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FLEXURAL STRENGTH OF FIBROUS ULTRA HIGH PERFORMANCE REINFORCED CONCRETE BEAMS

Wasan I. Khalil[1] and Tayfur Y. R.[2]
[1]Building and Construction Engineering Department, University of Technology, Baghdad, Iraq
[2]Structural Engineering, Sulaimaniyah, Iraq
E-Mail: Wasan1959@yahoo.com

ABSTRACT

The flexural behavior of eleven 150×150×1950 mm ultra high performance conventionally reinforced concrete beams containing hooked and crimped steel fibers with different volume fractions (0.5%, 0.75% and 1%) in full and partial depths of beams cross sections is studied in this investigation. The load deflection relationship, resilience, toughness indices, first crack load, ultimate load and concrete strains were investigated. The experimental results show that the addition of steel fibers slightly enhances the load-deflection relationship and ultimate load for beam specimens. The type of steel fibers (crimped and hooked) has a little effect on load-deflection behavior, ultimate moment capacity, cracking pattern, while in resilience and toughness, beam specimens with hooked steel fiber showed slightly better behavior than those with crimped steel fibers. The ultimate tensile strength of beams has been rederived and contributed in order to calculate the moment capacity. The calculated ultimate moment capacity was in good agreement with the experimental ultimate moment capacity.

Keywords: ultra high performance concrete, hooked steel fibers, crimped steel fibers, volume fraction.

INTRODUCTION

Ultra High Performance Concrete (UHPC) is an important recent development in construction materials with high compressive strength and other perfect properties [1, 2]. This type of concrete shows very brittle failure behavior and therefore a limited post-crack behavior, so the elements fail explosively without any omen [3], to overcome this problem; fibers should be incorporated into cement matrix to increase the toughness and tensile strength and to improve the cracking deformation characteristics of the resultant composite. This type of concrete named Ultra High Performance Fiber Reinforced Concrete (UHPFRC). UHPFRC belong to the group of High Performance Fiber Reinforced Cement Composites (HPFRCC) which defined as the kind of Fiber reinforced Concrete (FRC) that exhibit strain-hardening under uniaxial tension force. In addition, UHPFRC is characterized by a dense matrix and consequent a very low permeability when compared to HPFRCC and normal strength concretes. UHPFRC is suitable for use in (i) the fabrication of precast element such as bridge components, (ii) archi- structural features, (iii) durable component exposed to marine or aggressive environment, (iv) blast or impact protective structures, (v) strengthening material for repair/rehabilitation work for deteriorated reinforced concrete structures [4]. To achieve excellent mechanical behavior some special techniques and raw materials must be adopted in the preparation of UHPC, one of these is the removal of coarse aggregate to enhance the homogeneity of concrete [5].

The use of UHPFRC offers many benefits to structures in comparison to conventional concrete. These include the followings:

a) Superior ductility and energy absorption provides greater structural reliability even under overload conditions or earthquakes.
b) The elimination of supplemental reinforcing steel allows nearly limitless structural member shape and form freedom and also reduces high labor costs associated with it.
c) Enhanced absorption resistance provides extended life for bridge decks and industrial floors.
d) Superior corrosion resistance provides protection from deicing, chemicals and continuous exposition to humid environments.
e) Superior strength results in significant weight reduction which produces more slender transportation structures, reduces overall costs and increases usable floors space in high rise buildings [6].

Considerable research studies have been performed to investigate the mechanical properties of the UHPFRC. However little has been published to investigate the structural behavior of UHPFRC. Arunachalam and Vigneshwari [7] carried out an experimental investigation on the effect of using fly ash and rice husk ash (10% and 20% as replacement by weight of cement) as mineral admixtures on the properties of UHSFRC containing 25% silica fume as addition by weight of cement and 2.5% steel fibers by volume of concrete. Test results show that UHSFRC containing mineral admixtures have satisfactory mechanical performance. Rice husk ash at 20% replacement of cement gave higher compressive and split tensile strength than the other. Victor and Cornelia [8] present the bending behavior of UHPFRC beams. The experimental was composed of creating and testing eight beams (I-shaped cross section), two at the same time for each percent of reinforcement (1.5% and 2.55%) and for each type of used fibers (hybrid and long fibers). Each

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beam have span of 3m and was tested at bending in four points. The results revealed very good behavior in exploiting of the beams in bending. It is remarked the linear behavior between the moment and the deflection, until a load stage of approximately 85-90% from the value of the ultimate bending moment.

In this investigation, UHPC with good workability and compressive strength of about 149 MPa was prepared without the removal of the coarse aggregate in order to reduce the cost of UHPC. It includes studying the effect of fiber content, fiber shape and dispersion depth of steel fibers on the flexural behavior of UHPC beams with conventional steel reinforcement.

EXPERIMENTAL WORK

Materials

Ordinary Portland cement with physical and chemical properties conform to the provisions of Iraqi specification No. 5, local natural sand it's gradation lays in zone (2) according to Iraqi specification No. 45, local normal weight crushed aggregate of maximum size 12.5mm it's grading conforms to the Iraqi specification No.45 were used in this investigation. High range water reducing admixture (HRWRA) commercially known as Flocrete Sp95 was also used, it is type F according to ASTM C 494 specifications. The reduction in water content was determined in order to obtain a constant workability with slump 60±5 mm. Condensed silica fume was used as pozzolanic admixture, it's physical requirement, pozzolanic activity index and chemical oxide compositions conform to the requirements of ASTM C 1240-05 specifications. Grade 60 ordinary deformed mild-steel reinforcing bars with two different diameters (12 and 10mm) are used as reinforcement in this investigation. The dimensions and strength characteristics of these bars are summarized in Table-1. The results show that the properties and strength of the reinforcing steel bars are within the requirements of the ASTM $_{A615-05}$ specifications. Two types of high carbon steel fibers (hooked and crimped) were also used in this investigation with diameter 0.6mm, length 30mm, tensile strength 2000 MPa, tensile modulus of elasticity 210 GPa.

Table-1. Properties of main and secondary steel bars.

Diameter (mm)	**Yield strength** (MPa)	**Ultimate tensile strength** (MPa)	**Elastic modulus** (Gpa)	**Elongation** (%)	**ASTM $_{A615-05}$ specifications**	
					Min. yield strength (MPa)	**Maximum tensile strength** (MPa)
12	461	721	200	16.1	420	620
10	501	742	200	17.6		

Mixing of concrete

Mixing process was performed by using revolving drum tilting mixer of 0.1 m^3 capacity. The aggregate used is in saturated surface dry condition. Coarse aggregate, sand and silica fume were primary placed in the mixer and they were dry mixed for about 1 minute. The water and super-plasticizer were mixed apart to each other and then they were added to the mix and mixed for about 30 seconds. After that the cement was loaded to the mixer and mixed for about 5 minutes until the mix became homogeneous, finally the steel fiber is added by spreading over the mixture during a period of about one minute and mixed for about another two minutes. This method of mixing is recommended in the ACI committee report 544.

