comparative load-deflection relations for (NWR-Exp.), (NWR-FE), (NWF90-Exp.), (NWF90-FE), (NWF0-Exp.), (NWF0-FE) samples.

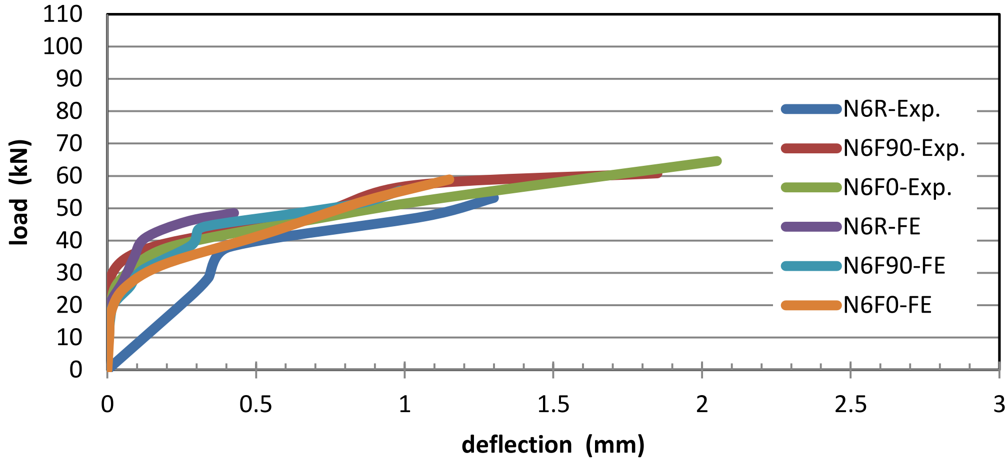



**Figure 14:** Comparative load-deflection relations for (N6R-Exp.), (N6R-FE), (N6F90-Exp.), (N6F90-FE), (N6F0-Exp.), (N6F0-FE) samples.

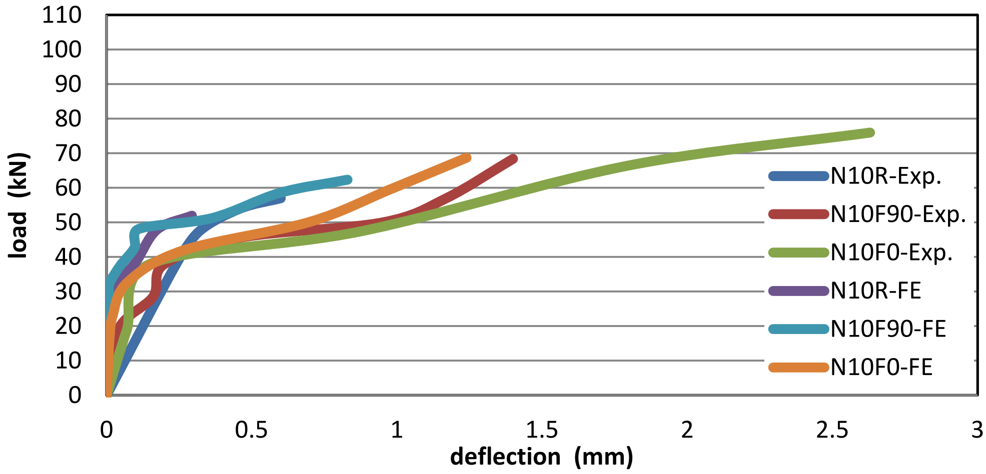



Fig (15) Comparative load-deflection relations for (N10R-Exp.), (N10R-FE), (N10F90-Exp.), (N10F90-FE), (N10F0-Exp.), (N10F0-FE) samples

