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STRENGTH AND DUCTILITY OF ULTRA HIGH PERFORMANCE FIBER REINFORCED CONCRETE BEAMS WITH EXTERNALLY CFRP SHEETS*

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ABSTRACT:

The addition of carbon fiber reinforced polymer (CFRP) composites, which is another form of tension reinforcement, affects the ductility of UHPFRC beams strengthened with CFRP sheets. However, the flexural ductility of beams with respect to amount and and yield strength of existing ordinary steel bars has not been investigated in depth. In addition, delamination of CFRP sheets dominates the ultimate mode of failure of flexural members strengthened with CFRP sheets, which limits the ductility of strengthened members. There is a need to investigate the effect of CFRP anchorage system on the overall ductility of strengthened UHPFRC concrete beams. In this research, tests were carried out on thirty two ultra high performance fiber reinforced concrete (UHPFRC) beams to study the effects of some parameters on the flexural strength and ductility of UHPFRC beams with externally strengthening with CFRP sheets. The major parameters included in this research were the amount of internal steel reinforcement, the volume fraction of the fiber and the configurations of placement of transverse anchorages along the external longitudinal CFRP. The test results showed that externally bonding CFRP sheets on the bottom flange of UHPFRC beams led to increase load-carrying capacity but reduce flexural ductility. Also the results showed that the CFRP anchorages could significantly increase the flexural ductility, Furthermore the transverse anchorages were sufficient to eliminate debonding, and the UHPFRC beams failed when the longitudinal CFRP materials fractured, in other cases, the transverse anchorages simply delayed debonding, and the longitudinal CFRP materials debonded after the transverse anchorages fractured.

KEY WORDS*:* CFRP, UHPFRC, Flexural Strength, Ductility.

RÉSISTANCE ET LA DUCTILITÉ FIBRE ULTRA HAUTES PERFORMANCES POUTRES EN BÉTON ARMÉ AVEC DES FEUILLES CFRP EXTERNE

L'ajout de polymère renforcé de fibres de carbone (CFRP) composites, qui est une autre forme d'armatures tendues, affecte la ductilité des poutres en BFUP renforcée avec des feuilles de PRFC. Cependant, la ductilité en flexion de poutres en ce qui concerne le montant et la limite d'élasticité et de barres d'acier ordinaires existantes n'a pas été étudiée en profondeur. En outre, la délamination de feuilles de PRFC domine le mode ultime de l'échec des membres de flexion renforcés avec des feuilles de PRFC, ce qui limite la ductilité des membres renforcés. Il est nécessaire d'étudier l'effet de CFRP système d'ancrage sur la ductilité globale de renforcement des poutres en béton BFUP. Dans cette recherche, les tests ont été effectués sur 32 fibres ultra haute performance en béton armé (BFUP) des faisceaux d'étudier les effets de certains paramètres sur la résistance à la flexion et la ductilité des poutres en BFUP avec l'extérieur renforcer avec des feuilles de PRFC. Les principaux paramètres inclus dans cette étude étaient la quantité d'acier d'armature interne, la fraction volumique de la fibre et les configurations de mise en place des ancrages transversales le long de la fibre de carbone longitudinale externe. Les résultats des tests ont montré que l'externe des feuilles de PRFC de collage sur l'aile inférieure des poutres en BFUP conduit à augmenter la capacité de charge, mais de réduire la ductilité en flexion. De plus, les résultats ont montré que les ancrages en CFRP pourrait augmenter considérablement la ductilité en flexion, outre les ancrages transversaux étaient suffisantes pour éliminer décollement, et les poutres en BFUP a échoué lorsque les matériaux CFRP longitudinales fracturé, dans d'autres cas, les ancrages transversaux simplement retardé décollement et la longitudinales matériaux CFRP décollée après les ancrages transversaux fracturé.

MOTS CLÉS: CFRP, BFUP, Résistance à la flexion, ductilité.

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1. INTRODUCTION

Typical recent strengthening techniques involve the use of: a) external bonded steel plates[1,2]; b) bonded fiber-reinforced polymer (FRP) plates or fabrics[3,4,5]; or c) externally applied FRP post-tensioning strands [6]. FRP plates, fabrics, or rods offer numerous advantages such as: a) structural benefits: FRP materials have very high strength, and have a higher strength/weight ratio than steel or concrete [7,8,9]; hence, the strength and stiffness can be increased without a significant increase in the loads ; b) life cycle benefits: FRP materials offer high resistance against corrosion and other elements [10,11]; and c) economic benefits: installation time and cost are less than traditional retrofit techniques [12]. Recognizing the benefits of FRP plates and fabrics as external flexural and shear reinforcement to revitalize the deteriorating infrastructure, a significant amount of research has been conducted in recent years to characterize the material properties and behavior of FRP, to examine various issues related to behavior of members and systems strengthened with FRP, and to develop analysis and design methods for FRP reinforced concrete members.

Structural performances of steel fiber reinforced concrete beams with externally bonded FRP developed by Yin and Wu [13] to improve the FRP strengthening performance to concrete beams by mixing short steel-fibers into the concrete matrix. Test results [13] showed that for FRP-strengthened concrete beams by increasing steel-fiber volume fraction, leads to a smeared crack distribution in the concrete. The failure mode also changed from peeling-induced debonding to FRP rupture so that the FRP sheet can exert its strengthening effect sufficiently.