Preparation of specimens

Steel mould was prepared with internal dimensions of 150 x 250 x 1950 mm in order to cast the beam specimens. Flexural and shear design for all eleven beam specimens were done according to the ACI $_{318M-11}$ design code, all beams were singly reinforced with two 12 mm diameter steel bars (minimum reinforcement ratio) as longitudinal tensile reinforcement at the bottom of the specimen and at effective depth of 210 mm, two straight 10 mm bars at the top were used for fixing stirrups and they were cut at the mid-span of the beam. The specimens were also designed to resist shear stresses at the yield of the tensile reinforcement to obtain pure flexural failure at the center of the beam. For this purpose 10 mm diameter stirrups of 8cm c/c were used from the supports to one third of the span for each side. The details of the beams reinforcement is shown in Figure-1. Concrete mix in full-depth fiber reinforced beams was placed in three equal layers; each batch was prepared to have a volume of about one-third of the beam volume. Each layer was compacted by means of internal vibration for about 40-60 seconds. Partial-depth fiber reinforced beams were casted with fiber concrete till the half depth of the beam with two layers; each layer compacted (with internal vibrator). The remained plain concrete placed with another two layers up to the top level of the mould and compacted with the same vibrator. The surface of the beam was leveled with the top of the mould by using steel trowel. Control specimens were prepared from the same mix of plain (without fiber) and steel fiber reinforced concrete for each beam

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specimen. They were compacted by using external vibrating table. The fresh concrete was casted with approximately equal layers each 50 mm for all specimens. Control specimens include cubic specimens of 100 mm for compressive strength, cylindrical specimens of 100×200 for splitting tensile strength, prismatic specimens of 100×100×500 mm for flexural strength. The beam and control specimens were covered with polyethylene sheet for about 24 hours, and then demoulded and covered again with wet burlap for 24 hours before curing.

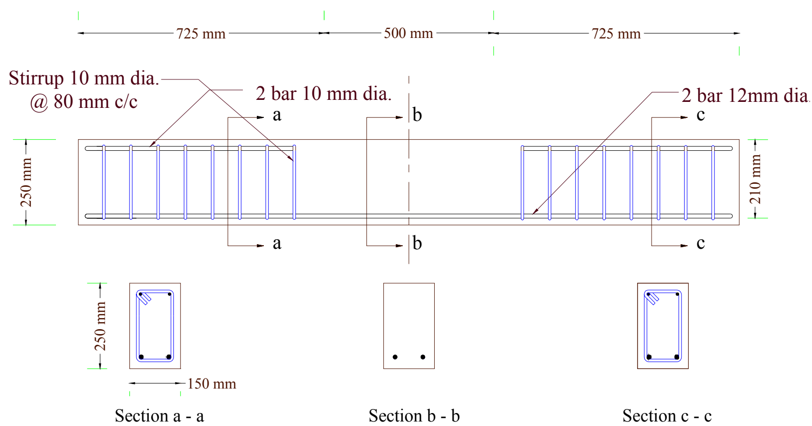

Figure-1. Details of reinforcement for beam specimens (units: mm).

Curing of specimens

After 48 hours from casting, the specimens were totally immersed in water, then the water was heated with slow rate (20°C per hour) until it reaches 60°C to prevent the formation of micro cracks in concrete. The temperature of the curing water remains constant at 60°C during 20 days. After curing the specimens were kept in the laboratory till the time of testing. The beams and the control specimens were immersed in the same curing pool and two automatic control heaters were fixed in the pool to heat the water and to keep its temperature approximately constant during the curing period.

Test variables and flexural strength test for beam specimens

Beam specimens were classified into six series, the variables studied in the test series were, the shape of the steel fiber (hooked and crimped steel fiber), fiber volume fraction (0.5 %, 0.75% and 1%) and the depth of fiber inclusion (full depth fiber inclusion and half depth fiber inclusion). Details of the beam specimens are shown in Table-2. UHPFRC beam specimen was simply supported on 1800 mm span and two concentrated line loads were applied at third points. The distance between the two concentrated loads was 600 mm. The locations of dial gauge used to measure the deflection and demec points for a 200 mm mechanical extensometer are shown in Figure-2. The load was applied step by step with a constant rate of 9 kN. Readings of deflection, mechanical strains were recorded at each load step. At each stage of loading the beam was carefully inspected with amplifier lens in order to detect the first crack and then the load corresponding to the first crack was recorded.

EXPERIMENTAL RESULTS AND DISCUSSIONS

Selection of mix proportions for ultra high performance concrete

Concrete mix was designed to have a compressive strength greater than 60 MPa without the addition of admixtures. The design was done according to ACI Committee 211. Several trail mixes were carried out in order to produce UHPC using silica fume and high range water reducer (HRWR) admixture in the mix. The water cementations ratio was adjusted to have a slump value of 60±5 mm. Two types of high strength steel fibers (hooked and crimped) were also used. Concrete mix with mix proportions 1: 0.65: 1.16 by weight, 900 kg/m^3 cement content, water/cementations ratio 0.22, HRWR dosage 3.5% by weight of cement and silica fume dosage of 15% and 5% by weight of cement as a replacement and addition of cement content respectively was selected. The compressive strength of 100mm cube specimens is about 137 MPa for plain concrete (without fibers) after 20 days of hot water curing. Hooked and crimped steel fibers with volume fractions of 0.5%, 0.75% and 1% was used to produce UHPFRC. This mix is then used to produce the UHPC beams in full and partial depth fiber inclusion. The results in Table-3 show that the addition of steel fibers enhanced the compressive, splitting tensile and flexural (modulus of rupture) strengths of the control specimens. The compressive strength slightly increases with the increase of fiber volume fraction for both hooked and crimped steel fibers; the maximum percentage increase in compressive strength is about 9% for UHPC containing crimped steel fibers with volume fraction of 1%. This is due to the fact that the presence of steel fibers increases strainability in compression failure and hence the compressive strength increases [9]. The addition of steel fibers caused a significant increase in the splitting tensile

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and flexural strengths relative to plain UHSC (without fibers) and the increase in fiber content enhanced both splitting tensile and flexural strengths. The percentage increase in splitting tensile strength relative to plain UHSC mix was in the range of about 18% to 49% and 14 to 45% for mixes reinforced with hooked and crimped steel fibers respectively. This may be because that the addition of hooked and crimped steel fibers changes the brittle mode of failure for unreinforced specimens (without fibers) into a ductile one, which was observed to improve the energy absorption capacity of concrete. All mixes reinforced with hooked steel fibers have slightly higher splitting tensile and flexural strengths than those reinforced with crimped steel fibers, this may attribute to the excellent mechanical anchorage of hooked steel fibers at their ends which leads to high fiber matrix bond strength compared with those with crimped steel fibers [10, 11].

Table-2. Details of the tested beam specimens.

Series No.	**Beam symbol**	**Fiber shape**	**Volume fractions of fiber** (V_f) **%**	**Depth of fiber inclusion**
1	$D_{250}V_0$	-------	0	--------
2	$D_{250}V_{0.5h}$	Hooked	0.5	Full depth (250 mm)
	$D_{250}V_{0.5c}$	Crimped	0.5	Full depth (250 mm)
3	$D_{250}V_{0.75h}$	Hooked	0.75	Full depth (250 mm)
	$D_{250}V_{0.75c}$	Crimped	0.75	Full depth (250 mm)
4	$D_{250}V_{1h}$	Hooked	1	Full depth (250 mm)
	$D_{250}V_{1c}$	Crimped	1	Full depth (250 mm)
5	$D_{125}V_{0.5h}$	Hooked	0.5	Half depth (125 mm)
	$D_{125}V_{0.5c}$	Crimped	0.5	Half depth (125 mm)
6	$D_{125}V_{0.75h}$	Hooked	0.75	Half depth (125 mm)
	$D_{125}V_{0.75c}$	Crimped	0.75	Half depth (125 mm)

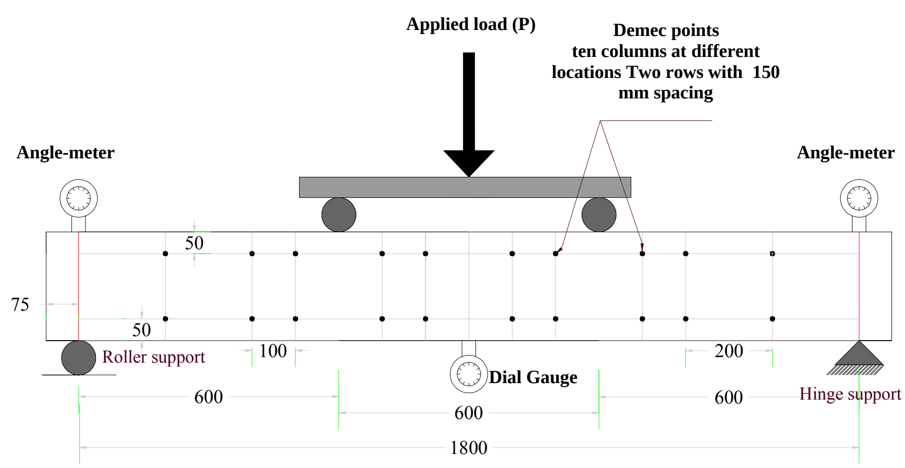

Figure-2. Schematic drawing showing the general flexural test setup for beam Specimens (Units: mm).