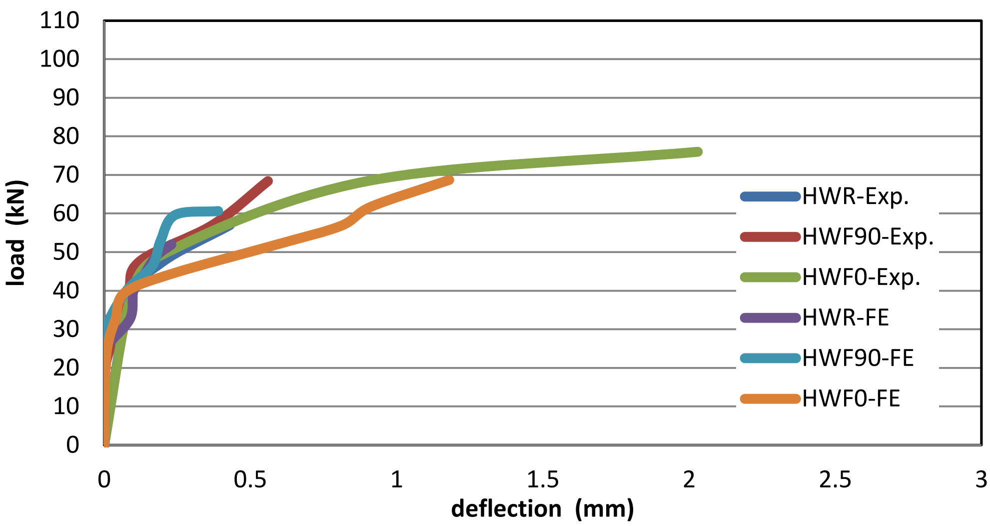



**Figure 16:** Comparative load-deflection relations for (HWR-Exp.), (HWR-FE), (HWF90-Exp.), (HWF90-FE), (HWF0-Exp.), (HWF0-FE) samples.

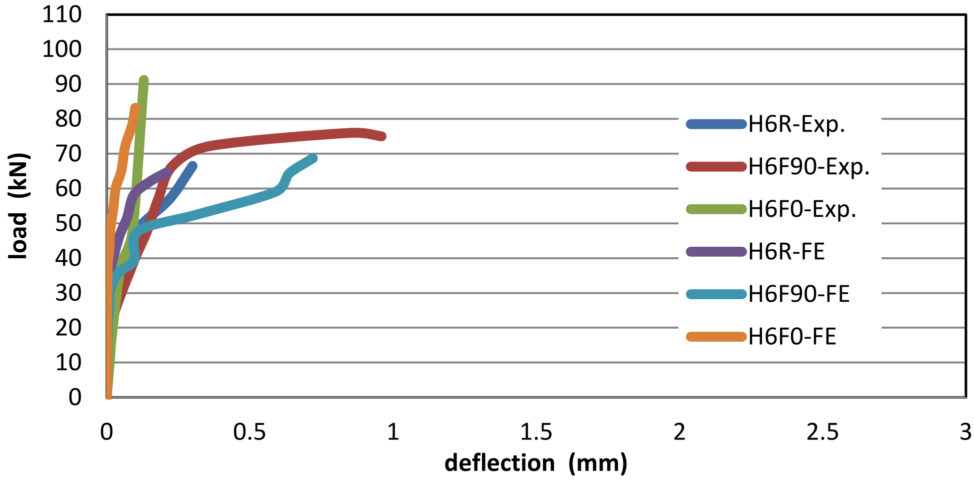



**Figure 17:** Comparative load-deflection relations for (H6R-Exp.), (H6R-FE), (H6F90-Exp.), (H6F90-FE), (H6F0-Exp.), (H6F0-FE) samples.

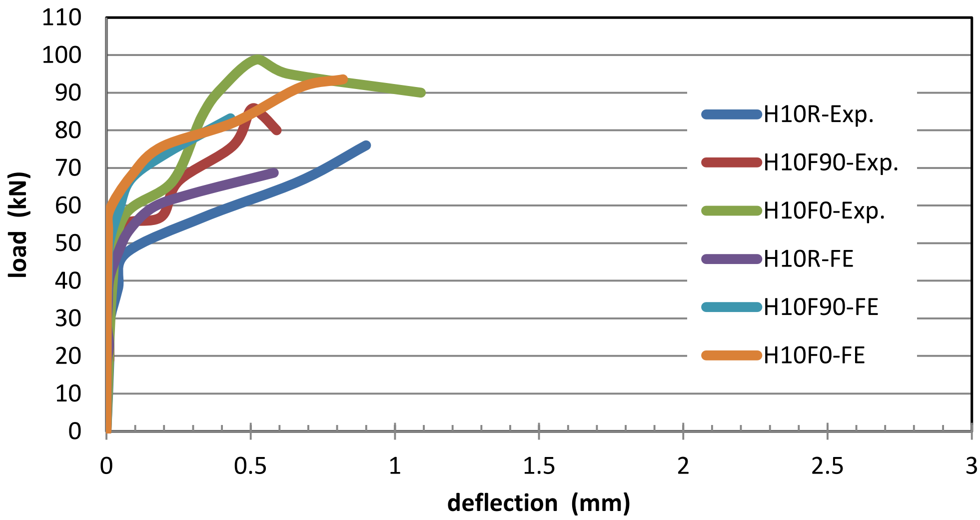



**Figure 18:** Comparative load-deflection relations for (H10R-Exp.), (H10R-FE), (H10F90-Exp.),(H10F90-FE), (H10F0-Exp.), (H10F0-FE) samples.

