International Journal of Scientific & Engineering Research Volume 8, Issue 12, December-2017
ISSN 2229-5518

Shear Behavior Of Ultra–High Performance Fiber Reinforced Concrete Beams With Minimum Web Reinforcement

Ahmed Yousef, Ahmed Tahwia, Nagat Mareamy

Abstract—An experimental and analytical investigation to study the shear behavior of Ultra-High Performance Fiber Reinforced Concrete (UHPFRC) beams with minimum web reinforcement has been conducted. Five simply supported beams with a characteristic concrete cube strength approaching 188 MPa have been tested under two-points symmetric top loading. The main studied parameters are the shear-span-to depth ratio (a/d) and the minimum vertical web reinforcement ratio in the form of the spacing between stirrups and the stirrup bar diameter. The steel fibers volume fraction of 1.5% is kept constant for all the tested beams. The results of these tests have been used to examine the applicability of the shear design equations of ACI 318-2014 building code, Eurocode 2 (EC-2) and the Egyptian code (ECP-203-2017) when applied to UHPFRC beams. The design recommendations proposed by the France Association of Civil Engineers (AFGC-2002) and also the design recommendations of the Korea Concrete Institute (KCI-2012) for UHPFRC beams have been evaluated. An analytical model for predicting the shear strength and deformations of the studied UHPFRC is proposed using three dimensional finite element program. The results of this study showed that the maximum spacing between stirrups required by the ECP-203-2017 is applicable for UHPFRC beams, while the maximum spacing between stirrups required by ACI 318-2014 is not practically suitable for beams with relatively small height and can be safely increased from *0.5d* to be *0.75d*. The equations of the shear strength of the reinforced concrete beams used by the studied international codes highly underestimate the shear strength of the tested UHPFRC beams. The minimum recommendations of AFGC-2002 and also KCI-2012 are safe and conservative for design of UHPFRC beams provided with shear reinforcement less than that required by the studied codes. The predictions of the ultimate shear strength of the tested beams using AFGC-2002 are approximately similar to that of KCI-2012. The steel fibers volume fraction of 1.5% of the tested beams contributes by about 60% in the ultimate shear strength of the tested beams while the contribution of the concrete and the vertical web reinforcement is about 40%. Including the effect of steel fibers in the finite element model showed good predictions for the ultimate shear strength and the deformation response of the studied UHPFRC beams.

Index Terms— Beams, Codes, Finite Element Model, Minimum Web Reinforcement, Shear Strength, Shear Span-to-Depth Ratio, Ultra-High Performance Fiber Reinforced Concrete (UHPFRC).

1 Introduction

Ultra High-Performance Fiber Reinforced Concrete (UHPFRC) is an emerging material that has been developed in the beginning of 1990 by France's research group. Because of its excellent mechanical behavior, in the form of high compressive strength of values greater than 150 MPa and a design value of tensile strength more than 8 MPa, durability, energy absorption capacity and fatigue performance. Its very high strength properties results in a great reduction in the structural weight and consequently UHPFRC can be used in wide range of applications such as long span bridges and high rise structures [1], [2], [3], [4].

Numerous researches on UHPFRC have been carried out in many countries all over the world [5], [6], [7]. In addition, experimental tests on the shear behavior of UHPFRC beams with and without web reinforcement have been reported [8], [9], [10]. All the international building codes such as ACI 318-2014 Building Code [11], the Eurocode 2 (EC-2) [12] and the Egyptian code (ECP-203-2017) [13] do not contain any provisions for design of UHPFRC beams. In order to take into account the contribution of the steel fibers, the first design recommendations for UHPFRC structures has been reported in 2002 [14] by the France Association of Civil Engineers (AFGC-2002) [14]. Another design recommendations for UHPFRC has been proposed by the Japan Society of Civil Engineers (JSCE-2008) [15]. Recently, the Korea Concrete Institute also developed design recommendations for UHPFRC structures (KCI-2012) [16].

In this paper, an experimental and analytical study to investigate the shear behavior of UHPFRC beams with minimum shear reinforcement has been reported. The results of these tests have been used to examine the applicability of the shear design provisions of ACI 318-2014 building code, EC-2 and ECP-203-2017 when applied to UHPFRC beams. The recommendations of AFGC-2002 and also that of KCI-2012 for the shear design of UHPFRC beams have been also evaluated. An analytical model for predicting the shear strength and deformations of UHPFRC beams is proposed using a three dimensional finite element program.

- *Prof. Dr. Ahmed M. Yousef is currently professor of concrete structures and the head of Structural Engineering Department, Faculty of Engineeing Mansoura University, Egypt, E-mail: ahmyousef@mans.edu.eg*
- *Prof. Dr. Ahmed M. Tahwia is currently professor of strength of Materials, Mansoura University, Egypt, E-mail: atahwia@mans.edu.eg*
- *Eng. Nagat A. Mareamy is currently pursuing PhD degree in Structural Engineering in the Faculty of Engineering, Mansoura University, Egypt, E-mail: najatali123@yahoo.com*

2 Recommendations for Shear Design of UHPFRC Beams

2.1 KCI-2012

The design shear strength V_d of UHPFRC is obtained as follows:

$$V_d = V_c + V_{fb} + V_s \tag{1}$$

Where V_c, V_{fb}, and V_s are the shear strength provided by the contribution of cement matrix, steel fiber and shear reinforcement, respectively. The shear strength provided by the cement matrix is given as follows:

$$V_c = \varnothing_b\, 0.18 \sqrt{f_c^{'}}\, b_w\, d \tag{2}$$

Where ϕ_b is the member reduction factor equal to 0.77, $f_c^{'}$ is the compressive cylinder strength, b_w is the beam width and d is the effective depth of the beam. The shear strength of the steel fibers V_{fb} is given as follows:

$$V_{fb} = \varnothing_b\ (f_{vd} / \tan\beta_u)\, b_w\, z \tag{3}$$

Where f_{vd} is the design average tensile strength in the direction perpendicular to diagonal tensile crack, β_u is the angle between the diagonal tensile crack plane and axial direction of the beam and shall be larger than 30^o, z is the distance from the position of the resultant of the compressive stresses to the centroid of tensile steel, generally equal to $d/1.15$. The value of f_{vd} is calculated as follows:

$$f_{vd} = 1/w_v \int_0^{w_v} \phi_c\, \sigma_k\ (w)\, dw = 1/w_v \int_0^{w_v} \sigma_d\ (w)\, dw \tag{4}$$

Where w_v = max (w_v , 0.3 mm), w_u is the ultimate crack width at the peak stress on the outer fiber, ϕ_c is the material reduction factor taken equal to 0.8, $\sigma_k\,(w)$ is the tension softening curve and $\sigma_d\,(w)$ is equal to $\phi_c\ \sigma_k\,(w)$.