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Table-3. Compressive, splitting tensile, flexural strengths for control specimens.

Series No.	Beam symbol	Compressive strength (f_{cu}) (MPa)		Flexural strength (MPa)		Splitting tensile strength (MPa)	
		Plain concrete	Fibrous concrete	Plain concrete	Fibrous concrete	Plain concrete	Fibrous concrete
1	$D_{250}V_{0}$	137.4			7.41		6.4
2	$D_{250}V_{0.5h}$		139.6		8.93		7.45
	$D_{250}V_{0.5c}$		144.2		8.68		7.1
3	$D_{250}V_{0.75h}$		141.1		9.13		8.12
	$D_{250}V_{0.75c}$		142.8		8.89		7.99
4	$D_{250}V_{1h}$		146.4		9.4		9.39
	$D_{250}V_{1c}$		149.09		9.36		9.14
5	$D_{125}V_{0.5h}$	135.9	137.7	7.05	8.93	6.29	7.41
	$D_{125}V_{0.5c}$	138	142.3	7.37	8.68	6.16	7.27
6	$D_{125}V_{0.75h}$	132.3	140.4	7.69	9.13	6.22	8.23
	$D_{125}V_{0.75c}$	141.3	144.2	7.56	9.17	6.54	8.05

Behavior of ultra high performance steel fiber reinforced concrete beams in flexure

a) Load -deflection relationship

Figures 3 and 4 show the load-deflection relationships for concrete beam specimens containing different volume fraction of hooked and crimped steel fibers dispersed over full depth respectively. Generally it can be observed that fiber volume fraction has little effect on load-deflection relationship but there is a relatively stiffer response at the post cracking stage for all beam specimens containing steel fibers in comparison with reference concrete beam (without steel fiber). This may be due to the high specific strength of the steel fibers used in this investigation. It can also be seen that incorporation of steel fibers causes a desired modification of conventionally reinforced UHPC beams subjected to pure bending for almost of the entire range of loadings (first crack to ultimate loads). The ultimate flexural loads increase while the deflection at a given load decreases relative to the plain beam specimen (without fiber). These improvements may attribute to the high bond strength between the fibers and the matrix which is achieved with the densification of the matrix by silica fume combined with low water/cement ratio [12]. The behavior of fibers as crack bridging made fiber reinforced concrete capable to carry the load well after the development of cracks on the concrete [13].

Figures 5 and 6 show the load versus deflection curves for beam specimens reinforced with hooked and crimped steel fibers respectively with different volume fractions (0.5 % and 0.75 %) and dispersed over full and half depth of beam specimens. It can be noted that partial depth (half depth) fiber reinforced beams have about the same deformational characteristics as that for full depth fiber reinforced concrete beams even in some cases they behaved better. Thus by half depth fiber inclusion, an economical and efficient use of expensive steel fibers can be realized.

Figure-7 represents the load-deflection curves for beam specimens containing various types (hooked and crimped) and various volume fractions (0.5%, 0.75%, and 1%) of steel fiber, with full dispersion depth of fibers. It can be noted that crimped steel fibers showed slightly stiffer behavior than beams with hooked fibers, especially for fiber volume fraction 1 %. Figure-8 shows the load-deflection curves for beam specimens with various types (hooked and crimped) and volume fractions (0.5%, 0.75%) of steel fibers with half dispersion depth of fibers. It can be observed that the type and volume fractions of steel fibers have little effect on the load-deflection behavior.

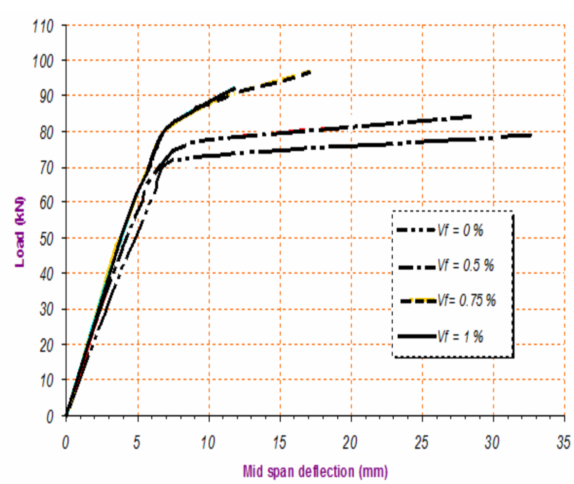

Figure-3. Load-deflection relationship at mid-span of full-depth hooked steel fiber reinforced UHSC beams.

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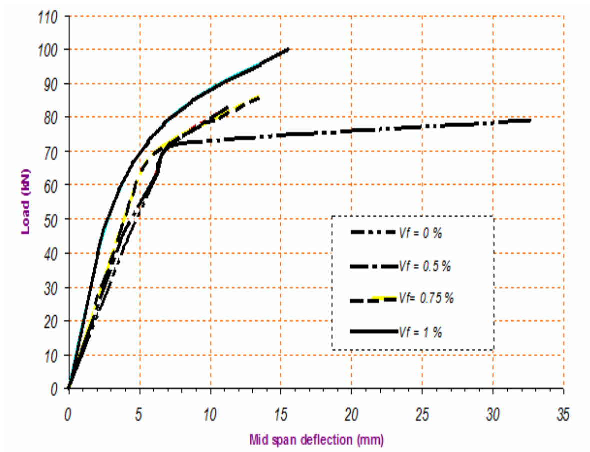

Figure-4. Load-deflection relationship at mid-span of full-depth crimped steel fiber reinforced UHSC beams.

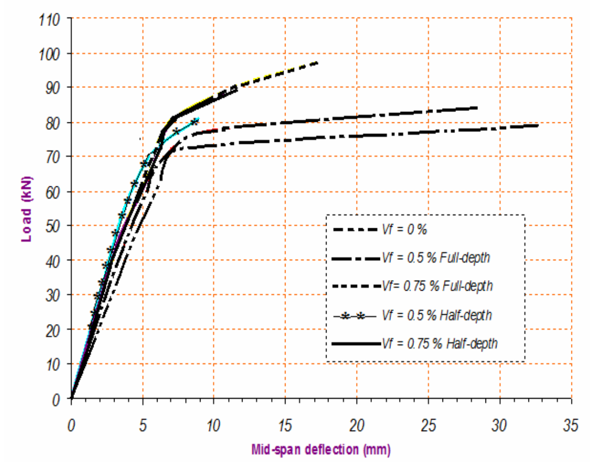

Figure-5. Load-deflection relationship at mid-span of UHSC beams with full and half dispersion depth of hooked steel fibers.