### 4.3 Comparison Between the Finite Element and the Experimental Result fo the Effect of Reinforcement in the Shear Plane

The finite element model also solved the reinforced samples in the three types of reinforcement and compared the results with the experimental results. Table(3) shows the results and the differences between the experimental and the finite elements results. same different percentage between the experimental and the finite elements results range between 0% to 9%.

**Table 3:** Percentages of increases in the ultimate shear strengths due to the effect of introducing reinforcement in the shear planes obtained by experimentally and numerically.

| Samples | Experimental results | Finite elements results | Difference |
|---|---|---|---|
| (N6R),(NWR) | 28% | 24% | 4% |
| (N10R),(NWR) | 33% | 29% | 4% |
| (N10R),(N6R) | 7% | 6.7% | 0.3% |
| (N6F90),(NWF90) | 22% | 25% | -3% |
| (N6F0),(NWF0) | 12% | 11.7% | 0.3% |
| (N10F90),(NWF90) | 30% | 33% | -3% |
| (N10F0),(NWF0) | 25% | 24% | 1% |
| (N10F90),(N6F90) | 11% | 11% | ….. |
| (N10F0),(N6F0) | 15% | 14% | 1% |
| (H6R),(HWR) | 14% | 19.7% | -5.7% |
| (H10R),(HWR) | 25% | 24% | 1% |
| (H10R),(H6R) | 12.5% | 5.8% | 6.7% |
| (H6F90),(HWF90) | 5.52% | 11.7% | -6.18% |
| (H6F0),(HWF0) | 17% | 17% | …. |
| (H10F90),(HWF90) | 20% | 27% | -7% |
| (H10F0),(HWF0) | 23% | 26.5% | -3.5% |
| (H10F90),(H6F90) | 15.5% | 17% | -1.5% |
| (H10F0),(H6F0) | 7.7% | 11% | -3.3% |

Table(3) indicates the agreement of the finite element model and the experimental results. The same difference produced by experimental problems such as problem when putting the sample in the mold or when putting it in the test machine arises, where the model exposed to shock attending a reduction in the strength of samples initial cracking before applying load. The figures (19-24) show the comparison between samples in the finite elements and experimental results in the respect of existence or absence of transverse reinforcement.

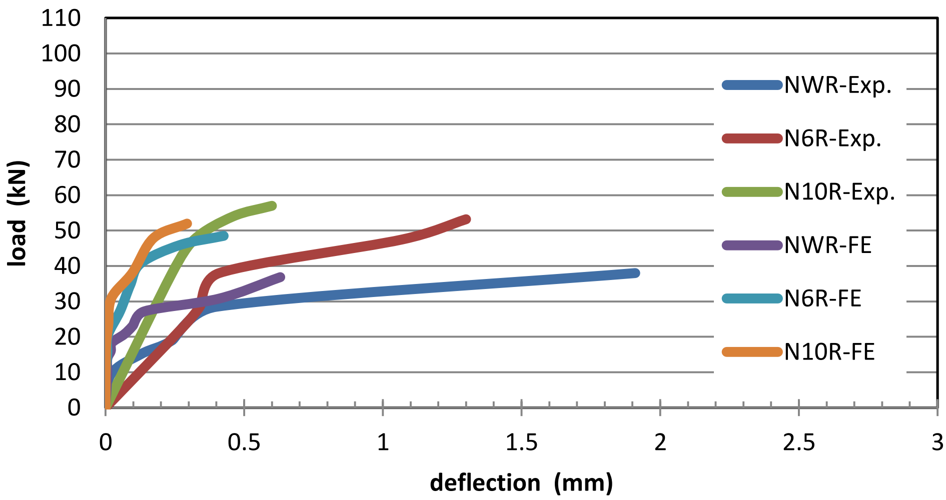



**Figure 19:** Comparative load-deflection relations for (NWR-Exp.), (NWR-FE), (N6R-Exp.), (N6R-FE), (N10R-Exp.), (N10R-FE) samples