The shear strength provided by the shear reinforcement V_s is given as follows:

$$V_s = \varnothing_b\ [\, A_v\ f_{yv}\ (\sin\alpha_s + \cos\alpha_s) / s_v]\, d \tag{5}$$

Where A_v is the cross sectional area of shear reinforcement, f_{yv} is the design yield strength of shear reinforcement, α_s is the angle between longitudinal axis of beam and shear reinforcement and s_v is the spacing of shear reinforcement.

2.2 AFGC-2002

Equation (1) shows the design shear strength V_d which given by the same (1), while the term of shear strength provided by the cement matrix V_c is given as follows:

$$V_c = (0.21 / \gamma_{cf}\ \gamma_E)\, k\, \sqrt{f_c^{'}}\ b_w\ d \tag{6}$$

Where γ_{cf} is the partial safety factor on fibers and is assumed to be a value of *1.30*, γ_E is a safety coefficient, $\gamma_{cf}\ \gamma_E$ is equal to 1.5, and k is a factor for the case of prestressing. The contribution of steel fibers V_{fb} can be calculated as follows:

$$V_{fb} = (A_{fv}\ \sigma_{Rd,f} / \tan\theta\ ) \tag{7}$$

Where A_{fv} is the area of fiber effect and is assumed to be $b_w\ z$ for rectangular sections, z is equal to $0.9d$, θ is the angle between the principal compression stress and the beam axis and can be taken with a minimum value of 30^o, and $\sigma_{Rd,f}$ is the residual tensile strength which can be calculated as follows:

$$\sigma_{Rd,f} = (1 / k\ \sigma_{cf})\ (1 / w_{lim}) \int_0^{w_{lim}} \sigma_f\ (w)\, dw \tag{8}$$

Where w_{lim} = max (w_u , w_{max}), K is the fiber orientation factor and can be taken equal to *1.25*, $\sigma_f\,(w)$ is a function of the tensile stress and crack width, w_{max} is the maximum crack width.

The shear strength by the vertical shear reinforcement is computed as follows:

$$V_s = (A_v\ / s_v\ )\ z\ f_{yv} \cot\theta \tag{9}$$

3 EXPERIMENTAL PROGRAM

3.1 Details of Test Specimens

The details of the test program are given in Table 1. The test specimens included five simply supported UHPFRC beams from the same concrete mix tested under two points loads. All the tested beams have constant rectangular cross-section of total height 210 mm and width 120 mm as shown in Fig. 1. The effective span of the beams l_e is equal to 1150 mm and the distance c between the two loads has been varied to achieve the desired a/d ratio. In order to ensure shear failure of the tested beams, the main longitudinal tensile reinforcement of the beams consists of 6 deformed bars with diameter 18 mm placed in two layers. The yield strength of these bars is equal to 491.2 MPa. The upper longitudinal reinforcement of the beams consisted of two 12 mm diameter deformed bars with yield strength equal to 480.1 MPa. The yield strengths f_{yv} of the stirrups bar diameters 6 mm and 8 mm are equal to 336.2 MPa and 308.5 MPa, respectively.

In order to examine the applicability of the minimum vertical web reinforcement required by the studied codes, the provided vertical web reinforcement of the tested beams is varied in the tested beams by using different spacing between stirrups (s_v = 200 mm and 100 mm) and different stirrups bar diameters (d_v = 6 mm and 8 mm). Table 2 compares the provided vertical web reinforcement ratio ($\rho_v = A_v\ /\ b_w\ s_v$) of the tested UHPFRC beams with the minimum requirements of the codes $\rho_{v,min}$ and also compares the provided spacing between stirrups s_v with the maximum allowable spacing between stirrups $s_{v,max}$ required by the studied codes. According to ECP-203-2017, $\rho_{v,min}$ is equal to the greater of 0.15% or $0.4/f_y$ and the minimum bar diameter $d_{v,min}$ is equal to 6 mm, while $s_{v,max}$ is equal to 200 mm. According to ACI 318-2014, $\rho_{v,min}$ is equal to the greater of ($0.062\sqrt{f_c^{'}}\ /\ f_{yv}$) or $0.35/f_y$ and $s_{v,max}$ is equal to the least of $0.5d$ or 600 mm, while for the case of shear strength contributed by shear reinforcement V_s exceeds ($0.33\sqrt{f_c^{'}}\ b_w\ d$), $s_{v,max}$ should be reduced by one-half. According to EC-2, $\rho_{v,min}$ is equal to ($0.08\sqrt{f_c^{'}}\ /\ f_{yv}$) and $s_{v,max}$ is equal to the least of 0.75d or 600 mm. From Table 2, it can be seen that the tested beams BSU1 and BSU2 provided with stirrups with diameter 6 mm which satisfies $d_{v,min}$ required by ECP-203-2017, while beams BSU2, BSU4 and BSU5 with stirrups spacing equal to 200 mm which satisfies $s_{v,max}$. The provided ρ_v of the tested beam BSU2 is equal to 0.12 which is less than $\rho_{v,min}$ required by ECP-203-2017. The provided s_v for all the tested beams is greatly more than $s_{v,max}$ required by ACI 318-2014 and the provided ρ_v of the tested beams BSU2, BSU4 and BSU5 is less than $\rho_{v,min}$ required by this code, while ρ_v for beam BSU1 is equal to $\rho_{v,min}$. For EC-2, the provided s_v for beams BSU2, BSU4 and BSU5 is more than $s_{v,max}$ required by this code, while the provided ρ_v of beams BSU1, BSU2, BSU4 and BSU5 is less than $\rho_{v,min}$.

3.2 Mix Design of UHPFRC and Test Setup

In the Ultra High-Performance Fiber Reinforced Concrete (UHPFRC) mix design, ordinary Portland cement conforming with the requirements of the Egyptian standards with specific gravity of 3.16 and silica sand with a grain size smaller than 2.5 mm was adopted as the coarse aggregate. The detailed mixture proportions for one cubic meter are summarized in Table 3. The steel fibers used in the present study were hooked-ended straight fiber of a type available in the Egyptian market. The steel fiber length and equivalent diameter of the fibers are equal to 25.0 mm and 1.0 mm, respectively, and thus the aspect ratio is equal to 25. The tensile strengths of the fiber has been supplemented by direct tension test, and these fibers have yield strength and tensile strength equal to 552.2 MPa and 828.3 MPa, respectively. The amount of steel fibers volume fraction used for all the tested beams has been kept constant of about 1.5%. The cube concrete compressive strength f_{cu} based on an average of three cube specimens (cube 50 mm x 50 mm x 50 mm) and the cylinder concrete compressive strength $f_c^{'}$ based on (cylinder 50 mm x 100 mm) of the UHPFRC mix are 188.1 MPa and 172.9, respectively, while the splitting cylinder tensile strength f_{sp} and the flexural strength f_r (based on 40 mm x 40 mm x 60 mm prisms) are 11.9 MPa and 39.7 MPa, respectively.