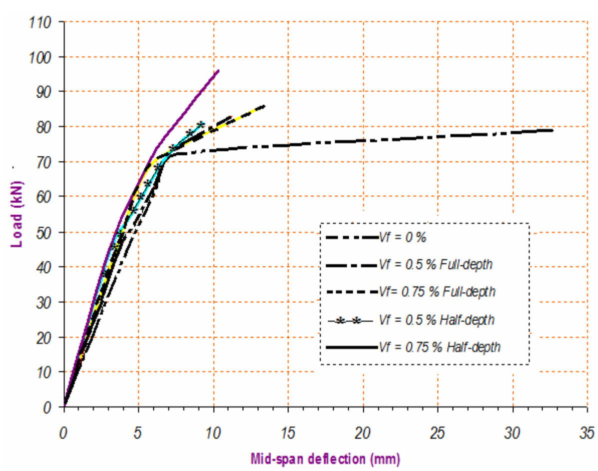

Figure-6. Load-deflection relationship at mid-span of UHSC beams with full and half dispersion depth of crimped steel fibers.

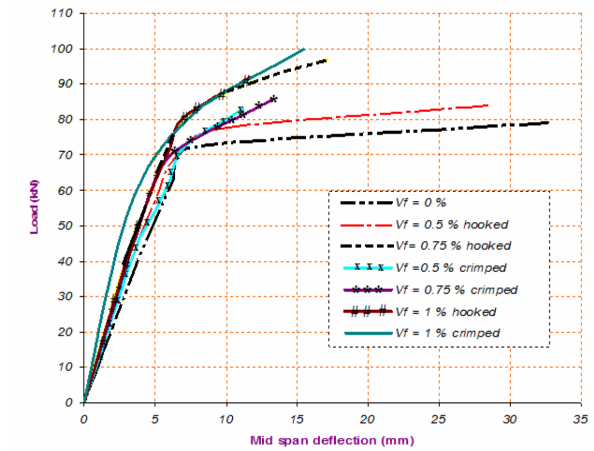

Figure-7. Load-deflection relationships at mid-span of UHSC beams with full depth dispersion of hooked and crimped steel fiber.

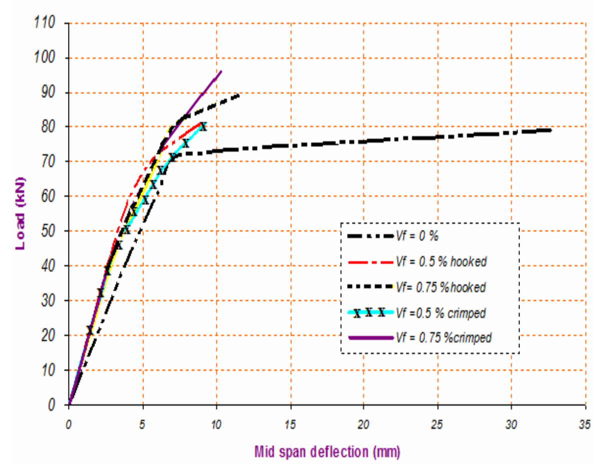

Figure-8. Load-deflection relationships at mid-span of UHSC beams with half depth dispersion of hooked and crimped steel fiber.

b) First crack and ultimate loads

The flexural test results of UHPFRC beams are shown in Table-4. Generally the results show that the addition of steel fibers has a little effect on the values of the first crack load except for beam specimens containing hooked steel fibers with volume fractions 0.75% and 1%. This may attribute to the fact that the addition of steel fibers has no important effect on the pre-cracking strength and even with relatively large volume fractions of high-modulus fibers, fully bonded and aligned in the most favorable direction; the cracking strength in the composite is not high compared with unreinforced matrix (without fibers) [9]. Table-4 also indicates that for beam specimens (reinforced with either hooked or crimped steel fibers) with full depth fiber dispersions, the ultimate load increases with the increase of fiber volume fraction in comparison with reference beam specimen (without steel fibers). The value of ultimate load increases by about 4%,

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9% and 27% for crimped steel fibers with volume fractions 0.5%, 0.75% and 1%, respectively in comparison with the reference specimen. There is a different effect of steel fiber type on the ultimate load of UHPC beam specimens; this effect depends on fiber volume fraction. The percentage increase in ultimate load is about 9 % and 17% for beam specimens containing crimped or hooked steel fibers with 0.75% volume fraction respectively, while the percentage increase is about 27% and 23% for beam specimens reinforced with crimped or hooked steel fibers with volume fractions 1% respectively relative to the reference beam specimen. This may be because at low fiber volume fractions (0.5% and 0.75%) the high mechanical anchorage of hooked steel fibers at their ends leads to high fiber-matrix bond strength compared with that reinforced with crimped steel fibers, while at high fiber volume fractions hooked steel fibers may reduce the efficiency of the compaction of the concrete mix more than that for crimped steel fibers [10, 11, 14]. The value of ultimate load for beam specimens reinforced either by hooked or crimped steel fibers with half depth fiber dispersion and different fiber volume contents are also shown in Table-4. It can be seen that the ultimate load increases with the increase in fiber volume content (0.5% and 0.75%), but the percentage increase in ultimate load is less than that for beam specimens with full depth fiber dispersion except for beam specimen containing crimped steel fiber with volume fraction 0.75% and half-depth fiber dispersion. This may be because the high contribution of fibers enhances matrix properties for specimens with full depth fibers dispersion relative to specimens with half depth dispersion.

c) Resilience

The total energy absorbed by a beam specimen under a flexural stress up to the first crack is called resilience; it is equal to the area under the elastic portion of the load-deflection curve [15]. From Table-4 and Figures 9 and 10, it can be seen that hooked steel fibers with volume fractions 0.75% and 1% improve the resilience of the beam specimens with full depth fiber inclusion. The percentage increase was about 36% and 35% for volume fractions 0.75% and 1% respectively relative to the reference beam (without fiber). Also the resilience of beam specimens with half depth fiber inclusion increases by using hooked steel fibers with volume fraction 0.75%. This may be due to the increase in the first crack load for those specimens relative to the reference beam specimen which is due to high mechanical anchorage of hooked steel fibers at their ends which leads to high fiber-matrix bond strength.

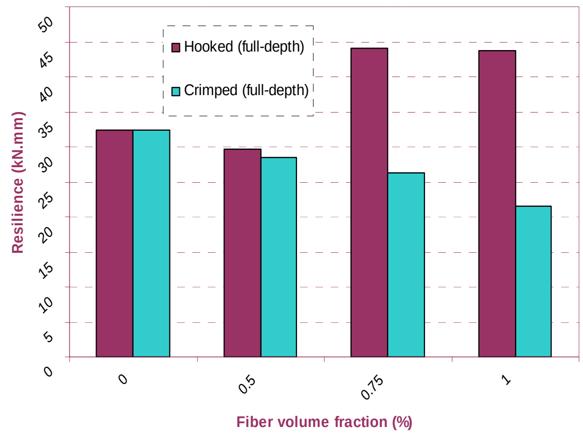

Figure-9. Resilience values for full depth fiber reinforced UHSC beam specimens.

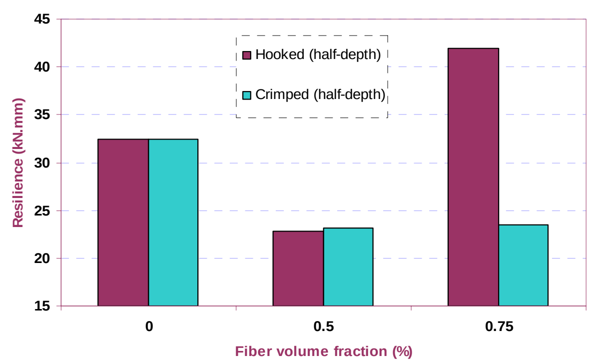

Figure-10. Resilience values for half depth fiber reinforced UHSC beam specimens.