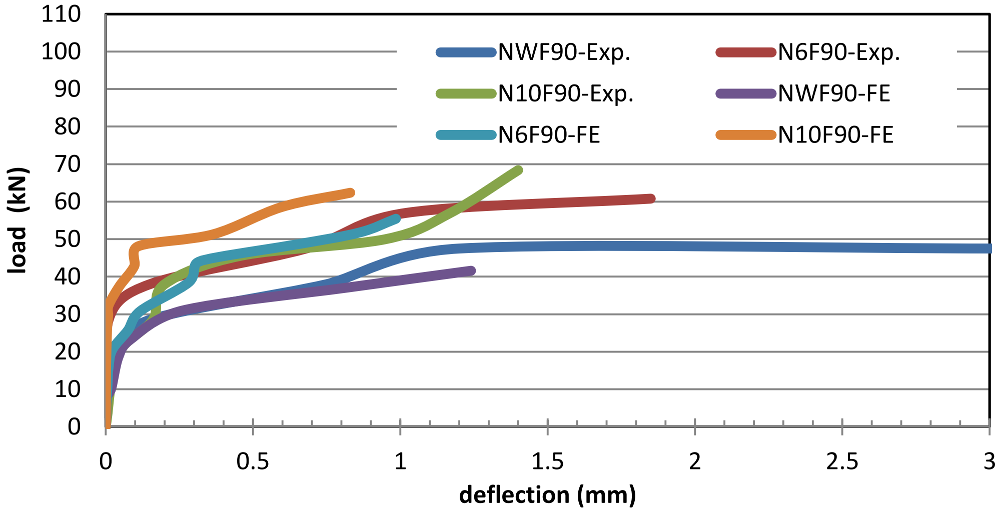



**Figure 20:** Comparative load-deflection relations for (NWF90-Exp.), (NWF90-FE), (N6F90-Exp.), (N6F90-FE), (N10F90-Exp.), (N10F90-FE) samples.

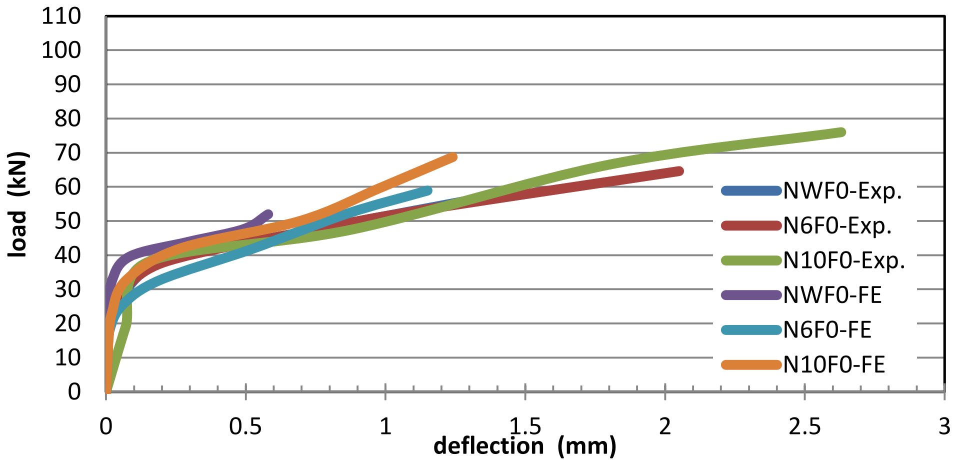



**Figure 21:** Comparative load-deflection relations for (NWF0-Exp.), (NWF0-FE), (N6F0-Exp.), (N6F0-FE), (N10F0-Exp.), (N10F0-FE) samples.

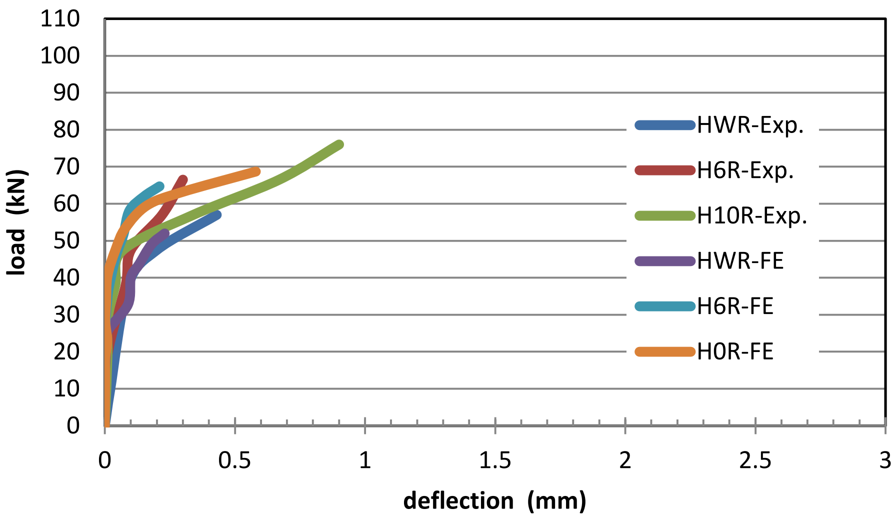



**Figure 22:** Comparative load-deflection relations for (HWR-Exp.), (HWR-FE), (H6R-Exp.), (H6R-FE), (H10R-Exp.), (H10R-FE) samples.