The beams were simply supported and tested in a loading frame with a capacity of 200 ton under two-point loading as shown in Fig. 2. Dial gauges were mounted at the bottom face of the beams at mid-span and under the loading points. Two days before testing the beams were allowed to dry and painted with white color to facilitate crack detection. Each beam specimen was instrumented with electrical strain gauges on the main longitudinal reinforcing bars at mid-span and on the vertical web reinforcement in the shear zone. Load was applied in small increments and all deformation readings were recorded at the end of each load increment. The initiation and propagation of cracks were marked and the widths of cracks within the shear span zone were recorded.

4 EXPERIMENTAL RESULTS

4.1 Cracking behavior and Modes of Failure

All the tested UHPFRC beams failed in shear mode, except the tested beam BSU3 which failed in the compression zone at the mid span before reaching the shear failure. Photographs of of the tested beams showing typical observed cracking patterns and failure mode are given in Fig. 3. The numbers written along the cracks on the photographs indicate the termination of cracks observed at the end of a particular load stage, where loads indicated on the photographs are the total applied load $2V$ in tons. Table 4 presents the measured diagonal cracking strength (V_{cr}) and the ultimate strength ($V_{u,exp}$) of the tested beams in this program. Diagonal cracks usually occurred in both shear spans of the beam after the formation of flexural cracks in the mid-span region. After the development of diagonal shear cracks, the width of the flexural crack becomes very limited. The diagonal shear crack usually originated suddenly in the middle of the shear span and propagated toward the support and loading point from a subsequent increase of applied load. With a further increase of the applied load, the existing diagonal shear cracks propagated very slowly while a few numbers of new inclined cracks were formed. Finally, shear failure occurred suddenly by fracture of the concrete along the inclined crack. As shown from the photographs, the span of the tested beams BSU1, BSU2, BSU4, and BSU5 collapsed due to excessive destruction of concrete in the shear span, while beam BSU3 collapsed due to crushing of the compression zone at the mid BSU4 with *a/d* equal to 2.75.

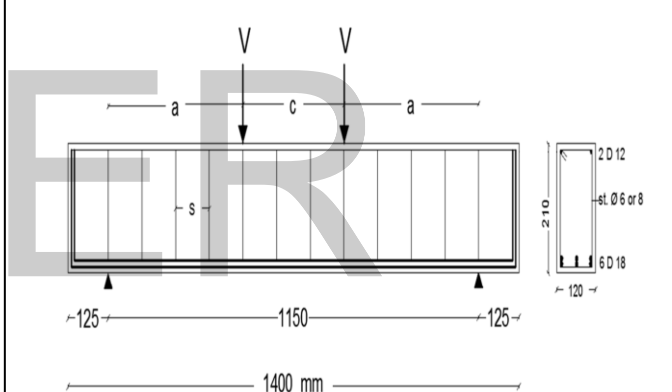

Fig. 1 Details and reinforcement of the tested beams.

Fig. 2 Test setup of the tested beams.

TABLE 1
DETAILS OF TEST PROGRAM

Beam	f_{cu} MPa	f_c' MPa	$b \times h$ mm	a/d ratio	Main longitudinal bars lower	Main longitudinal bars upper	Stirrups (vertical web reinforcement) s_v mm	d_v mm	ρ_v (%)
BSU1	188.1	172.9	120×210	2.75	6D18	2D12	100	6	0.24
BSU2	188.1	172.9	120×210	2.75	6D18	2D12	200	6	0.12
BSU3	188.1	172.9	120×210	2.75	6D18	2D12	100	8	0.42
BSU4	188.1	172.9	120×210	2.75	6D18	2D12	200	8	0.21
BSU5	188.1	172.9	120×210	3.00	6D18	2D12	200	8	0.21

TABLE 2
COMPARISON BETWEEN THE PROVIDED STIRRUPS AND THE MINIMUM SHEAR REINFORCEMENT REQUIRED BY THE STUDIED CODES

Beam	a/d Ratio	Provided vertical web reinforcement (stirrups)			Requirements of the international codes for $s_{v,max}$ and $\rho_{v,min}$					
					ACI 318-2014		EC-2		ECP-203-2017	
		s_v mm	d_v mm	ρ_v %	$s_{v,max}$ mm	$\rho_{v,min}$ (%)	$s_{v,max}$ mm	$\rho_{v,min}$ (%)	$s_{v,max}$ mm	$\rho_{v,min}$ (%)
BSU1	2.75	100	6	0.24	85	0.24	125	0.31	200	0.15
BSU2	2.75	200	6	0.12	85	0.24	125	0.31	200	0.15
BSU3	2.75	100	8	0.42	85	0.26	125	0.34	200	0.15
BSU4	2.75	200	8	0.21	85	0.26	125	0.34	200	0.15
BSU5	3.00	200	8	0.21	85	0.26	125	0.34	200	0.15

TABLE 3
UHPFRC MIX PROPORTIONS FOR ONE CUBIC METER

Water kg	Cement kg	Silica Fume kg	Silica Sand kg	Quartize kg	Superplasticize kg
162	900	225	774	270	36

Records of the longitudinal steel strains at the mid-span of the tested beams showed that the tensile strains in the region of maximum bending moment at the mid span are almost uniform at every load level. Failure of all the tested beams occurred before yielding of the longitudinal bars. Tensile steel strains increase approximately at a constant rate. Formation of inclined diagonal shear cracks has no effect on the strain readings of the longitudinal bars. For the same a/d ratio, the strain readings in the longitudinal bars of the tested UHPFRC beams are approximately similar. Records of the strains in the vertical legs of the stirrups in the shear span of the tested beams are compared in Fig. 6. It can be seen that the recorded tensile steel strain in the stirrup leg is very small at the first stage of loading and increase approximately at a constant rate. When the diagonal shear crack is initiated, the strain rate increases more fast until reaching the yield value just before the applied ultimate load. After yielding of the stirrup leg, the recorded strains increase with very fast rate with decreasing the applied load and very fast increase of the shear crack width until crushing of the concrete in the shear zone. For the same *a/d* ratio, increasing the stirrup diameter and reducing the spacing between stirrups reduce the recorded strains of the tested beams at the same load.