Also Yongchang Guo and Feng Liu [14] tested two different types of fiber reinforced concrete (FRC) beams, which are strengthened with three different types of fiber reinforced plastic (FRP) sheets. Three strengthening schemes have been used, which are mono-layered carbon fiber reinforced plastic (CFRP) sheet strengthening, mono-layered glass fiber reinforced plastic (GFRP) sheet strengthening and CFRP mixed GFRP bi-layer sheets strengthening, respectively. The failure modes of test beams [14] also changed from peeling-induced debonding into FRP rupture, which shows that the FRP sheets can exert its strengthening effect sufficiently.

Structural performances of high strength reinforced concrete beams with externally bonded FRP developed by Akbarzadeh and Maghsoudi [15] to study the flexural behavior and redistribution in moment of reinforced high strength concrete (RHSC) continuous beams strengthened with CFRP and GFRP sheets. Test results [15] showed that with increasing the number of CFRP sheet layers, the ultimate strength increases, while the ductility, moment redistribution, and ultimate strain of CFRP sheet decrease.

In this research, tests were carried out on ultra high performance fiber reinforced concrete (UHPFRC) beams to study the effects of some parameters on the flexural strength and ductility of UHPFRC beams with externally strengthening with CFRP sheets and reaching to improved properties of UHPFRC.

The objectives of this study have been:

1. How effective is placement of transverse anchorages along the longitudinal CFRP in enhancing anchorage and ductility of UHPFRC beams with externally CFRP Sheets?
2. Study the effective of amount of internal steel and fiber volume fraction on flexural strength and ductility of UHPFRC beams with externally CFRP Sheets.
3. Evaluating the failure modes of UHPFRC beams with externally CFRP Sheets.

2. EXPERIMENTAL WORK

2.1 Test Specimens and Loading Arrangement

A total of thirty two UHPFRC beams with 100 mm × 200 mm cross section, 1600 mm total length, with different volume fractions of steel-fiber, different amount of internal steel reinforcement and different strengthening configurations are cast and tested in this research. These UHPFRC beams were arranged into two groups as following. Details of experimental programs for all tested UHPFRC beams are given in table (1).

Group (A) consist from sixteen UHPFRC beams, had the top reinforcement of two 10 mm diameter bars and bottom reinforcement of two 12 mm diameter bars had a ratio 1.33 %, with transverse reinforcement of 8 mm diameter stirrups at 110 mm centers had a ratio 0.91 %.
Group (B) consist from sixteen UHPFRC beams, had the top reinforcement of two 10 mm diameter bars and bottom reinforcement of four 12 mm diameter bars had a ratio 2.83 %, with transverse reinforcement of 8 mm diameter stirrups at 110 mm centers had a ratio 0.91 %.
The first part of each group consist from four UHPFRC beams without external strengthening used as control beams. The second part of each group consist of twelve UHPFRC beams strengthened with CFRP sheets. Each beam in the second part of each group strengthened with one external layer of CFRP sheets, with three different configurations of placement of transverse anchorages along the external longitudinal CFRP as shown in Fig. (1).

2.2 Materials

Natural crushed basalt graded from 2.36 mm to 9.5 mm (nominal max. size) with fineness modulus equal 5.3 was used. Harsh desert fine sand with fineness modulus equal 2.28 was used, it was almost free from impurities, silt, loam and clay. Ordinary Portland cement with high grade 52.5N was used.
The silica fume used in this work, is locally produced by Sika Egypt (Sika Fume-HR), It is used in preparing ultra high strength concrete mixes. Sikament-NN, is used as a high-range water-reducing admixture, It complies with

Table (1) Groups details and test program

Beam Group	Beam Notation	Main Steel	Volume frication of fiber	Case of external Strengthening	Total No. of transverse Anchorages	CFRP configuration
Group A	A0	2Ø12	0%	—	—	
	A0-S0			strengthened	0	I
	A0-S6			strengthened	6	II
	A0-S12			strengthened	12	III
	A1	2Ø12	1%	—	—	—
	A1-S0			strengthened	0	I
	A1-S6			strengthened	6	II
	A1-S12			strengthened	12	III
	A2	2Ø12	2%	—	—	—
	A2-S0			strengthened	0	I
	A2-S6			strengthened	6	II
	A2-S12			strengthened	12	III
	A3	2Ø12	3%	—	—	—
	A3-S0			strengthened	0	I
	A3-S6			strengthened	6	II
	A3-S12			strengthened	12	III
Group B	B0	4Ø12	0%	—	—	—
	B0-S0			strengthened	0	I
	B0-S6			strengthened	6	II
	B0-S12			strengthened	12	III
	B1	4Ø12	1%	—	—	—
	B1-S0			strengthened	0	I
	B1-S6			strengthened	6	II
	B1-S12			strengthened	12	III
	B2	4Ø12	2%	—	—	—
	B2-S0			strengthened	0	I
	B2-S6			strengthened	6	II
	B2-S12			strengthened	12	III
	B3	4Ø12	3%	—	—	—
	B3-S0			strengthened	0	I
	B3-S6			strengthened	6	II
	B3-S12			strengthened	12	III

ASTM C494 type F and BS 5075 Part 3, In this work, it is used in preparing ultra high strength concrete mixes after many trails.