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Table-4. First crack and ultimate loads, moment and resilience results for tested beam specimens.

Series No.	Beam symbol	Fiber content		Depth of fiber inclusion	Applied load		Applied moment		% increase in ultimate load	Resilience (kN.mm)
		Type	Volume fraction (%)		First crack load (kN) ($P_{cr.}$)	First crack moment (kN.M) ($M_{cr.}$)	Ultimate load (kN) ($P_{exp.}$)	Ultimate moment (kN.M) ($M_{exp.}$)		
1	$D_{250}V_{0}$	-------	0	--------	27	8.1	79	23.7	----	32.4
2	$D_{250}V_{0.5h}$	Hooked	0.5	Full depth	27	8.1	84	25.2	6.3	*29.6*
	$D_{250}V_{0.5c}$	Crimped	0.5	Full depth	27	8.1	82	24.6	3.8	28.4
3	$D_{250}V_{0.75h}$	Hooked	0.75	Full depth	36	10.8	92	27.6	16.5	44.1
	$D_{250}V_{0.75c}$	Crimped	0.75	Full depth	27	8.1	86	25.8	8.9	26.2
4	$D_{250}V_{1h}$	Hooked	1	Full depth	36	10.8	97	29.1	22.8	43.7
	$D_{250}V_{1c}$	Crimped	1	Full depth	27	8.1	100	30	26.6	21.6
5	$D_{125}V_{0.5h}$	Hooked	0.5	Half depth	27	8.1	81	24.3	2.5	22.8
	$D_{125}V_{0.5c}$	Crimped	0.5	Half depth	27	8.1	81	24.3	2.5	23.1
6	$D_{125}V_{0.75h}$	Hooked	0.75	Half depth	36	10.8	89	26.7	12.7	41.9
	$D_{125}V_{0.75c}$	Crimped	0.75	Half depth	27	8.1	96	28.8	21.5	23.5

e) Toughness indices results

Toughness index values were calculated by using ASTM $_{C1018}$ method, because of its wide acceptance in design and construction [16]. The areas under the load-deflection curve up to the first crack deflection and at selected multiples of first crack deflection of reinforced concrete beam specimens were calculated by Auto CAD software computer program. The values of toughness indices and residual strength factors for all tested specimens have been presented in Table-5. Generally, it can be noted that the values of indices I_5 and I_{10} are higher than the standard values 5 and 10, respectively. This indicates that up to 5.5 times of the cracking deflection, the post cracking performance of these specimens is better than the elastic-plastic behavior and the load carrying capacity is increased beyond that of matrix cracking, as indicated by the residual strength factor $R_{5,10}$ for these specimens which are higher than 100. Also it can be seen that for some specimens the value of I_5 are less than that for reference beam. This may be due to the results for first crack load; Table-5 shows that there is a slight increase in the value of toughness index I_{10} for most of the beam specimens relative to the reference beam specimen. It can be seen that to enhance the toughness of UHPC beams, higher fiber volume fraction must be used (more than 1% volume fraction). The ratio of I_5/I_{10} for the tested beam specimens was in the range of 2.54 to 3 as shown in Table-5. These values indicate that the behavioral pattern of these specimens approaches perfectly-plastic condition [17].

Concrete strains for beam specimens

Figures 11 to 14 show the strain diagrams for all beam specimens with volume fractions 0%, 0.5%, 0.75% and 1 % of hooked and crimped steel fibers at an applied load of 45 KN. Strains in concrete corresponding to each stage of loading was taken in both compression and tension zone, since the specimens flexural strains were readable only up to about 50% of the ultimate load so the strains at final stages of loading were not taken into consideration in this investigation. Generally the addition of steel fiber decreases both the compressive and tensile strains at the same value of applied load, a lesser effect of fiber incorporation was seen on compressive strain than tensile strain. This may be because that in compression when unreinforced cement based material is subjected to uniaxial compressive loading, internal micro-cracking parallel to the direction of load will occur for a certain stress level, no drastic change can therefore be expected in the course and character of the compression stress-strain curve by reinforcing these materials with fibers [18].

Failure mode and crack pattern for beam specimens

All beam specimens failed due to the yielding of the tensile reinforcement because of the minimum ratio of steel area used in the tensile zone. Generally, it can be seen from Figure-15, that the cracks at failure for fiber reinforced beams are greater in numbers and more distributed over the span of the beam specimen in comparison to reference beam (without fibers). This is due to the mode of action of fibers which increase the number of cracks and reduce the spacing between cracks at failure.

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Table-5. Toughness indices and residual strength factor for the tested beam specimens.

Series No.	Beam symbol	Toughness index		Residual strength factor $R_{5,10}$	I_{10}/I_5
		I_5	I_{10}		
1	$D_{250}V_0$	7.78	20.3	250	2.61
2	$D_{250}V_{0.5h}$	8.82	22.4	271	2.54
	$D_{250}V_{0.5c}$	7.18	21.4	285	2.98
3	$D_{250}V_{0.75h}$	7.86	19.9	240	2.53
	$D_{250}V_{0.75c}$	7.29	21.1	276	2.89
4	$D_{250}V_{1h}$	6.87	----------	----------	
	$D_{250}V_{1c}$	7.47	22.4	299	3.00
5	$D_{125}V_{0.5h}$	7.44	---------	---------	
	$D_{125}V_{0.5c}$	6.98	---------	---------	
6	$D_{125}V_{0.75h}$	7.18	---------	---------	
	$D_{125}V_{0.75c}$	7.44	21.6	283	2.90

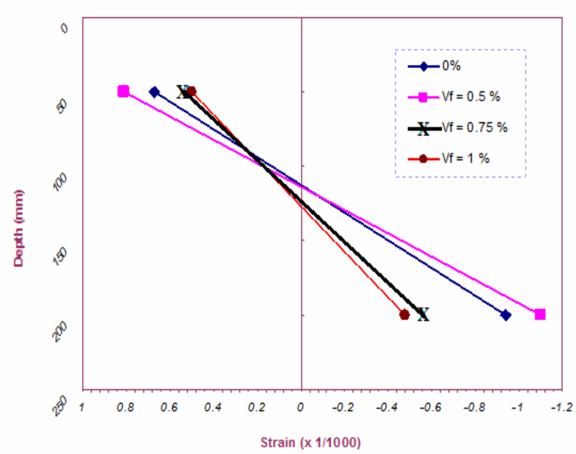

Figure-11. Strain diagram for full depth hooked steel fiber reinforced UHSC beam specimens at applied load (45 kN).

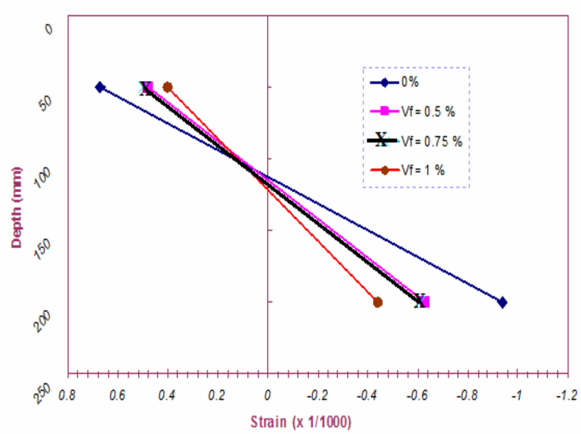

Figure-12. Strain diagram for full depth crimped steel fiber reinforced UHSC beam specimens at applied load (45 kN).