TABLE 4
SUMMARY OF TEST RESULTS

Beam	a/d	$2V_{cr}$ (kN)	$2V_{u,exp}$ (kN)	V_{cr} (kN)	$V_{u,exp}$ (kN)	$V_{cr}/V_{u,exp}$	$\frac{V_{u,exp}}{b.d\sqrt{f_c'}}$
BSU1	2.75	175	450	87.5	225.0	0.389	0.891
BSU2	2.75	150	405	75.0	202.5	0.370	0.790
BSU3	2.75	190	505	95.0	252.5	0.376	0.998
BSU4	2.75	160	430	80.0	215.0	0.372	0.852
BSU5	3.00	150	380	75.0	190.0	0.395	0.753

4.2 Load-Displacement Relationships and Strain Response

The total applied load ($2V$) versus the mid-span deflection curves for the tested UHPFRC beams with different *a/d* ratio and different percentages of ρ_v are shown in Fig. 5. In early stages of loading, the beams behaved in a truly elastic manner. In general, changing the value of ρ_v% of the tested beams with the same *a/d* ratio did not have considerable effect on the ascending part of the load deflection curves. After reaching the ultimate load there was minor differences in deflection magnitudes of the beams of the same *a/d* ratio and different ρ_v%. As can be seen from Fig. 5, beam BSU2 with *a/d* equal to 2.75 and with provided ρ_v equal to 0.12, which is less than the minimum required by the codes, has similar stiffness as the beam BSU1 with the same *a/d* ratio and with ρ_v is equal to 0.24. Increasing the *a/d* ratio leads to a considerable reduction in the stiffness of the tested beams. Beam BSU5 with *a/d* ratio equals to 3.0 is less rigid than beam BSU4 with *a/d* equal to 2.75.

Records of the longitudinal steel strains at the mid-span of the tested beams showed that the tensile strains in the region of maximum bending moment at the mid span are almost uniform at every load level. Failure of all the tested beams occurred before yielding of the longitudinal bars. Tensile steel strains increase approximately at a constant rate. Formation of inclined diagonal shear cracks has no effect on the strain readings of the longitudinal bars. For the same *a/d* ratio, the strain readings in the longitudinal bars are approximately similar. Records of the strains in the vertical legs of the stirrups in the shear span of the tested beams are compared in Fig. 6. It can be seen that the recorded tensile steel strain in the stirrup leg is very small at the first stage of loading and increase approximately at a constant rate. When the diagonal shear crack is initiated, the strain rate increases more fast until reaching the yield value just before the applied ultimate load. After yielding of the stirrup leg, the recorded strains increase with very fast rate with decreasing the applied load, and at the same time very fast increase of the shear crack width takes place until crushing of the concrete in the shear zone. For the same *a/d* ratio, increasing the stirrup diameter and reducing the spacing between stirrups reduce the recorded strains of the tested beams at the same load.

4.3 Effect of Vertical Web Reinforcement Ratio.

The effect of the provided vertical web reinforcement ratio ρ_v% in the form of the spacing between stirrups s_v and the stirrup bar diameter d_v on the diagonal cracking strength and the ultimate shear strength of the tested beams can be observed from Table 4 and Fig. 4. It can be seen that, the smaller the s_v, the slower the diagonal crack development and the smaller the d_v, the faster the diagonal crack development. For the tested beam BSU1 with s_v equals to 100 mm, the development of diagonal cracks was considerably slower than that of the similar beam BSU2 but with s_v equal to 200 mm. For beam BSU4 with stirrup bar diameter 8 mm and s_v equals to 200 mm, the development of diagonal cracks was slightly slower than that of the similar beam BSU2 with stirrup bar diameter 6 mm and s_v equal to 200 mm. It should be noted that, beams BSU2, BSU4 and BSU5 have the maximum spacing between stirrups $s_{v,max}$ allowed by the ECP-203-2017. It can be seen that, reducing the spacing between stirrups slower the development of the diagonal cracks more than increasing the diameter of the stirrups. For Beam BSU2 with s_v equal to 200 mm which is equal to $s_{v,max}$ allowed by ECP-203-2017, the recorded diagonal cracking load V_{cr} was about 86% of that of the similar beam BSU1 with the same d_v and half the spacing between stirrups (s_v = 100 mm), while V_{cr} for beam BSU2 with d_v equal to 6 mm was 94% of beam BSU4 with the same s_v but with d_v equal to 8 mm. It should be noted that the provided s_v for all the tested beams is greatly more than $s_{v,max}$ required by ACI 318-2014. However, good overall behavior was observed for UHPFRC beams reinforced with the maximum spacing between stirrups.

The results of the tests showed that the requirements of ECP-203-2017 for $s_{v,max}$ can be safely applied for UHPFRC beams. The requirements of ACI 318-2014 for $s_{v,max}$ is not practically suitable for UHPFRC beams with relatively small heights and can be safely increased to be *0.75d* instead *0.50d*. From Table 4, it can be seen that the ultimate shear strength of the tested beams slightly increases as the provided vertical web reinforcement ratio ρ_v% increases for the beams of the same *a/d* ratio. Despite the provided ρ_v% of beam BSU1 is equal to 0.24% which increases about 200% of that of the similar beam BSU2 with $\rho_{v,min}$% according to ECP-203-2017, the increase in the shear strength was only about 9.7%. In addition, beam BSU4 with ρ_v% is equal to 0.21% which increases about 175% of that of the similar beam BSU2 with $\rho_{v,min}$%, the increase in the shear strength was only about 4.8%. For beam BSU3 with ρ_v% equal to 0.42% which increases about 350% of that of the similar beam BSU2 with $\rho_{v,min}$%, the increase in the shear strength was about 22.9%. This showed that, the steel fibers plays the great role in resisting the shear stresses of UHPFRC beams and the effect of increasing ρ_v% more than the minimum required by ECP-203-2017 has not considerable effect on the shear strength of the tested beams, and consequently the requirements of this code for $\rho_{v,min}$% can be safely applied for UHPFRC beams containing steel fibers of 1.5%. It should be noted that, according to ACI 318-2014, the provided ρ_v of beams BSU2, BSU4 and BSU5 is less than $\rho_{v,min}$, while ρ_v for BSU1 is equal to $\rho_{v,min}$, while according to EC-2, the provided ρ_v of beams BSU1, BSU2, BSU4 and BSU5 is less than $\rho_{v,min}$. However, good overall behavior was observed for all the tested UHPFRC beams reinforced with ρ_v less than $\rho_{v,min}$ required by these codes. This indicates that the requirements of ACI 318-2014 and EC-2 for the minimum vertical web reinforcement ratio can be safely reduced when applied to UHPFRC beams containing steel fibers of 1.5%.