Two types of reinforcing bars were used in this work. The first was locally produced high strength steel with (f_y/f_{ult}=36/52) as deformed bars used as longitudinal reinforcement. The second one was ordinary plain mild steel with (f_y/f_{ult}=24/35) was used as stirrups (web reinforcement). A steel fibers of diameter 0.60 mm , were used in fiber concrete with length 30 mm, with hooked end and tensile strength equal 1100 Mpa, it complies with ASTM A 8 and ACI S44-3R.

CFRP sheets are the strengthening materials used. The carbon fibers used in this research study were in the form of dry unidirectional flexible sheets, commercially named as SikaWarp Hex-230c and its impregnating resin was Sikadur-330. Some of these specifications for CFRP sheets as shown in table (2) [16]. Strengthening configuration of tested beams are shown in Fig. (2).

Table (2) Typical CFRP Sheets Properties

Property	Value
Sheet design thickness	0.12 mm (based on total area of carbon fibers)
Tensile strength of fibers	4100 N/mm^2
Tensile E-modulus of fibers	231000 N/mm^2
Elongation at break	1.7 %

2.3 Concrete Mix Design

The absolute volume method recommended by the ACI Committee was used to compute the quantities of material required for the test batch. Four mixes were designed in this work in order to get the required ultra-high compressive strength. Many trial mixes were made to adjust the proportions of the used materials to give the needed compressive strengths. The concrete mix proportions required for 1 m3 concrete are given in table (3). Compressive strength of all cubes at 28 day of tested beams varied between 170 and 180 N/mm^2 .

Since it is important to have a homogeneous concrete mix. First, cement, silica fume, coarse and fine aggregate were premixed for about 2-3 min. Then, water and water-reducing admixture were added and mixed for about 1 min. When the mixture became flowable, the steel fibers were added and mixed for an additional 1 min. All mixing procedures were carried out at room temperature about (20 - 25°).

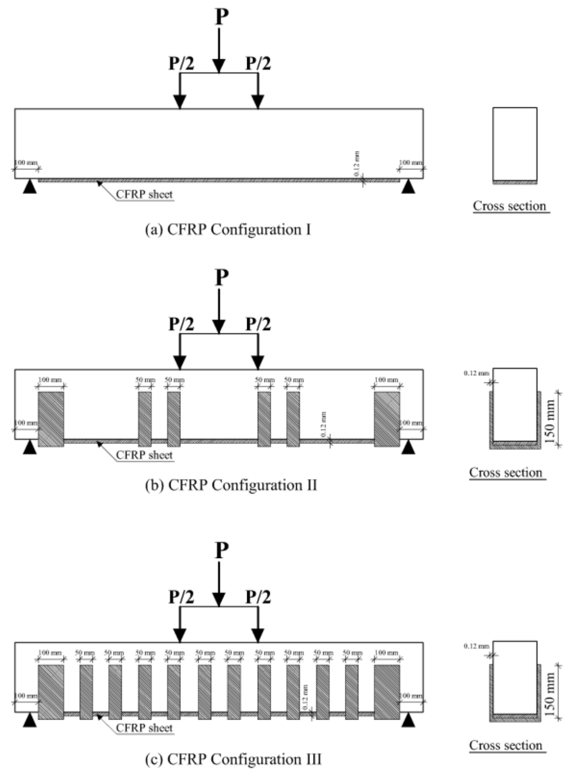

Fig. (1) Scheme of strengthening of tested beams

Table (3) Concrete mix proportions

Ingredient	Amount (kg/m^3)			
	Batch 1	Batch 2	Batch 3	Batch 4
Cement, c	680	680	680	680
Silica fume, s	204	204	204	204
Basalt 2.36 – 5 mm	556.6	545.8	534.9	524
Basalt 5 – 9.5 mm	320	313.7	307.5	301.2
Sand	472	462.8	453.6	444.3
Steel fibers	0	78	156	234
Water, w	149.6	149.6	149.6	149.6
Superplastici-zer	35.36	35.36	35.36	35.36
w/c	0.22	0.22	0.22	0.22
w/(c+s)	0.17	0.17	0.17	0.17

2.4 Testing And Measurements

Deflections were measured at mid span of the beams. One Linear Variable Distance Transducers ,LVDT, having a maximum range of 100 mm and reading to 0.01 mm were used in the test. Strains were determined at different positions at top concrete fiber, on tensile steel reinforcement and on longitudinal CFRP at mid span of the beam as shown in Fig. (2). Electrical strain gages produced by KYOWA ELECTRONIC INSTRUMENTS CO., LTD., Tokyo, Japan. The product type for the steel, the CFRP sheets and the concrete were KFG-5-120-C1-11, KFG-20-120-C1-11 and KFG-50-120-C1-11 respectively. The strain gages data were collected using a data logger system.

The available hydraulic testing machine (Avery Denison-England, 1000 KN PU) was used, which controlled the concrete dimensions of the tested beams. The beams were rested on two roller supports to avoid restraint to the elongation of the bottom of fibers of the beams as load was applied. The applied load by the testing machine was transmitted to the tested beams through a spreader beam (I-beam) supported on two cylinder bars giving two point loading test in order to obtain a zone of constant bending moment and zero shear. The distance between the two loading points was taken 300 mm for beams with span 1440 mm. Fig. (2) shows the loading arrangement and the overall test setup.