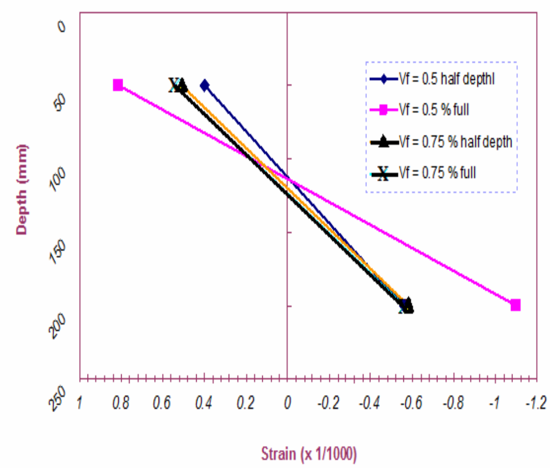

Figure-13. Strain diagram for full depth and half depth hooked steel fiber reinforced UHSC beam specimens at applied load (45 kN).

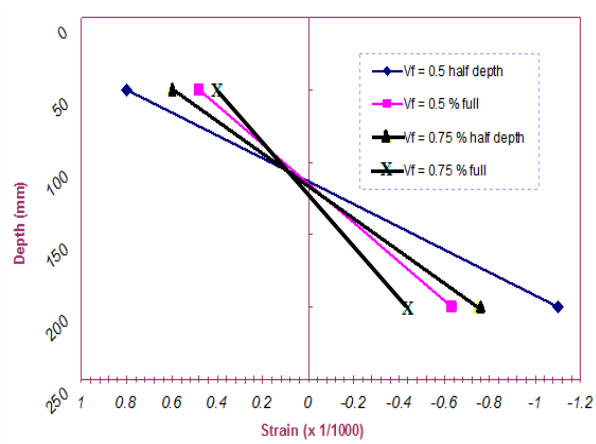

Figure-14. Strain diagram for full depth and half depth cpimped steel fiber reinforced UHSC beam specimens at applied load (45 kN).

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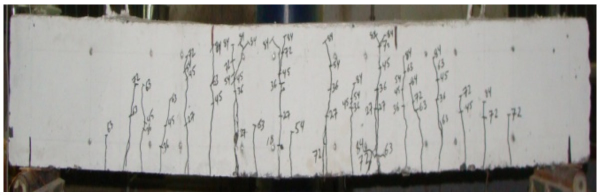
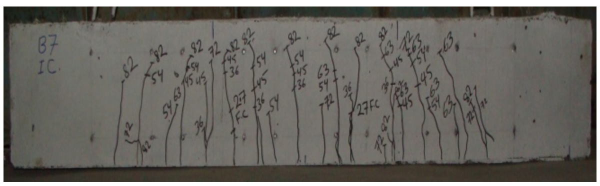
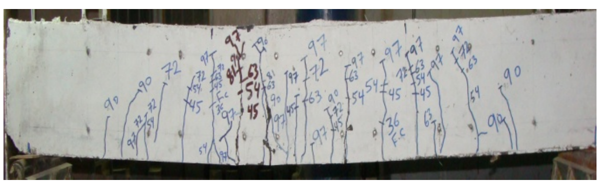
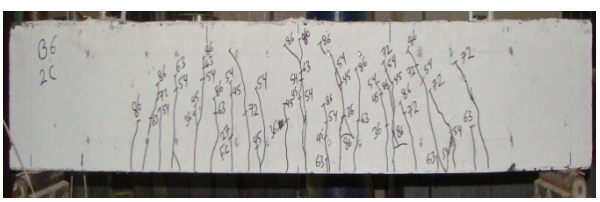
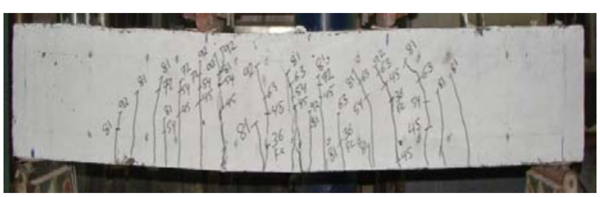
(a). Failure mode for beam ($D_{250}V_0$)


(b). Failure mode for beam ($D_{250}V_{0.5h}$)

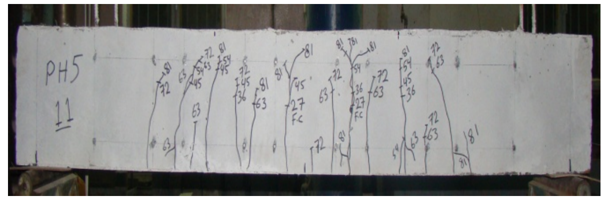

(c). Failure mode for beam ($D_{250}V_{0.5c}$)

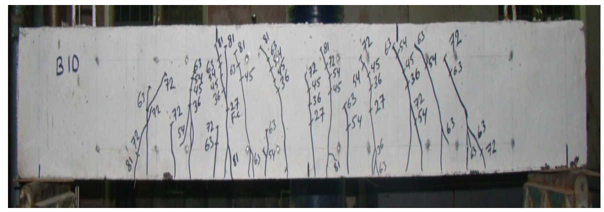

(d). Failure mode for beam ($D_{250}V_{0.75h}$)


(e). Failure mode for beam ($D_{250}V_{0.75c}$)


(f). Failure mode for beam ($D_{250}V_{1h}$)


(g). Failure mode for beam ($D_{250}V_{1c}$)


(h). Failure mode for beam ($D_{125}V_{0.5h}$)


(i). Failure mode for beam ($D_{125}V_{0.5c}$)


(j). Failure mode for beam ($D_{125}V_{0.75h}$)

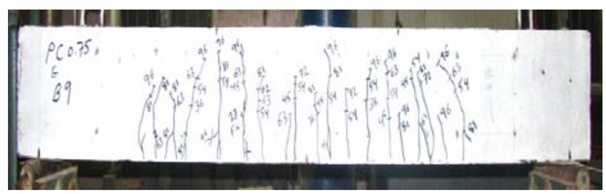

(k). Failure mode for beam ($D_{125}V_{0.75c}$)

Figure-15. Failure mode for the tested beam specimens.

THEORETICAL ANALYSIS

a) Flexural analysis of steel fiber reinforced concrete beams

Primarily, assuming that all fibers are aligned in the direction of the stress, before cracking the fibers are fully bonded to the matrix, equal strains in fiber and matrix occurs, and the Poisson's ratio in fiber and matrix is equal to zero [9], then:

$$\sigma_C = \sigma_f V_f + \sigma_m V_m \quad [9] \tag{1}$$

σ_C: *Ultimate tensile strength of steel fiber reinforced composite prior to cracking (MPa),*
σ_f : *Ultimate pull out strength of steel fiber (MPa)*
σ_m: *Ultimate tensile strength of uncracked concrete matrix (MPa),* V_f : *Volume fraction of fibers,* V_m : *Volume fraction of the matrix*

Since the orientation, length and bonding characteristics of fibers will influence the strength of fiber reinforced concrete; these parameters must be incorporated into equation (1).

$$\sigma_C = \sigma_m V_m + 2\, \eta_o \eta_b \eta_l\, V_f \tau_f\, (l_f / d_f) \tag{2}$$

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Based on the law of mixture, neglecting the contribution of the matrix in carrying any stress and applying the corrections of orientation, bond efficiency and length efficiency factor, the most common expression of ultimate strength of fiber reinforced concrete in use is given by:

$$\sigma_t = 2\ \eta_o \eta_b \eta_l\ V_f\ \tau_f\ (l_f / d_f) \quad [19] \qquad (3)$$