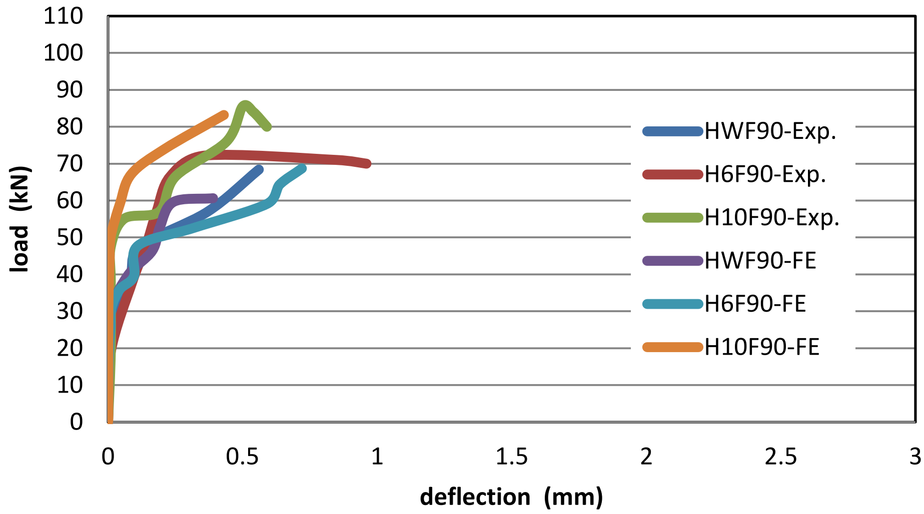



**Figure 23**: Comparative load-deflection relations for (HWF90-Exp.), (HWF90-FE), (H6F90-Exp.), (H6F90-FE), (H10F90-Exp.), (H10F90-FE) samples.

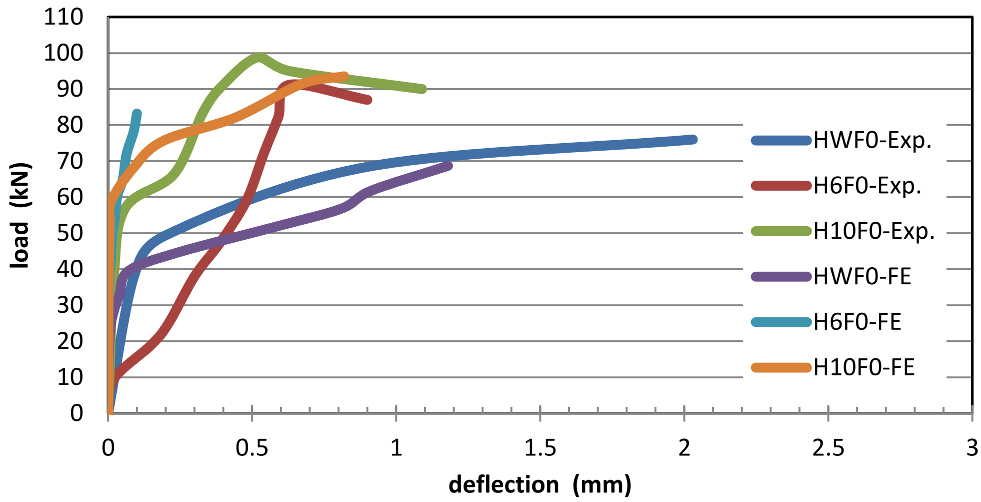



**Figure 24:** Comparative load-deflection relations for (HWF0-Exp.), (HWF0-FE), (H6F0-Exp.), (H6F0-FE), (H10F0-Exp.), (H10F0-FE) samples.

## 5. Failure modes:

For unstrengthened specimens, when the applied load was small, no cracks or split could be seen on the specimen surfaces. When the load approached the ultimate load, some vertical shear cracks appeared on the specimen surfaces. For strengthened specimens, the crack width is less than strengthened specimens. Also in high concrete strength and strengthened specimens

The cracks were banded in diagonal cracks along the shear plane. Cracks width increased with increase the shear displacement on both sides of the block specimens. The specimens failed with much wider cracks width and shear displacement. The testing records showed that the stirrups had yielded. There exists an only slight difference between all the failure modes. The representative crack patterns at failure are shown in figure (25, a–f).