Fig. (2) Loading arrangement and measuring instruments

3. RESULTS AND DISCUSSION

3.1 Crack Patterns and Modes of Failure

Modes of failure of the tested beams are shown in table (3). The modes of failure observed for the tested beams without external strengthening were ductile flexural failure (tension failure) observed in all beams. All strengthened beams failed by yielding of tension steel reinforcement followed by three different modes of failure of external CFRP. The first was debonding of CFRP from concrete surface , as shown in Figs. (3), (4). The second was debonding of CFRP from concrete surface followed by rupture of CFRP sheets, as shown in Figs. (5), (6). The third was rupture of CFRP sheets , as shown in Figs. (7), (8).

In the uncracked elastic stage, the same behavior was observed for all strengthened tested beams, indicating larger beams cracking load than the control beams. In the cracked preyield stage, the stiffness and yield load of the CFRP strengthened beams were moderately larger than that of the control beam. However, significant decreases in beams stiffness was observed after yielding the tensile steel. After yielding, the strengthened beams. Secondary cracks formed in the vicinity of the flexural cracks after yielding of the longitudinal reinforcement, and these cracks tended to propagate along the sides of the CFRP materials. The beams experienced small relative vertical movements on either side of flexural cracks within the shear span, and this movement caused the composite materials to pry off the concrete surface. The combination of the longitudinal cracks along the edges of the CFRP materials and prying action within the composite led to debonding of the CFRP materials from the surface of the concrete. Debonding was initiated in regions of high moment within the shear span in all specimens. The transverse anchorages tended to control debonding of the longitudinal composite materials because the growth of the longitudinal cracks along the edges of the composites was delayed by the anchorages, as shown in Fig. (9). In some cases, the transverse anchorages were sufficient to eliminate debonding, and the beams failed when the longitudinal CFRP materials fractured, as shown in Fig. (8). In other cases, the transverse anchorages simply delayed debonding, and the longitudinal CFRP materials debonded after the transverse anchorages fractured as shown in Fig. (10).

The main difference in the failure mode of beams noted with S12 with supplementary anchorages in addition to end anchorages and additional anchorages at crtical sections compared to that of in beams noted with S6 with only end anchorages and the additional anchorages at crtical sections consisted in the greater ductility, slow debonding and slippage of CFRP sheets, more diffused and less destructive and nose damage of the overall beam at failure due to partial confinement of concrete and the better positioning of the supplementary anchorages.

Adding fibers to UHPC beams can change the crack patterns, delay the crack appearance and restrain the crack expansion in concrete specimen, the same results can be obtained by externally bonding CFRP sheets on the bottom flange of tensile sections of UHPC beams. Compared with UHPC beams, the UHPC beams strengthened with CFRP sheets have closer and thinner cracks under loads. The beams of higher steel-fiber fraction and supplementary anchorages noted with S12, had better deformational behavior with longer gentle softening curves. Also, at the same load level the crack propagation was effectively controlled.

As the tension steel ratio and amount of internal fiber increased, the debonding load of strengthened beams was delayed. In addition, the cracks occurs later and were thinner and closer at the same load levels.

Fig. (3) Mode of failure of tested beam A0-S0

Fig. (4) Mode of failure of tested beam B2-S0

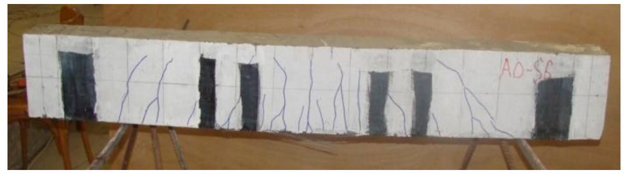

Fig. (5) Mode of failure of tested beam A0-S6


Fig. (6) Close view showing debonding and rupture of CFRP sheet of tested beam A0-S12

Fig. (7) Mode of failure of tested beam A2-S12

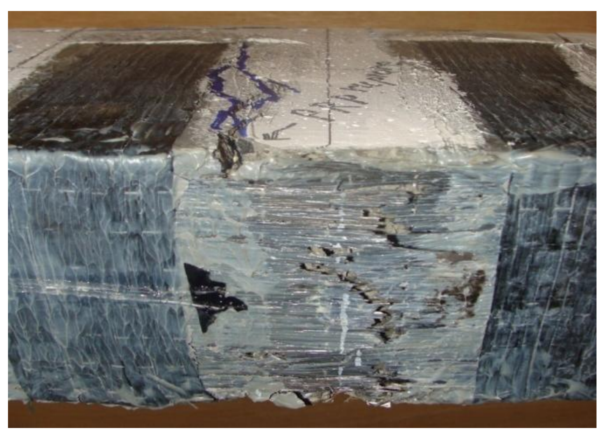
Fig. (8) Close view showing sudden rupture of CFRP sheet of tested beam A3-S12


Fig. (9) Mode of failure of tested beam B0-S12

Photo (10) Mode of failure of tested beam B3-S12

3.2 Load – Deflection Relationship

Deflection of the tested beams was measured at mid-span for each beam. Table (4) shows the max. values of deflections for each beam at different stage of loading. Figs. from (11) to (18) were plotted to represent the relation between load and the corresponding central deflection. It can be seen that all the beams exhibited linear behavior from initial loading up to the occurrence of the first crack. After the formation of the cracks, all beams showed non-linear behavior. The response of the strengthened beams was essentially the same as the response of the control beams before the concrete cracked. After cracking, the strengthened beams tended to be stiffer than the control beams. Comparison of the peak deflection at ultimate load reveals that there was great improvement in the deflection levels, due to the inclusion of steel fiber. Although the effect of steel fibers on the strengthened beams was essentially the same as the response of the