Where

σ_t: *Ultimate tensile strength of fibrous concrete (MPa),* η_o: *Orientation factor = 0.41 [20],* η_b: *Bond efficiency factor = 1.2 for hooked fibers [21],* η_l : *Length efficiency factor [20]*

$$\eta_l = 1 - [\tanh(\beta\, l_f/2) / (\beta\, l_f/2)],$$

$$\beta = \sqrt{\frac{2\pi G_m}{E_f A_f Ln(S/r_f)}}, \; S = 25\,(l_f / V_f\, d_f)^{1/2}$$

V_f: *Volume fraction of steel fiber,* τ_f: *Bond strength between the fiber and matrix (MPa),*
l_f: *Length of fibers (mm),*
d_f: *Diameter of fibers (mm),* G_m: *Shear modulus of concrete matrix (MPa),* E_f : *Modulus of elasticity of steel fibers (MPa),*
A_f: *Cross-sectional area of steel fibers (mm2),* r_f: *Radius of steel fibers (mm).*

Provided that the average sliding friction bond strength (τ_f) is known and assuming that it does not vary with the angle of the fiber to the crack and also assuming that the mean fiber pullout length is (*l/4*) then the average pullout stress per fiber (F) is given by :

$$F = \tau_f\,(l / d) \quad [9] \qquad (4)$$

Bond stress depends on a variety of factors such as water cement ratio, curing conditions, fiber surface characteristics, fiber geometry, and age, the measured value for τ_f varies between 3 and 8.3 N/mm^2 [9]. According to Chan *et al*. [22] who studied the effect of silica fume on the bond characteristics of steel fiber in matrix of UHSC, the average bond strength between the straight steel fiber and the surrounding matrix ranges between 4.8 and 5.5 MPa with 0 % to 40 % of silica fume respectively. According to Henager and Doherty [23] the frictional bond strength fiber matrix is given by:

$$\tau_f = 0.66\,\sqrt{fc'} \quad [23] \qquad (5)$$

fc' : *Compressive strength of normal strength concrete (MPa)*

By substituting the average compressive strength of UHSC produced in this investigation (fc' = 136 MPa) in equation (5), value of τ_f is equal to 7.7 MPa. Conservatively, taking the maximum value of τf equal to 7.7 MPa multiplying by the bond efficiency factor for hooked and crimped fibers equal to 1.2 and substituting in equation (4) Yields the value of pullout stress equal to 385 MPa and this value are much lower than the fiber's yield strength which is about 2000 MPa, so the certain composite failure is by pullout of steel fibers as it was seen by visual inspection during flexure tests. Consequently, substituting values 0.41, 1.2, 0.86 for orientation, bond, and length efficiency factor respectively in equation (2), the value of post cracking strength of fiber reinforced composite will be given by:

$$\sigma_t = 0.85\ V_f\ \tau_f\ (l_f / d_f) \qquad (6)$$

In this analysis, the maximum usable strain at the extreme concrete compression fiber is taken to be 0.0035. There are some data that indicate 0.003 may be conservative. Work by Williamson (1973) and Pearlman (1979) indicate that 0.0033 may be more realistic for steel fiber concrete. Swamy and Al-Ta'an (1981) recommend 0.0035. Based on a study of plastic hinges, Hassoun and Sahebjam (1985) recommend a failure strain of 0.0035 for concrete with 1.0 percent steel fibers [24].

Taking into account the large ductility and compressive strength of UHPFRC the equivalent compressive stress block values can be specified. For strengths above 30 MPa, β1 shall be reduced continuously at a rate of 0.05 for each 7 MPa of strength in excess of 30 MPa, but β1 shall not be taken less than 0.65 [25]. Figure-16 shows the stress-strain distribution and equivalent compressive stress block model for plain (without fiber) ultra high strength concrete, and Figure-17 shows the assumed and simplified stress distribution and strain diagram of singly reinforced ultra high strength concrete beams with half depth and full depth fiber inclusion. With the completion of compressive and tensile strength blocks, the process of flexural analysis of the beams can be carried out using the principles of force equilibrium and strain compatibility.

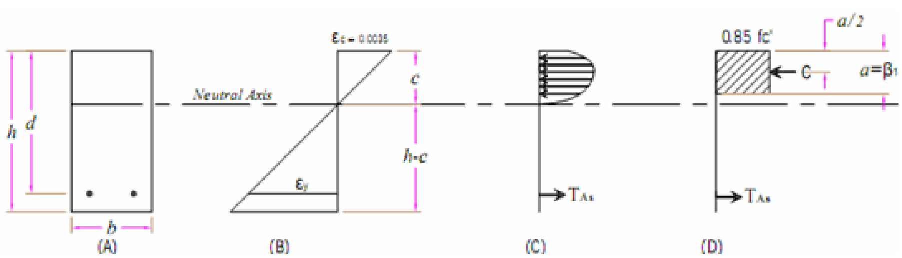

Figure-16. Strain and stress distribution for cross section of the singly reinforced plain concrete beam (ACI 318M - 011).

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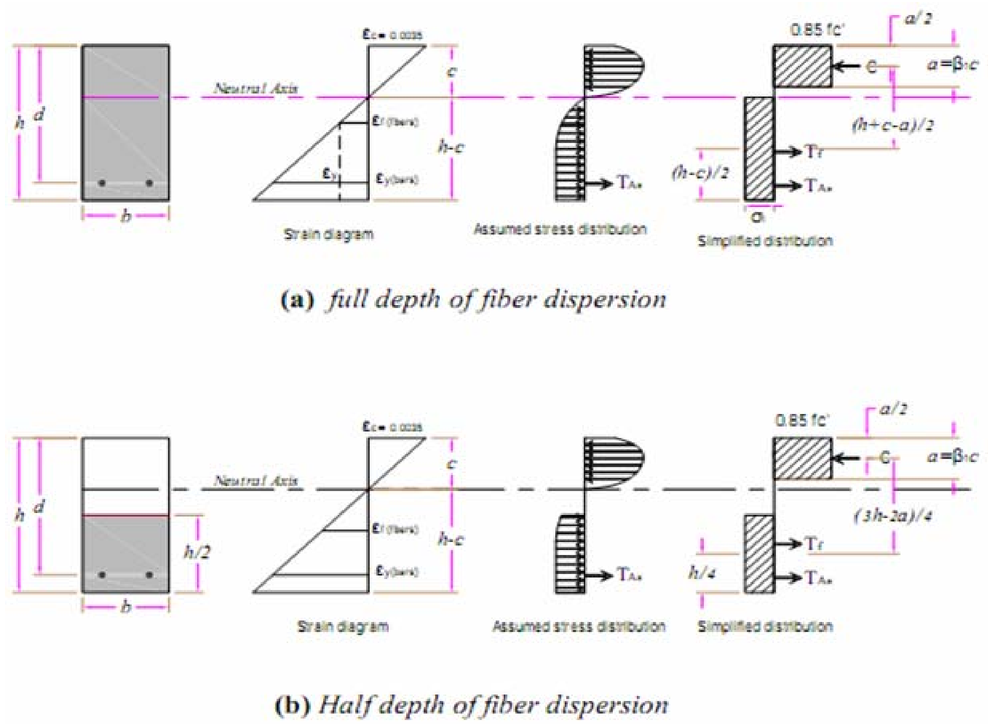

Figure-17. Design assumptions for analysis of singly reinforced concrete beams containing steel fibers (Hengar and Doherty, 1976 [23]).