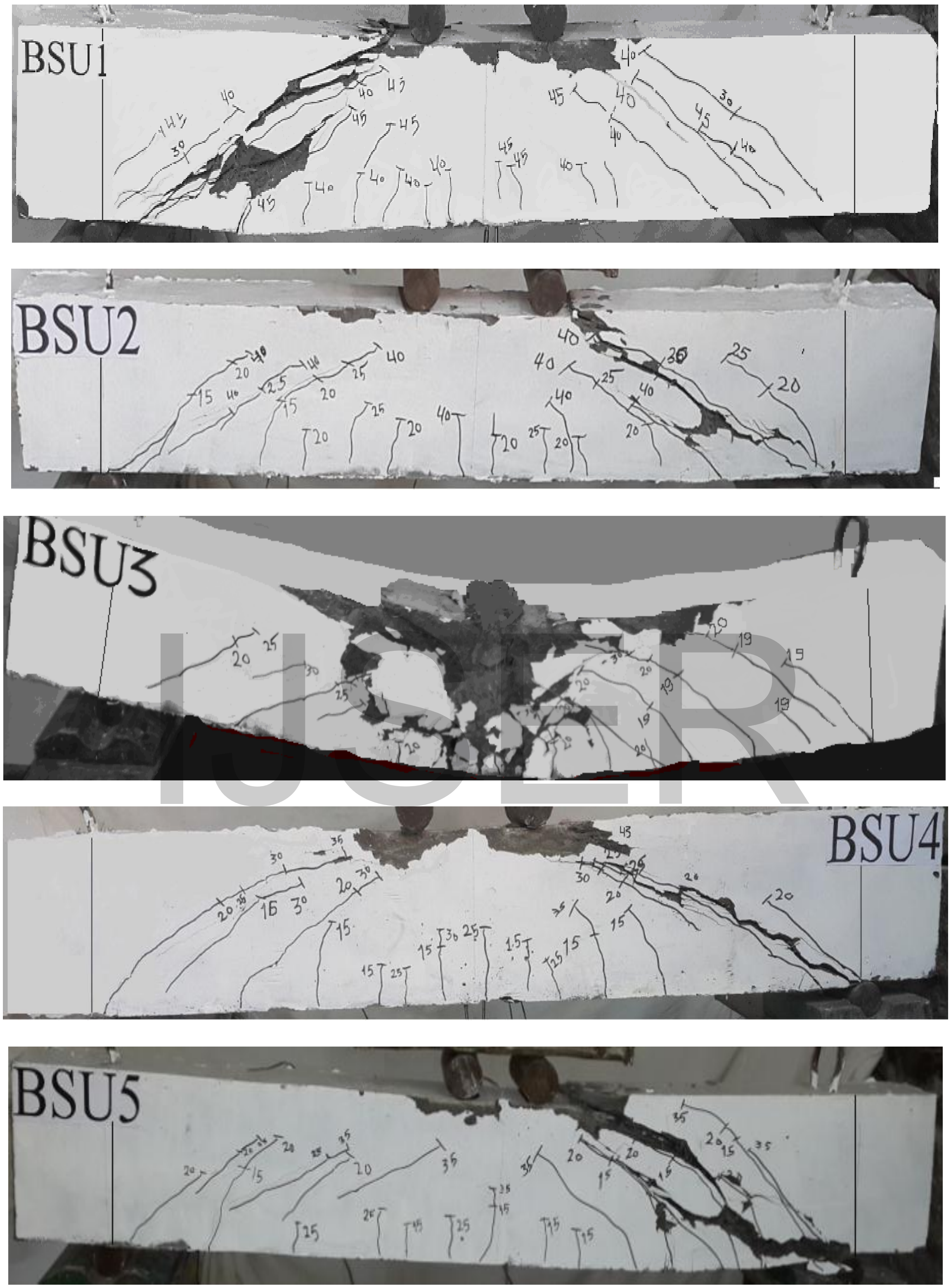

Fig. 3. Photographs of the crack patterns of the tested beams (Note: load marked on the specimens are in tons)

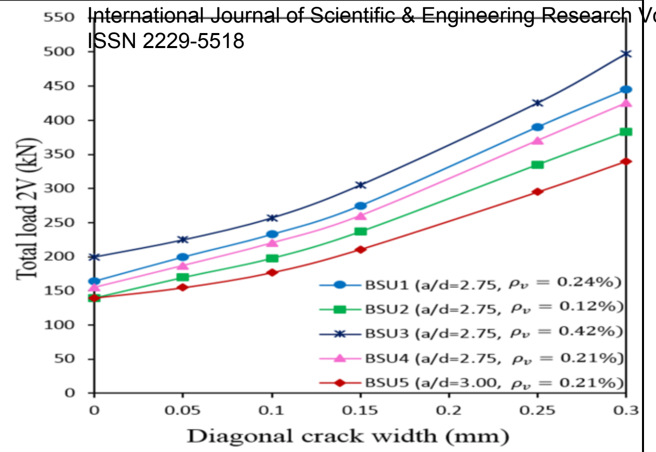

Fig. 4. Diagonal crack development for beams with different *a/d* ratio.

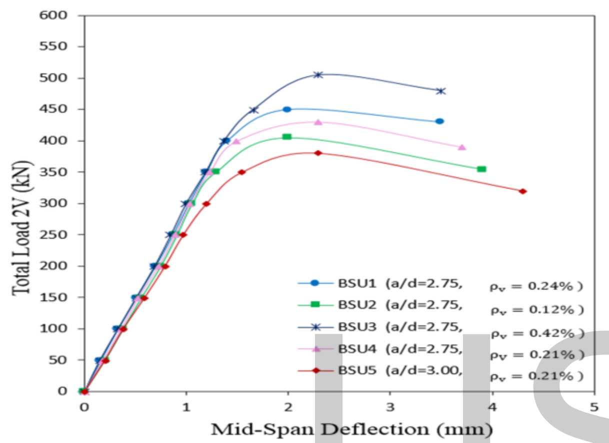

Fig. 5. Total load-mid span deflection relationships for the tested beams.

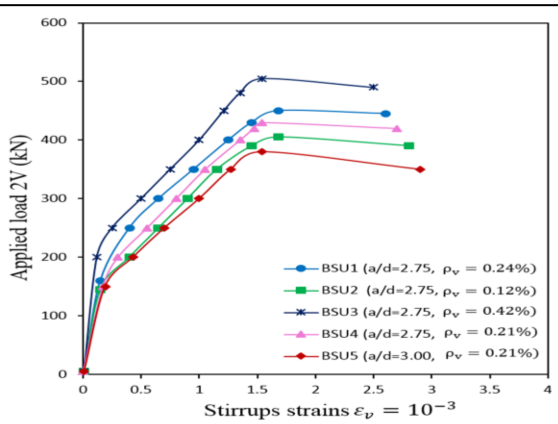

Fig. 6. Total load- strains in the vertical legs of stirrups relationships of the tested beams.