Table (4) Test results and observed failure modes

Group	Notion	Load (KN)		Deflection (mm)		Ductility Index	Failure mode
		Yield	Ultimate	Yield	Ultimate		
	A0	59	70	12.9	25.5	1.98	Flexural tension failure
	A0-S0	83	98	12.8	15.3	1.2	Debonding of CFRP sheet
	A0-S6	90	112	12.8	18.3	1.43	Debonding and rupture of CFRP sheet
	A0-S12	96	116	12.4	19	1.53	Debonding and rupture of CFRP sheet
	A1	61	75	13.2	27.3	2.07	Flexural tension failure
	A1-S0	90	109	11.5	15.8	1.37	Debonding of CFRP sheet
	A1-S6	98	121	11.6	18.7	1.61	Debonding and rupture of CFRP sheet
	A1-S12	104	124	11.8	19.3	1.64	Debonding and rupture of CFRP sheet
A	A2	64	83	13.4	30.2	2.25	Flexural tension failure
	A2-S0	96	129	11.3	16.3	1.44	Debonding of CFRP sheet
	A2-S6	101	139	12	19.1	1.59	Debonding and rupture of CFRP sheet
	A2-S12	105	141	11.4	19.6	1.72	Rupture of CFRP sheet
	A3	70	93	13.7	34.1	2.48	Flexural tension failure
	A3-S0	104	149	11.2	17.1	1.53	Debonding of CFRP sheet
	A3-S6	109	158	11.7	19.3	1.65	Rupture of CFRP sheet
	A3-S12	112	160	11.5	19.9	1.73	Rupture of CFRP sheet
	B0	126	143	11.5	18.1	1.57	Flexural tension failure
	B0-S0	160	190	11.6	14.8	1.28	Debonding of CFRP sheet
	B0-S6	169	205	11.3	15.5	1.37	Debonding and rupture of CFRP sheet
	B0-S12	175	207	11.6	17.4	1.5	Debonding of CFRP sheet
	B1	130	152	11.4	22.5	1.97	Flexural tension failure
	B1-S0	168	204	10.7	17.6	1.64	Debonding of CFRP sheet
	B1-S6	174	216	10.9	18.5	1.7	Debonding and rupture of CFRP sheet
	B1-S12	180	220	10.9	19.7	1.81	Debonding of CFRP sheet and rupture of anchorage
B	B2	137	161	11.7	25.4	2.17	Flexural tension failure
	B2-S0	174	225	10.6	18.8	1.77	Debonding of CFRP sheet
	B2-S6	179	235	10.8	19.4	1.8	Debonding and rupture of CFRP sheet
	B2-S12	183	237	10.6	20.3	1.92	Debonding of CFRP sheet and rupture of anchorage
	B3	143	173	11.9	27.3	2.29	Flexural tension failure
	B3-S0	180	251	10.2	19.1	1.87	Debonding of CFRP sheet
	B3-S6	184	256	10.2	19.6	1.92	Debonding and rupture of CFRP sheet
	B3-S12	187	258	10	20.5	2.05	Debonding of CFRP sheet and rupture of anchorage

control beams, the results showed that the strengthened beams of higher steel-fiber fraction and supplementary anchorages had better deformational behavior.

Figs. from (11) to (16) showed that although most of the strengthened beams failed at loads that exceeded the capacity of the control beams, all the strengthened beams failed at deflections levels that were considerably less than the capacity of the control beams. Also the results confirm that providing end and other anchorages enhances the deflection behavior.

Figs. (17), (18) indicated that the increase in terms of ultimate load provided by external strengthening with CFRP sheets for UHPC beams was as significant as higher the volume fraction of steel fiber (V_f) due to the effect of adding steel fiber in reduce concrete compressive stresses and strain. Flexural strengthening with CFRP sheets for UHPC beams with ρ=1.33% causes maximum increase in ultimate loads by 65.71% for V_f = 0%, by 65.33% for V_f = 1%, by 69.88% for V_f =2% and by 72% for V_f =3% , while for beams with ρ=2.83% increased by 44.76% for V_f =0%, by 44.74% for V_f = 1%, by 47.2% for V_f =2% and by 49.13% for V_f =3%. Also the results confirm that the increase in terms of ultimate loads provided by steel fiber reinforcement or by external CFRP were as significant as lower the percentage of the conventional tension steel ratio.

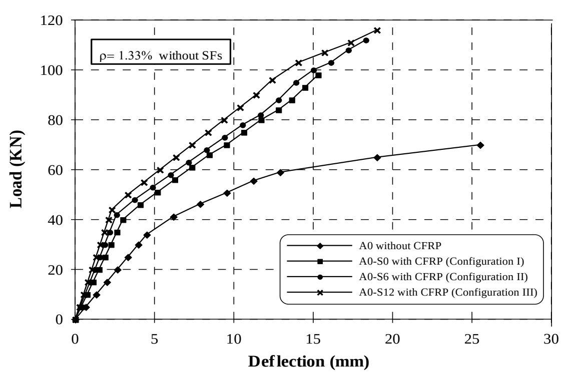

Fig. (11) Load – Deflection curve of beams without steel fiber

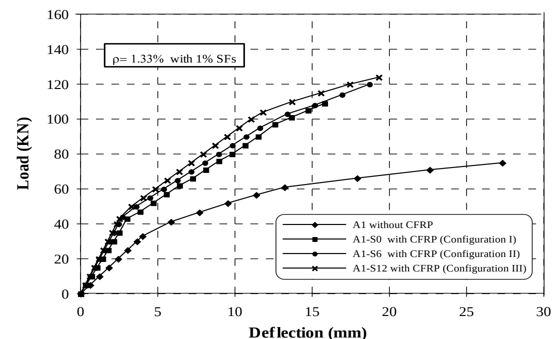

Fig. (12) Load – Deflection curve of beams with 1% volume fraction of steel fiber