b) Flexural analysis of singly reinforced plain UHPC beams

The equation for nominal moment of a singly reinforced UHPC beam is:

$$M_n = A_s f_y (d - a/2) \quad [25] \qquad (7)$$

$$a = A_s f_y / (\lambda f_c' b) \quad [25] \qquad (8)$$

A_s: Area of tensile steel bars (mm^2) f_y: Yield strength of tensile reinforcement bar (MPa), c: Neutral axis depth (mm)

b: Width of beam cross section (mm), h: Height of beam cross section (mm), λ: Concrete stress block parameter, (equal to 0.86 for f'c ≥ 55 MPa) [25] ,f'_c: Compressive strength of plain concrete, (MPa), β_1: Concrete stress block parameter, (equal to 0.65 for f'c ≥ 55 (MPa) [25],

a: Depth of the equivalent compressive block (mm).

c) Flexural analysis of full depth fiber reinforced UHPC beams

The value of nominal moment capacity for singly reinforced UHPFRC beams shown in Figure-17 is given by:

$$M_n = A_s f_y (d - a/2) + \sigma_t b (h - c) (h + c - a)/2 \quad [20] \qquad (9)$$

$$a = (A_s f_y + \sigma_t b h) / (\lambda f_c{}'_f b + \sigma_t b) \quad [20] \qquad (10)$$

Where

f'_{cf} = Compressive strength of fibrous concrete (MPa)

d) Flexural analysis of half depth fiber reinforced UHPC beams

The ultimate moment capacity of UHPC beams with half depth of fiber dispersion can be derived as shown in Figure-17:

$$M_n = A_s f_y \ (d - a/2) + \sigma_t b\, h\, (3h - 2c)/8 \qquad (11)$$

$$a = (A_s f_y + \sigma_t b\, h/2) / \lambda f_c' b \qquad (12)$$

As it was reported by ACI_{544} [24], a method has been developed by Henager and Doherty (1976) [23] for predicting the strength of beams reinforced with both conventional steel bars and fibers for normal strength concrete. This method is similar to ACI ultimate strength design method. The tensile strength computed for the fibrous concrete is added to that contributed by the reinforcing bars to obtain the ultimate moment. The basic design assumptions made by Henager and Doherty (1976), and the equation for nominal moment M_n of a singly reinforced steel fibrous concrete beam is:

$$M_n = A_s f_y (d - a/2) + \sigma_t b (h - e) (h + e - a)/2 \quad [26] \qquad (13)$$

$$e = (\varepsilon_f + 0.0035)\, C / 0.0035$$

Where

$$\sigma_t = 0.772\, V_f\, (l_f / d_f)\, F_{be} \quad [26] \qquad (14)$$

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F_{be}: *Bond efficiency factor = 1.2 for hooked and crimped steel fibers,* e: *The distance from the extreme compression fiber to the top of the tensile stress block of fibrous concrete,* ε_f: *Fibers tensile strain corresponds to the fiber stress at pullout* σ_f

In equation (14) the coefficient 0.772 which developed for normal strength concrete incorporates a factor for a typical bond strength of fiber's τ_f that was taken as 2.3 MPa for normal strength concrete, in order to account for ultra high strength concrete τ_f is taken as 5 MPa, so this coefficient was modified and taken as 2 [26].

e) Theoretical results and discussions

Table-6 shows the results of flexural analysis of UHPC beams reinforced with both conventional steel bars and different types of fibers (hooked and crimped) with different volume fractions (0.5%, 0.75%, and 1%) and with half-depth and full depth of fiber dispersion. It can be seen from the analytical and the experimental results in Table-6 that there is a small deviation of analytical results relative to the experimental results. The ratio between the calculated moments to the experimental moments varies from 0.91 for plain concrete beam to 1.16 for the fibrous beam with 0.75% of full depth crimped steel fiber dispersion. Generally a slight overestimation for all fibrous beams can be seen except for beam $D_{125}V_{0.75c}$. Comparison between the results of derived equations and ACI_{544} equation was done in predicting the moment capacities of beam specimens, it has been observed that the ratio of calculated to experimental moments changes between 0.91 to 1.16 when applying equations (7), (9) and (11), while the ratio varies from 0.93 to 1.02 for the results taken from ACI_{544} equation. It can be concluded that the results obtained from both models are quite accurate but the ACI_{544} model showed better estimation for ultimate moment capacity.

Table-6. Measured and calculated moment capacity for beam specimens.

Series No.	Beam symbol	Fiber type	Volume fraction (%)	Depth of fiber inclusion	Experimental failure moment (kN.m)	Calculated moment (kN.m)	Calculated moment according to ACI 544 (kN.m)	Ratio Cal./Exp. Moments	Ratio Cal. ACI 544/Exp. Moments
1	$D_{250}V_0$	-------	0	--------	23.7	21.57	21.6	0.91	0.91
2	$D_{250}V_{0.5h}$	Hooked	0.5	Full depth	25.2	27.24	24.8	1.08	0.98
	$D_{250}V_{0.5c}$	Crimped	0.5	Full depth	24.9	27.24	24.8	1.09	0.99
3	$D_{250}V_{0.75h}$	Hooked	0.75	Full depth	27.6	30.03	26.3	1.09	0.95
	$D_{250}V_{0.75c}$	Crimped	0.75	Full depth	25.8	30.03	26.3	1.16	1.02
4	$D_{250}V_{1h}$	Hooked	1	Full depth	29.1	32.78	27.9	1.13	0.96
	$D_{250}V_{1c}$	Crimped	1	Full depth	30	32.78	27.9	1.09	0.93
5	$D_{125}V_{0.5h}$	Hooked	0.5	Half depth	24.3	25.96	-----	1.07	-----
	$D_{125}V_{0.5c}$	Crimped	0.5	Half depth	24.6	25.96	-----	1.06	-----
6	$D_{125}V_{0.75h}$	Hooked	0.75	Half depth	26.7	28.14	-----	1.05	-----
	$D_{125}V_{0.75c}$	Crimped	0.75	Half depth	28.8	28.14	-----	0.98	-----

CONCLUSIONS

From the experimental results and theoretical analysis presented in this investigation, the following conclusions can be drawn:

a) The addition of steel fibers has no significant effect on the values of the first crack load except for beam specimens containing hooked steel fibers with volume fractions 0.75% and 1%.
b) The ultimate moment capacity for the beam specimens is considerably improved with the addition of steel fibers, the maximum percentage increase relative to reference beam is about 27% and 23% for beam specimens containing 1% volume fraction of crimped and hooked steel fibers, respectively.
c) The ultimate moment capacity for most of partially fiber reinforced beam specimens (half-depth fiber dispersion) is slightly lower than those of the

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corresponding full depth fiber reinforced concrete beams.

d) The resilience of full depth fiber reinforced specimens containing hooked steel fibers with volume fractions 0.75% and 1% is considerably improved, the percentage increase relative to the reference beam is about 36% and 35%, respectively.

e) The toughness indices I_5 and I_{10} for all beam specimens are higher than the standard values 5 and 10, respectively. The values of toughness index I_{10} for full depth fibers reinforced UHPC beam specimens are slightly increased relative to reference beam (without fibers).

f) Using the conventional efficiency factors and most common equations in predicting the ultimate strength of fiber-matrix composite in ultra high strength fibrous concrete slightly overestimates the ultimate moment capacity of beam specimens, the average ratio of calculated to measured moment capacity are 1.1 and 1.04 for beams with full and half depth fiber inclusion, respectively.

g) Applying ACI_{544} model, which used only for normal strength concrete reinforced with steel fibers with aspect ratio less than 100, shows good agreement with the measured ultimate moment capacity. The average ratio of calculated to measured ultimate moment capacity is 0.97.

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