In addition, beam BSU4 with ρ_v% is equal to 0.21% which increases about 175% of that of the similar beam BSU2 with $\rho_{v,min}$%, the increase in the shear strength was only about 4.8%. For beam BSU3 with ρ_v% equal to 0.42% which increases about 350% of that of the similar beam BSU2 with $\rho_{v,min}$%, the increase in the shear strength was about 22.9%. This showed that, the steel fibers plays the great role in resisting the shear stresses of UHPFRC beams and the effect of increasing ρ_v% more than the minimum required by ECP-203-2017 has not considerable effect on the shear strength of the tested beams, and consequently the requirements of this code for $\rho_{v,min}$% can be safely applied for UHPFRC beams containing steel fibers of 1.5%. It should be noted that, according to ACI 318-2014, the provided ρ_v of the tested beams BSU2, BSU4 and BSU5 is less than $\rho_{v,min}$, while ρ_v for BSU1 is equal to $\rho_{v,min}$, while according to EC-2, the provided ρ_v of beams BSU1, BSU2, BSU4 and BSU5 is less than $\rho_{v,min}$. However, good overall behavior was observed for all the tested UHPFRC beams reinforced with ρ_v less than $\rho_{v,min}$ required by these codes. This indicates that the requirements of ACI 318-2014 and EC-2 for the minimum vertical web reinforcement ratio can be safely reduced when applied to UHPFRC beams containing steel fibers of 1.5%.

5 Comparison of Test Results with Codes Predictions

The shear design equations adopted by ACI 318-2014, ECP-203-2017 and EC-2 are used to calculate the ultimate shear strength of the tested UHPFRC beams of this study. It should be noted that the provided ρ_v of the tested beams is compared in Table 2 with the minimum requirements of the studied codes, while the ratio between the calculated ultimate shear strength $V_{u,cal}$ using the studied codes and the measured experimental ultimate shear strength $V_{u,exp}$ are given in Table 5. It can be seen that, for the tested beams, the average ratio between $V_{u,cal}$ and $V_{u,exp}$ for ACI 318-2014, ECP-203-2017 and EC-2 is equal to 0.352, 0.376 and 0.386, respectively. This indicates that, the recorded values of $V_{u,exp}$ of all the tested beams was substantially greater than that calculated from all the studied codes although all the tested beams were provided with stirrups having s_v greatly more than $s_{v,max}$ required by ACI 318-2014, while for EC-2, the provided s_v for beams BSU2, BSU4 and BSU5 is more than $s_{v,max}$ required by this code. This showed that the equations for calculation of the shear strength adopted by the studied codes are not applicable to UHPFRC beams because the adopted equations do not take into consideration the considerable contribution of the steel fibers in resisting shear stresses. It should be noted that, according ACI 318-2014, if the normalized shear strength of reinforced concrete beam containing steel fibers is greater than 0.29, the steel fibers can use as the shear reinforcement for the beam (for $f_c' \leq$ 40 MPa, $d \leq 600$ mm). The calculated values of the normalized shear strength in Table 4 for all the tested UHPFRC beams with fibers percent 1.5% is considerably greater than 0.29 with an average value of 0.86.

6 Comparison of Test Results with the Design Recommendations of AFGC-2002 and KCI-2012 for UHPFRC Beams

In order to take into account the contribution of the steel fibers in the design of UHPFRC structures, two design recommendations has been proposed, the first by the France Association of Civil Engineers (AFGC-2002) and the second by the Korean Concrete Institute (KCI-2012). The shear design methods adopted by AFGC-2002 and KCI-2012 are used to calculate the ultimate shear strength of the tested UHPFRC beams of this study and the calculated values are compared with the recorded experimental ultimate shear strength in Table 6. It can be seen that, the average ratio between $V_{u,exp}$ and $V_{u,AFGC}$ of the tested beams is equal to 1.446, while the average ratio between $V_{u,exp}$ and $V_{u,KCI}$ is equal to 1.462. This indicates that, the AFGC-2002 and KCI-2012 predictions for the ultimate shear strength are safe and conservative when applied for UHPFRC beams provided with shear reinforcement less than the minimum required by the studied codes. The comparison showed that there is very small differences between the predictions of the ultimate shear strength using KCI-2012 and AFGC-2002 recommendations for all the tested beams. In fact, this little difference in the predictions of the two recommendations resulted from the small difference in the safety factors considered in each method. It should be noted from Table 6 that, the average percentage of the predicted contribution of the steel fibers V_{fb} compared with the predicted ultimate shear strength of the tested beams $V_{u,AFGC}$ using the AFGC-2002 recommendations is about 58.3%, while the average percentage of the predicted contribution of the concrete V_c and the shear reinforcement V_s is only 41.7%. The average percentage of V_{fb} compared with the predicted $V_{u,KCI}$ using the KCI-2012 recommendations is about 59.2%, while the average percentage of V_c and the shear reinforcement V_s is only 40.8%.

7 Analytical Modeling for UHPFRC Beams Using Finite Element Program

In order to predict the complete response of reinforced concrete beams such as, displacements, strains and stresses distributions, ultimate shear loads and failure modes, and cracking patterns, a three dimensional nonlinear finite element model using the computer program ABAQUS [17] is utilized. Concrete is modeled using a three dimensional reinforced concrete element named SOLID C3D8R element, which is capable of cracking in tension and crushing in compression. The element is defined by eight nodes having three translational degrees of freedom (x, y and z) at each node. The main and web reinforcement are modeled using a bar element (T2D3) within the concrete solid 65 element. The bar element is assumed to be smeared within the concrete solid element. Each specimen is meshed according to the reinforcement details and size. Fig. 7 shows the finite element meshing of the tested beam BSU1.

In this model, nonlinear constitutive models of UHPFRC and reinforcement are introduced. The modulus of elasticity for steel E_s, is taken equal to 200 GPa and the Poisson's ratio is equal to 0.30. The bond between concrete and reinforcement was assumed to be perfect. Based on experimental tests conducted in this study, the elasticity modulus E_c of UHPFRC is calculated from the following proposed simple equation:

$$E_c = 3737 \sqrt{f_{cu}} \quad (\text{MPa}) \qquad (10)$$

For the tested beams with f_{cu} is equal to 188.1 MPa, the adopted value of E_c is equal to 51253 MPa. The tensile strength of UHPFRC is taken equal to 8 MPa and the Poisson's ratio is taken equal to 0.20.