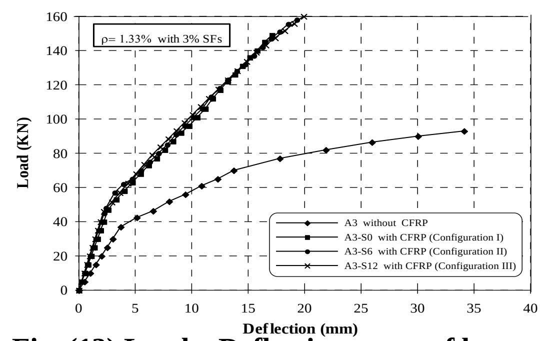

Fig. (13) Load – Deflection curve of beams with 3% volume fraction of steel fiber

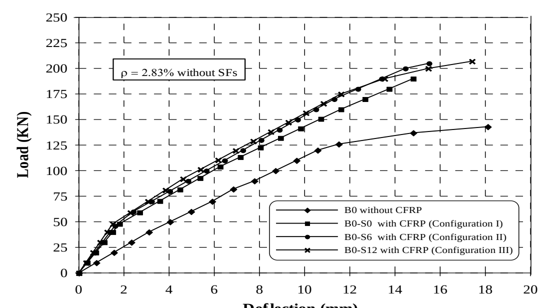

Fig. (14) Load – Deflection curve of beams without steel fiber

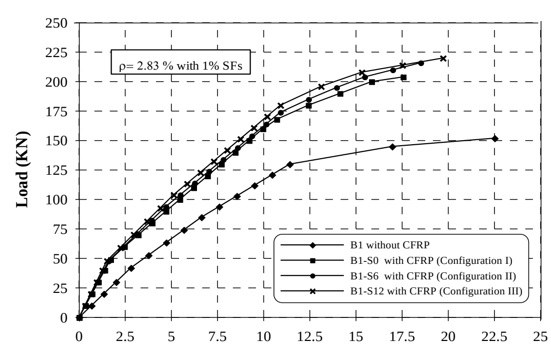

Fig. (15) Load – Deflection curve of beams with 1% volume fraction of steel fiber

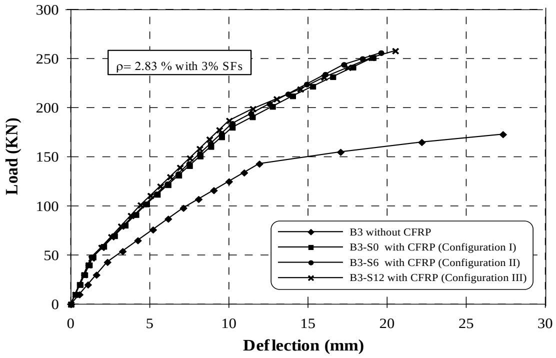

Fig. (16) Load – Deflection curve of beams with 3% volume fraction of steel fiber

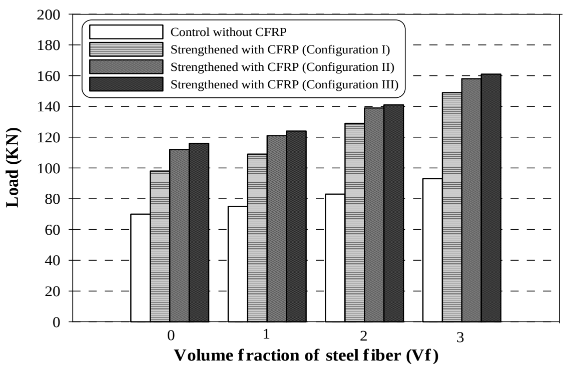

Fig. (17) Effect of CFRP on ultimate load of tested beams with internal steel ratio (ρ)= 1.33%

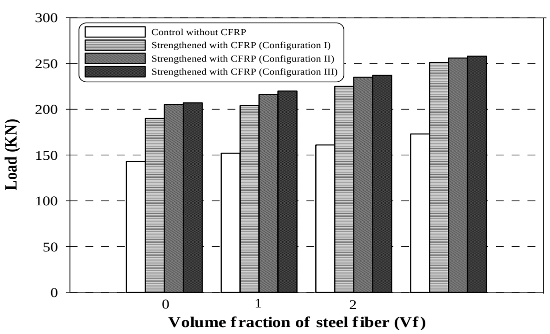

Fig. (18) Effect of CFRP on ultimate load of tested beams with internal steel ratio (ρ)= 2.83%

3.3 Ductility

Ductility is a desirable structural property because it allows stress redistribution and provides warning of impending failure. The ductility of each tested beam was determined by calculating its ductility index; that is, the ratio between the ultimate deflection and the yield deflection at mid span of the tested beam. The values of the flexural ductility of UHPC beams increased by the addition of fiber, and decreased by external strengthening with CFRP. Test results have also shown that CFRP transverse anchorage could significantly increase the flexural ductility.