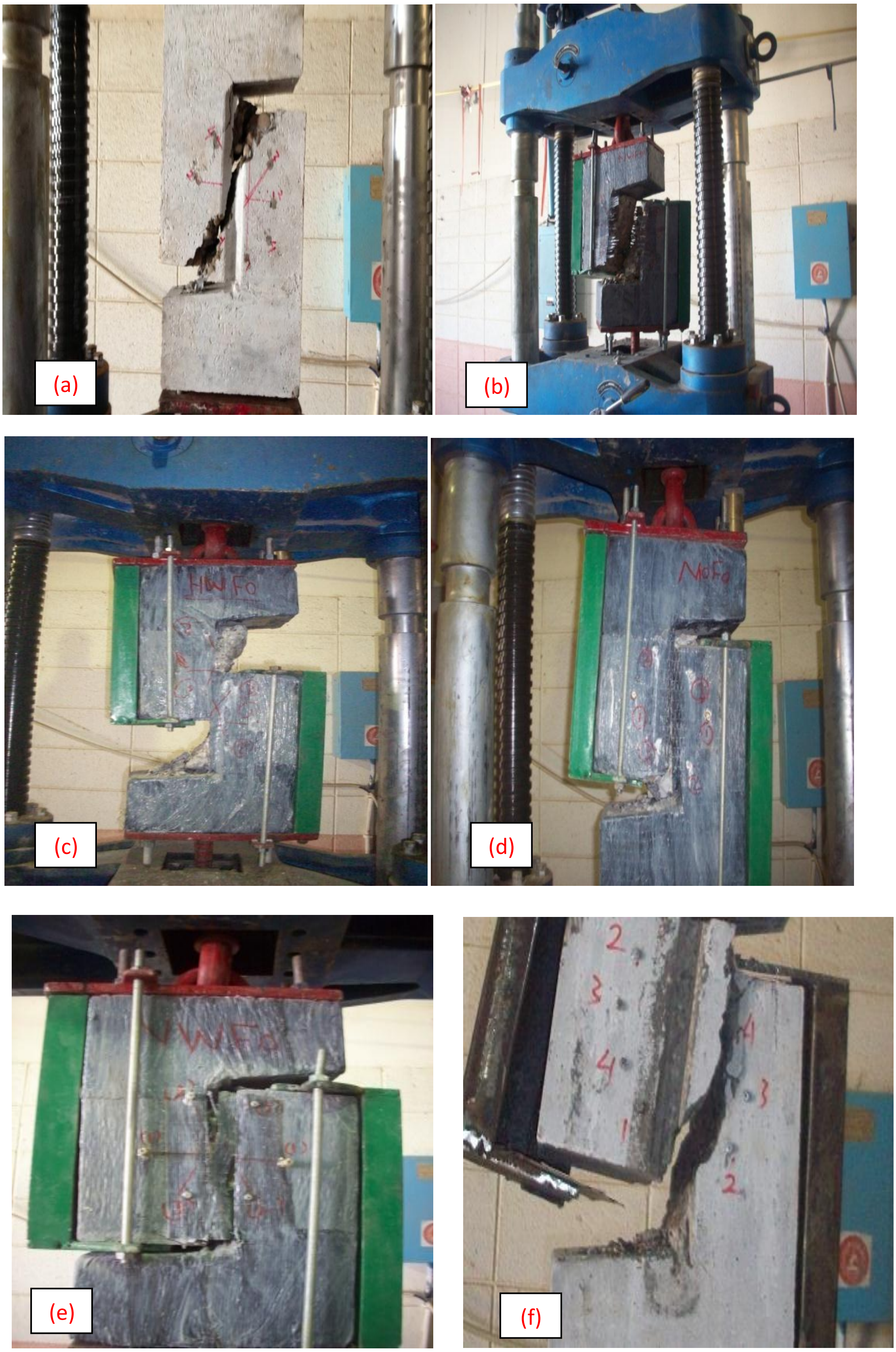



**Figure25:** Modes of failure.

## 6. Conclusions

The conclusions emerged from the experimental work and finite element modeling's are summarized below:

1. Strengthening schemes with CFRP sheets leads to increase the ultimate strength. In the experimental work this increase reached 20% in case of normal strength concrete strengthened in the direction perpendicular to the shear plane, 17% in case of high strength concrete strengthened in the direction perpendicular to the shear plane, 33% in case of normal strength concrete strengthened in the direction parallel to the shear plane and 27% in case of high strength concrete strengthened in direction parallel to the shear plane. In case of the finite element model strengthening schemes with CFRP sheets lead to increase ultimate strength. This increase reached 17% in case of normal concrete strengthened in the direction perpendicular to the shear plane, 17% in case of high strength concrete strengthened in direction perpendicular to the shear plane, 29% in case of normal concrete strengthened in direction parallel to the shear plane and 27% in case of high strength concrete strengthened in direction parallel to the shear plane. These results show that the load strength for the strengthened samples is greater than for unstrengthened samples in both case of normal and high strength concrete because of the effective bond of fiber to the concrete which delayed the failure. The strength in case of strengthening in the direction parallel to the shear plane was larger than in the case of strengthening in the direction perpendicular to the shear plane because the fibers are weak in the transverse direction.
2. The reinforcement in the shear plane increases the ultimate strength when compared with samples without reinforcement. In case of experimental work the increase reached 28% in case of normal concrete reinforced in shear plane by two 6mm diameter bars, 33% in case of normal strength concrete reinforced in shear plane by two 10mm diameter bars, 17% in case of high strength concrete reinforced in shear plane by two 6mm diameter bars and 25% in case of high strength concrete reinforced in shear plane by two bars diameter 10mm diameter bars. In the case of finite element the reinforcement in the shear plane increases the ultimate strength when compared with blocks without reinforcement. Those increases reached 25% in case of normal concrete reinforced in shear plane by two 6mm diameter bars, 33% in case of normal concrete reinforced in shear plane by two 10mm diameter bars, 20% in case of high strength concrete reinforced in shear plane by two 6mm diameter bars and 27% in case of high strength concrete reinforced in shear plane by two 10mm diameter bars. These results show that the samples are reinforced by steel bar of 10mm diameter which has more load and less deformation than the samples reinforced by a steel bar of 6mm diameter. The sample without reinforcement, has of course, the least load bearing capacity because the strength increased with increase of the diameter of steel bars.
3. Other main parameter is the strength of concrete. The ultimate strength increased and the deformation decreased in case of high strength concrete which reached 33% from the normal concrete in the experimental work and reached 31% in the finite elements results, the difference between the experimental and finite elements results range from 2.7% to 12.5%. From the result it can be noticed that all the experimental results are higher than the finite element results because in case of the finite element method the program stopped the solution when the concrete failed without taking into account the effect or the reinforcement bars which give more strength to the sample as in the experimental work, these reasons also explained the stiffness deterioration for samples by finite elements.
4. The adopted concrete (three-dimensional finite element) model used in the present work proved to be capable of providing good estimates of strength and deformations for concrete elements subjected to pure shear.
5. Comparisons of the FEM results with the experimental data showed that the maximum difference in shear transfer for most of the tested samples was less than 10 %.
6. To produce more shear transfer strength the concrete samples must be strengthened in the direction parallel to the shear plane.
7. The load-deflection curve was soft in normal strength concrete and stiffer in high strength concrete, and the crack was larger in normal concrete because high strength concrete failed rigidly but the normal concrete took more time to fail that allowed crack expansion.

## References


[1] **Abdesselam, Z. (2012), "**Numerical modeling of strengthened concrete beams" International Journal of Civil and Structural Engineering, Vol. 2, No.4, pp.1122-1128.

[2] **Abdul-Razaq, A.H. (2010),** "*Behavior of Concrete Beams Reinforced in Shear with Carbon Fiber Reinforced Polymer*",

Ph. D. Thesis, University of Baghdad, Iraq

[3] **Al-Hardan, S.A. (2012),** " Retrofitting Of Reinforced Concrete One–Way Damaged Slabs Exposed To High Temperature" M.Sc. Thesis, University of Anbar, Iraq.

[4] **Ali, D.D. (2007),** "Experimental And Theoretical Investigation Of The Behavior Of Reinforced Concrete Beams Strengthened By Fiber Reinforced Polymer", Ph.D. Thesis, University of Baghdad, Iraq.

[5] **Al-Mahaidi, R., Lee, K. and Taplin, G., (2001),** "Behavior and Analysis of RC T-Beams Partially Damaged in Shear and Repaired with CFRP Laminates", Structures congress, ASCE, Washington, USA.

[6] **Delorenzis, L. and Nanni, A. (2001),** " Shear Strengthening Of Reinforced Concrete Beams With Near-Surface Mounted Fibre Reinforced Polymer Rods", ACI Structural Journal, Vol. 98, No.1, pp.60-68

[7] **Duthinh, D. and Starnes, M., (2001),** "Strength and Ductility of Concrete Beams Reinforced with FRP and Steel", National Institute of Standards and Technology, NISTIR 6830, pp.493-498.