To examine the accuracy of the nonlinear finite element model, the obtained results are compared with the results of the tested UHPFRC beams of the present study. A comparison between the recorded experimental cracking load and The recorded ultimate load and the predicted values for the tested beams calculated from the finite element model are given in Table 7. The mean value of the ratio of $V_{u,exp}$ to $V_{u,FEM}$ for UHPFRC beams is equal to 1.012, while the mean value of the ratio of $V_{cr,exp}$ to $V_{cr,FEM}$ is equal to 1.028. This shows that the nonlinear finite element model provides accurate prediction of the diagonal cracking load and the ultimate shear load for the tested UHPFRC beams. The same results can be concluded for the deformation response of the tested beams. It is clear that the adopted nonlinear finite element model provides useful tool in understanding the shear behavior of UHPFRC beams.

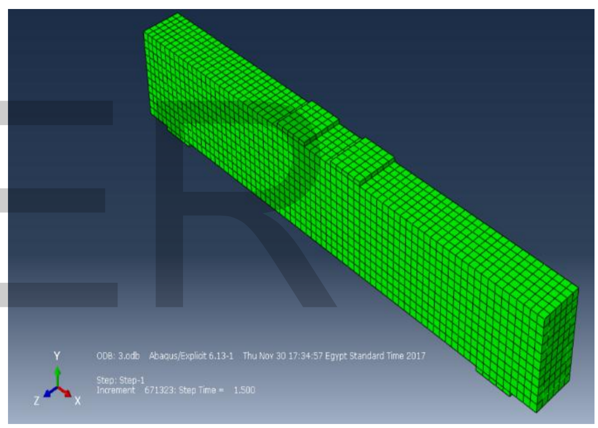

Fig. 7. Meshing of UHPFRC beam BSU1 using finite element model.

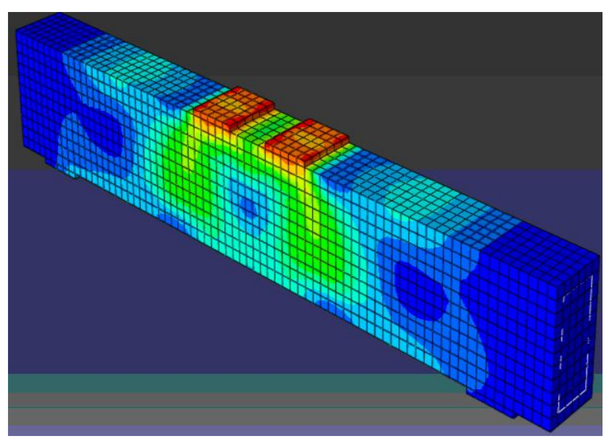

Fig. 8. Stress distribution of UHPFRC beam BSU1 using finite element model.

TABLE 5

COMPARISON BETWEEN THE EXPERIMENTAL RESULTS AND THE ULTIMATE SHEAR

STRENGTH PREDICTED BY THE STUDIED CODES

Beam		ACI 318-2014				ECP 203-2017				EC-2			
	$V_{u,exp}$	V_c (kN)	V_s (kN)	$V_{u,cal}$ (kN)	$\frac{V_{u,cal}}{V_{u,exp}}$	Vc (kN)	Vs (kN)	$V_{u,cal}$ (kN)	$\frac{V_{u,cal}}{V_{u,exp}}$	V_c (kN)	V_s (kN)	$V_{u,cal}$ (kN)	$\frac{V_{u,cal}}{V_{u,exp}}$
BSU1	225.0	45.6	32.3	77.9	0.35	54.8	28.1	82.9	0.37	56.0	29.1	85.1	0.38
BSU2	202.5	45.6	16.2	61.8	0.31	54.8	14.0	68.8	0.34	56.0	14.5	70.5	0.35
BSU3	252.5	45.6	52.7	98.3	0.39	54.8	45.8	100.6	0.40	56.0	47.4	103.4	0.41
BSU4	215.0	45.6	26.4	72.0	0.33	54.8	22.9	77.7	0.36	56.0	23.7	79.7	0.37
BSU5	190.0	45.6	26.4	72.0	0.38	54.8	22.9	77.7	0.41	56.0	23.7	79.7	0.42

TABLE 6
COMPARISON BETWEEN THE EXPERIMENTAL RESULTS AND THE ULTIMATE SHEAR STRENGTH CALCULATED USING THE DESIGN RECOMMENDATIONS FOR UHPFRC

Beam	a/d		KCI -2012					AFGC-2002				
		$V_{u,exp}$ (kN)	V_c (kN)	V_{fb} (kN)	V_s (kN)	$V_{u,KCI}$ (kN)	($V_{u,exp}/V_{u,KCI}$)	V_c (kN)	V_{fb} (kN)	V_s (kN)	$V_{u,KCI}$ (kN)	($V_{u,exp}/V_{u,AFGC}$)
BSU1	2.75	225.0	37.2	87.4	24.9	149.5	1.51	37.6	90.4	29.1	157.1	1.43
BSU2	2.75	202.5	37.2	87.4	12.5	137.1	1.48	37.6	90.4	14.5	142.5	1.42
BSU3	2.75	252.5	37.2	87.4	40.6	165.2	1.53	37.6	90.4	47.4	175.4	1.44
BSU4	2.75	215.0	37.2	87.4	20.3	144.9	1.48	37.6	90.4	23.7	151.7	1.42
BSU5	3.00	190.0	37.2	87.4	20.3	144.9	1.31	37.6	90.4	23.7	151.7	1.52

TABLE 7
COMPARISON BETWEEN THE EXPERIMENTAL RESULTS AND THE ANALYTICAL MODEL USING FINITE ELEMENT PROGRAM

Beam	Cracking load and Ultimate Load						Cracking Displacement and Maximum Displacement					
	$V_{cr,FEM}$ (kN)	$V_{cr,exp}$ (kN)	$\frac{V_{cr,\exp}}{V_{cr,FEM}}$	$V_{u,exp}$ (kN)	$V_{u,FEM}$ (kN)	$\frac{V_{u,\exp}}{V_{u,FEM}}$	$\Delta_{cr,FEM}$ (mm)	$\Delta_{cr,exp}$ (mm)	$\frac{\Delta_{cr,\exp}}{\Delta_{cr,FEM}}$	$\Delta_{u,exp}$ (mm)	$\Delta_{u,FEM}$ (mm)	$\frac{\Delta_{u,\exp}}{\Delta_{u,FEM}}$
BSU1	78.0	82.5	1.06	225.0	205.0	1.10	0.57	0.40	0.72	2.34	2.53	0.92
BSU2	57.0	62.5	1.10	202.5	193.5	1.05	0.48	0.37	0.77	2.95	3.04	0.97
BSU3	97.0	92.5	0.95	252.5	285.0	0.89	0.33	0.35	1.12	2.60	2.47	1.05
BSU4	63.0	70.0	1.11	215.0	205.5	1.05	0.27	0.28	1.04	2.63	2.53	1.04
BSU5	65.0	60.0	0.92	190.0	196.5	0.97	0.62	0.48	0.83	3.47	3.24	1.07