Table (4) indicate that the increase of volume fraction of fiber from 0% to 3% for control beams causes increase in ductility index by 25.3% for beams with internal steel ratio 1.33% and by 45.9% for beams with internal steel ratio 2.83%. Hence, the increase in terms of ductility index provided by steel fiber reinforcement was as significant as higher the percentage of the conventional tension steel ratio.

Fig. (19) indicate that strengthening with CFRP sheets for beams with tension steel ratio 1.33% causes maximum decrease in ductility index by 39.4% for beams with volume fraction of fiber 0%, by 33.8% for beams with volume fraction of fiber 1%, by 36% for beams with volume fraction of fiber 2% and by 38.3% for beams with volume fraction of fiber 3% . Fig. (20) indicate that strengthening with CFRP sheets for beams with internal steel ratio 2.83% causes maximum decrease in ductility index by 18.5% for beams with volume fraction of fiber 0%, by 16.8% for beams with volume fraction of fiber 1%, by 18.4% for beams with volume fraction of fiber 2% and by 18.3% for beams with volume fraction of fiber 3%. Hence, the decrease in ductility index provided by CFRP sheets was as significant as lower the volume fraction of fiber due to the early debonding behavior and the effect of adding steel fiber in control and redistribute cracks and compressive stresses.

3.4 Load – Strain Relationship

Strains of the tested beams were measured at mid-span for each beam. Table (5) shows the maximum values of strains for concrete in compression, steel and CFRP in tension.

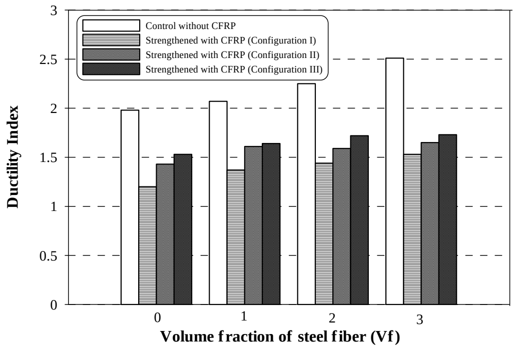

Fig. (19) Effect of CFRP on ductility of tested beams with internal steel ratio (ρ)= 1.33%

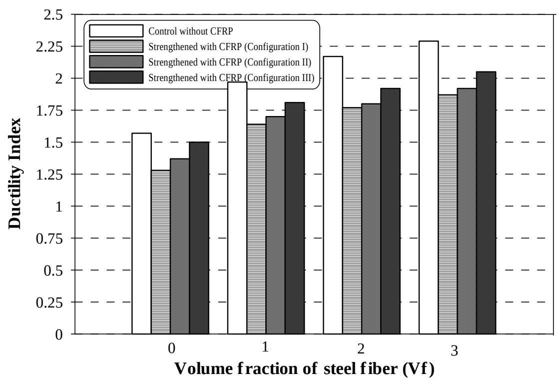

Fig. (20) Effect of CFRP on ductility of tested beams with internal steel ratio (ρ)= 2.83%

Comparison of the strains at ultimate load reveals that there was great improvement in the strain levels, due to the inclusion of steel fiber. Test results showed that the compressive concrete strain is directly proportional to amount of steel fibers. Also the results showed that for UHPFRC beams, when beams reached the ultimate, concrete was held together and the strain in concrete and strain in reinforcement kept increasing gradually. The effect of steel fibers on the strengthened beams was essentially the same as the response of the control beams. Also the results showed that the strengthened beams of higher steel-fiber fraction and supplementary anchorages had higher strain levels.

It was also observed that, in the unstrengthened beam, the stress in steel bars increases until the steel reaches its yield point. Thereafter, a large portion of any extra stress is absorbed by large deformations in the steel, which lowers the increase of concrete compressive strain. In strengthened beams, tensile stresses are shared between the steel bars and the strengthening sheets, so the stresses carried by the steel bars will be less. Therefore, concrete strains in the strengthened beams are higher than those in the control beam at the same load levels.

Table (5) Summary of measured strains

Group	Notion	Strain × 10^{-3}			CFRP Strain Ratio*
		Concrete	Steel	CFRP	
A	A0	2.1	32.4	—	—
	A0-S0	1.8	16.3	5.8	0.34
	A0-S6	1.95	18.8	7.1	0.42
	A0-S12	2.25	21.2	8.7	0.51
	A1	2.8	35.6	—	—
	A1-S0	2.5	19.6	7.5	0.44
	A1-S6	2.8	22.1	9.5	0.56
	A1-S12	3.25	24.5	10.5	0.62
	A2	3.5	38.3	—	—
	A2-S0	3.15	23	9.8	0.58
	A2-S6	3.7	25.9	11.9	0.70
	A2-S12	4.25	27.6	12.7	0.75
	A3	4.3	42.2	—	—
	A3-S0	4	27.4	11.4	0.67
	A3-S6	5.1	30	13	0.76
	A3-S12	5.6	31.5	13.7	0.81
B	B0	2.6	19.1	—	—
	B0-S0	2.3	11.5	7.3	0.43
	B0-S6	2.5	13.1	8.8	0.52
	B0-S12	2.75	14.4	10.9	0.64
	B1	3.5	21.3	—	—
	B1-S0	3.15	13.8	9.4	0.56
	B1-S6	3.5	15.3	11.9	0.70
	B1-S12	3.9	16.6	13.1	0.77
	B2	4.4	24.5	—	—
	B2-S0	4.05	17.2	11.7	0.69
	B2-S6	4.9	18.9	14.3	0.84
	B2-S12	5.3	19.8	15.2	0.89
	B3	5.4	26.7	—	—
	B3-S0	5.1	20	13.1	0.77
	B3-S6	6.5	21.5	15	0.88
	B3-S12	6.9	22.2	15.8	0.93

* CFRP strain ratio defined as ratio of measured strain in CFRP at capacity of tested beams divided by strain capacity reported by manufacturer of CFRP (Table 2)

Test results showed that the tensile steel strain and tensile CFRP strain are directly proportional to amount of steel fibers.