[8] **Hassan, T. and Rizkalla, S. (2003),** "Investigation of Bond in Concrete Structures Strengthened with Near Surface Mounted CFRP Strips", ASCE, Journal of Composite for Construction, North Carolina State University, pp.248-257.

[9] **Kachlakev, D.I. (2010),** "Finite Element Modeling of Reinforced Concrete Structures Strengthened with FRP Laminates", Oregon State University, Corvallis, USA.

[10] **Karunasena, W.M., Hardeo, P.P., Bosnich, G.D., (2002),** "Rehabilitation Of Concrete Beams By Externally Bonding Fiber Composite Reinforcement", Proc. of the ACUN-4, pp.222-226.

[11] **Lee, H.K., Cheong, S.H., Haa, S.K., Lee, C.G. (2011),** "Behavior and performance of RC T-section deep beams externally strengthened in shear with CFRP sheets", Composite Structures, Vol. 93, No.2, pp. 911-922.

[12] **Mahmoud, K.S., (2004),** " Nonlinear Analysis of Prestressed Concrete Deep Beams " M.Sc. Thesis, University of Technology, Iraq.

[13] **Mattock, A.H. and Hawkins, N.M. (1972),** " Shear Transfer in Reinforced Concrete", PCI Journal, Vol.17, No.2, pp.55-75.

[14] **Mohammed, A.A., and Faiud, Y.K., (2010),** " Parameters Affecting the Behavior of Reinforced Concrete Wrapped With CFRP Sheet", Al-Rafidain Engineering, Vol.20 No.2, pp.1-26

[15] **Mostofinejad, D. and Talaeitaba, S.B. (2006)**, "Finite Element Modeling of RC Connections Strengthened with Laminates", Iranian Journal of Science & Technology, Transaction B, Engineering, Vol. 30, No. B1, pp.21-30.

[16] **Ridha, A.D. (2008),** "Structural Behavior Of Fiber RC Deep Beams", Ph.D. Thesis, University of Technology, Iraq.

[17] **Sikadur-330 C/45 (2008), "** Sikadur-330: 2-part epoxy impregnation resin", Product &Technical Data Sheet, Sika Co., Istanbul branch, Turkey.

[18] **Wolanski, A.J. (2004),** "Flexural Behavior Of Reinforced And Prestressed Concrete Beams Using Finite Element Analysis", M.Sc. thesis, Marquette University, Milwaukee, Wisconsin.

[19] **Zhang, Z., Hsu, C.T., and Moren, J. (2004),** " Shear Strengthening of Reinforced Concrete Deep Beams Using Carbon Fiber Reinforced Polymer Laminates", ASCE, Journal of Composites for Construction, Vol. 8, No. 5, pp.403-414.

# التحقيقات التجريبية والعددية لعينات الخرسانة المسلحة المقواة بألياف الكاربون تحت قوى الشد

**عبد القادر إسماعيل عبدالوهاب الحديثي**
قسم هندسة السدود والموارد المائية – كلية الهندسة – جامعة الانبار

**أكرم شاكر محمود**
قسم الهندسة المدنية – كلية الهندسة – جامعة الانبار

**ياسمين سعد عبد الحميد**
قسم الهندسة المدنية – كلية الهندسة – جامعة الانبار

## الخلاصة :

يتضمن هذه البحث دراسة عملية و نظرية لسلوك نماذج للخرسانة المسلحة المعتدلة والعالية المقاومة والمقواة بطرق مختلفة بشرائح ألياف الكاربون. تمت دراسة المتغيرات الآتية: المتغير الأول هو مقاومة الخرسانة وذلك بصب نصف النماذج كخرسانة معتدلة المقاومة والنصف الآخر بخرسانة عالية المقاومة ، أما المتغير الثاني هو حديد التسليح في منطقة القص، فيما كان المتغير الثالث هو التقوية بألياف الكاربون.

استخدمت طريقة العناصر المحددة لتمثيل النماذج الخرسانية، حيث استخدم موديل ثلاثي الأبعاد بثماني عقد وتم تمثيل قضبان الحديد المطمور في جسم الخرسانة ، مع الأخذ بنظر الاعتبار التصرف الغير خطي للخرسانة بالانضغاط والشد والتقليل في معامل القص الناتج عن التشقق. نفس المتغيرات التي تم دراستها في البرنامج العملي طبقت في البرنامج النظري.

**الكلمات المفتاحية:** تحويل القص، ألياف البولميرية الكاربونية المسلحة، خرسانة عالية المقاومة، عناصر محددة، مقاومة القص للخرسانة، مستوى القص.