CONCLUSION

Based on the results of this experimental and analytical study on the shear behavior of Ultra-High Performance Fiber Reinforced Concrete (UHPFRC) beams containing steel fibers volume fraction of 1.5% with a characteristic concrete cube strength approaching 188.1 MPa and reinforced with minimum web reinforcement, the following can be concluded:

1- The steel fibers plays a great role in resisting the shear stresses of UHPFRC beams. The provided vertical web reinforcement ratio of the tested UHPFRC beams was considerably less than the minimum ratio required by ACI 318-2014 building code and Eurocode-2 (EC-2), however, good overall behavior was observed for all the tested UHPFRC beams.

2- The equations for calculating the shear strength adopted by the studied international codes highly underestimate the shear strength of the tested UHPFRC beams provided with minimum vertical web reinforcement ratio, and consequently, these equations are not applicable to UHPFRC beams because the adopted equations do not take into consideration the considerable contribution of the steel fibers in resisting shear stresses.

3- The provided shear reinforcement has slight effect on the ultimate shear strength of UHPFRC beams containing steel fibers volume fraction of 1.5%. For the tested beams of the same *a/d* ratio, increasing the provided vertical web reinforcement ratio by 350% greater than the minimum required by the Egyptian code ECP-203-2017, increases the ultimate shear strength by only 22.9%.

4- The requirements of the Egyptian code ECP-203-2017 for maximum spacing between stirrups ($s_{v,max}$ is equal to 200 mm) can be safely applied for UHPFRC beams. The requirements of ACI 318-2014 for the maximum spacing between stirrups is not practically suitable for UHPFRC beams with relatively small heights and can be safely increased to be *0.75d* instead *0.50d*.

5- The recommendations for design of UHPFRC proposed by AFGC-2002 and also that proposed by KCI-2012 are safe and conservative for the ultimate shear strength predictions of the tested beams. The predictions of the ultimate shear strength according to KCI-2012 is approximately similar to that of AFGC-2002 (the average ratio between the experimental ultimate shear strength and that predicted using AFGC-2002 and KCI-2012 is equal to 1.446 and 1.462, respectively). The average contribution of the steel fibers for the tested UHPFRC beams is about 60% of the recorded ultimate shear strength.

6- Including the effect of steel fibers in the proposed finite element model accurately predicts the ultimate shear strength and the deformation response of the tested UHPFRC beams provided with vertical web reinforcement ratio considerably less than the minimum required by the codes.

REFERENCES

[1] ACI Committee 363, "Ultra High Performance Concrete, A State-of-the-Art Report for the Bridge Community," Publication No. FHWA-HRT-13-060, June, pp. 13-44, 2013.

[2] P. Buitelaar, "Ultra High Performance Concrete: Developments and Applications during 25 Years, " Plenary Session International Symposium on UHPC, Kassel, Germany, pp. 1-13.L, 13-15 September 2004.

[3] M. Schmidt, E. Fehling, and C. Geisenhanslüke, "Ultra High Performance Concrete (UHPC), " International Symp. On Ultra High Performance Concrete, Kassel, Germany, September 13-15, 2004.

[4] Strategic Development Council, "Ultra-High Performance Concrete (UHPC): What It Is , Its History, and Why We Care," Chicago, Illinois, pp. 7-20, 21 September 2011.

[5] K. Rokugo, T. Kanda, H. Yokota, and N. Sakata, "Applications and recommendations of High Performance Fiber Reinforced Cement Composites with Multiple Fine Cracking (HPFRCC) in Japan, " Materials and Structures, vol. 42, 2009, pp. 1197-1208.

[6] K. Wille, A. E. Naaman, and G. J. Parra-Montesions, "Ultra-High-Performance Concrete with Compressive Strength Exceeding 150 MPa (22 ksi), A simpler Way, " ACI Materials Journal, vol. 108, no. 6, pp. 46-54, 2011.

[7] D. Y. Yoo, and Y. S. Yoon, " A Review on Structural Behavior, Design and Application of Ultra-High-Performance Fiber-reinforced Concrete," International Journal of Concrete Structures and Materials, vol. 10, no. 2, pp. 125-142, June 2016.

[8] F. Baby, P. Marchand, and F. Toutlemonde, "Shear Behavior of Ultrahigh Performance Fiber Reinforced concrete Beams. II: Analysis and Design Provisions," Journal of Structural Engineering, ASCE, vol. 140, no. 5, May 2014.

[9] E. Ferrier, A. Confrere, L. Michel, G. Chanvillard, and S. Bernardi, " Shear Behavior of New Beams Made of UHPC Concrete and FRP Rebar," Composites Part B, Elsevier Ltd, pp. 1-13,2016.

[10] W. Y. Lim, and S. G. Hong, " Shear Test for Ultra-High Performance Fiber Reinforced concrete (UHPFRC) Beams with shear reinforcement," vol. 10, no.2, pp.177-188, June 2016.

[11] ACI Committee 318, " Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14)," American Concrete Institute, Farmington Hills, Mich., 2014.

[12] European Committee for Standardization, "Eurocode 2: Design of Concrete Structures- Part 1," Brussels, December, 2004.

[13] Ministry of Housing, Utilities and Urban Communities, "Egyptian Code for Design and Construction of Reinforced Concrete Structures (ECP 203-2017)," Cairo, 2017.

[14] Association Francaise du Genil Civil (AFGC)," Ultra High-Performance Fiber Reinforced Concrete," France, 2002.

[15] Japan Society of Civil Engineers (JSCE), " Recommendations for Design and Construction of Ultra High-Strength Fiber Reinforced Concrete Structures," Concrete Engineering Series no. 82, March, 2008.

[16] Korea Concrete Institute, " Design Recommendations for Ultra High Performance Concrete (K-UHPC), KCI-M-12-003, " Seoul, Korea, 2012.

[17] ABAQUS, "Abaqus Analysis User's Manual, Version 6.9," Dassault Systems, 2009.