The design goal is the achievement of ‘‘ductile’’ structures that reach a significant level of strain in the compression zone at failure. For this purpose, it is necessary to use a material with higher tensile properties than steel, such as high resistance CFRP. The drawback of CFRP is their brittle mechanical behaviour. But, through the combination of CFRP and steel reinforcement the required mechanical behavior of UHPFRC can be achieved.

The strains in the steel increased more rapidly than the strains in the CFRP. This is because the CFRP materials had begun to debond from the surface of the concrete and the axial elongation of the CFRP was now distributed over a longer distance. It should be noted that, the measured CFRP strains for all beams at capacity were considerably less than the fracture strain reported by the manufacturers. The average strain in the CFRP at capacity was approximately 60% of the reported rupture strains when all the test beams were considered. This value increased to approximately 80% of the reported rupture strain in the beams that failed when the longitudinal composites ruptured.

4. CONCLUSIONS

Findings from the experimental study on the flexural strength and ductility of UHPFRC beams with externally flexural strengthening with CFRP, and the analysis of such test results and variables the following points:

1. Adding fibers to UHPC beams can change the crack patterns, delay the crack appearance and restrain the crack expansion in concrete specimen, the same results can be obtained by externally bonding CFRP sheets on the bottom flange of tensile sections of UHPC beams. Compared with UHPC beams, the UHPC beams strengthened with CFRP sheets have closer and thinner cracks under loads.
2. The increase in terms of ultimate load provided by external strengthening with CFRP sheets for UHPFRC beams was as significant as higher the volume fraction of steel fiber (V_f) due to the effect of adding steel fiber in reduce concrete compressive stresses and strain. Flexural strengthening with CFRP sheets for UHPFRC beams with ρ=1.33% causes maximum increase in ultimate loads by 65.71% for V_f = 0%, by 65.33% for V_f = 1%, by 69.88% for V_f =2% and by 72% for V_f =3% , while for beams with ρ=2.83% increased by 44.76% for V_f =0%, by 44.74% for V_f = 1%, by 47.2% for V_f =2% and by 49.13% for V_f =3%.
3. As the tension steel ratio and amount of internal steel fiber increased, the debonding load of UHPFRC strengthened beams was delayed. In addition, the cracks occurs later and were thinner and closer at the same load levels.
4. The increase in terms of ductility index provided by steel fiber reinforcement for strengthened UHPC beams with CFRP sheets was as significant as lower the number of transverse anchorages. The increase of volume fraction of steel fiber from 0% to 3% for strengthened UHPC beams without transverse anchorage (configuration I) causes maximum increase in ductility index by 27.5% for ρ =1.33% and by 46.1% for ρ=2.83% , while for strengthened UHPC beams provided with end anchorages and additional anchorages at critical sections (configuration II) increased by 15.4% for ρ =1.33% and by 40.1% for ρ=2.83%, while for strengthened UHPC beams provided with supplementary anchorages at full length(configuration III) increased by 13.1% for ρ = 1.33% and by 36.7% for ρ =2.83%.
5. The decrease in terms of ductility index provided by external strengthening with CFRP sheets for UHPC beams was as significant as lower the percentage of the conventional internal steel ratio (ρ). Flexural strengthening with CFRP sheets for UHPC beams with ρ=1.33% causes maximum decrease in ductility index by 39.4% for V_f = 0%, by 33.8% for V_f = 1%, by 36% for V_f =2% and by 38.3% for V_f =3% , while for beams with ρ=2.83% decreased by 18.5% for V_f =0%, by 16.8% for V_f = 1%, by 18.4% for V_f =2% and by 18.3% for V_f =3%.
6. The CFRP transverse anchorages could significantly increase the ductility and the higher the amount of steel fiber the lower the flexural ductility enhancement achieved by adding CFRP anchorages to strengthened beams.
7. The CFRP transverse anchorages were sufficient to eliminate debonding, and the beams failed when the longitudinal CFRP

materials fractured. In other cases, the transverse anchorages simply delayed debonding, and the longitudinal CFRP materials debonded after the transverse anchorages fractured.

8. Debonding initiated at the location of flexural cracks near the applied loads within the shear span. Therefore, transverse anchorages placed close to regions of high moment within the shear span are more effective than those positioned near the end of the span.
9. The increase in terms of ultimate load provided by CFRP transverse anchorages was as significant as lower the volume fraction of fiber due to the early debonding behavior of beams with low volume fraction of fiber.
10. The internal steel rebar ratio and the amount of steel fiber of the UHPFRC beams influence the type of anchorage system that will be most effective in enhancing the strength and ductility properties of the UHPFRC beams strengthened with CFRP sheets.

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