BEHAVIOR OF HIGH PERFORMANCE FIBER REINFORCED CONCRETE COLUMNS

Wasan Ismail Khalil, Ikbal Naeem Gorgis and Zeinab Raad Mahdi
Department of Building and Construction Engineering, University of Technology, Baghdad, Iraq
E-Mail: iqbalkorkess@yahoo.com

ABSTRACT

An experimental program was carried out in this research to investigate the behavior of square short high performance concrete columns with and without steel fiber under concentric compression loading. The effect of concrete strength, percent of longitudinal reinforcement, spacing of lateral confined stirrups, volumetric steel fiber ratio and aspect ratio of steel fiber on the uniaxial behavior of reinforced HPC columns were presented and discussed. The results show that adding discrete fibers to HPC mixtures in reinforced concrete columns not only prevents the premature spalling of the concrete cover but also increases the strength and ductility of the axially loaded reinforced columns. More confinement is required in HPC columns in comparison with that normal strength concrete to achieve the desired post-beak deformability.

Keywords: high performance concrete HPC, longitudinal and transverse reinforcement, cnfinement, steel fibers.

INTRODUCTION

The use of high strength concrete (HSC) is continuously increasing due to its mechanical and durability advantages over normal strength concrete (NSC). In high-rise buildings, HSC can reduce the dimensions of the lower-story columns, which makes it a more cost-effective choice for builders than NSC [1, 2].

High Performance Concrete (HPC) is often said to be equivalent to HSC. The efficiency of HPC, however, is not limited to this property, although a high strength is essential in heavy-loaded members, e.g. in high-rise buildings. An improved durability or an increased resistance of the concrete against chemical or physical attack is much more important. An essential property of HPC is that this resistance is improved by technological measures, i.e., the microstructure of the concrete is improved, so that the usual additional protective measures can be omitted. A dense microstructure of the concrete is always combined with an increased strength. Therefore, HSC and concrete with a dense microstructure do not differ very much in their technologies.

HSC is more brittle in comparison with NSC and that the confinement provided to HSC is less effective than in NSC. Therefore, greater confinement is required for columns made from higher strength concrete to achieve similar strength and ductility enhancements. This behavior is the main obstacle to HSC's widespread use [3, 4, 5, 6].

The inclusion of short discrete fibers into the concrete mixture can increase HSC compressive strength and ductility, as already demonstrated by several studies [7, 8, 9]. These studies have also shown that the combination of discrete fibers and transverse steel reinforcement can reduce the relatively high amount of confinement reinforcement required by design codes e.g., ACI 318-11 [10] and CSA 23.3-04 Canadian Standard Association [11] for HSC columns. Another advantage of adding discrete fibers to HSC mixtures in reinforced concrete columns is to prevent the premature separation of the concrete cover [12, 13]. Interest in using fiber-reinforced concrete (FRC) for structural members has increased in recent decades both for scientists and producers, and especially in the field of precast members. Several experimental and theoretical investigations highlight the effectiveness of using FRC for members of framed structures.

EXPERIMENTAL WORK

Materials

Cement

Ordinary Portland local cement from Tasluja factory was used in all mixes throughout this research. Chemical and Physical tests results indicated that the cement conforms to the provisions of Iraqi specification No.5/1984.

Fine aggregate

AL-Ukhaider natural sand of maximum size 4.75 mm was used. Its gradation lies in zone (2), the gradation and sulfate content results were within the requirements of the Iraqi specification No. 45/1980.

Coarse aggregate

Normal weight, crushed aggregate of maximum size 10 mm was used. It was brought from AL-Nabai region. The grading and sulfate content of coarse aggregate conform to the requirements of Iraqi specification No. 45/1980.

Admixtures

Two types of admixtures were used in this work:

(i) Super plasticizer (S.P.)

A high range water reducing admixture (HRWRA) was used to produce the HPC mix. It is commercially known as Top Flow SP 703. The dosage recommended by the manufacturer was (0.75-2) liters/100 kg of cementations material. This type of

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admixture conforms to the requirement of ASTM C494-04.

(ii) Silica fume

Silica fume was used as pozzolanic admixture; the ACI committee 234 instructions were followed to determine the optimum dosage of silica fume. Pozzolanic activity index and the chemical oxide compositions results of silica fume conform to the requirements of ASTM C1240-05 specifications.

Fibers

Two types of hooked end steel fibers were used, the first type commercially known as Dramix-Type ZC, these fibers were 50 mm long and 0.5 mm diameter (aspect ratio, l/d = 100), the density of the steel fibers is 7850 kg/m^3, and the ultimate tensile strength for individual fibers is 1117 MPa. The second type of hooked steel fibers known commercially as Sika Fiber SH 60/30, these fibers were 30 mm long and 0.5 mm diameter (aspect ratio, l/d = 60) with ultimate tensile strength for individual fibers of 1180 MPa. The density of the steel fibers was 7800kg/m^3.

Reinforcing steel bars

Deformed steel bars with nominal diameters 8mm, 10mm and 12 mm were used as longitudinal reinforcement. Three specimens for each bar diameter were tested in tension according to ASTM A996M-05 to determine their properties; the results were summarized in Table-1 and Figure-1.

Table-1. Properties of steel bars.

Nominal bar diameter (mm)	**Bar area** (mm^2)	**Modulus of elasticity** (GPa)	**Yield stress** (MPa)	**Strain at yield stress** (mm/mm)	**Ultimate stress** (MPa)	**Strain at ultimate stress** (MPa)	**Elongation** (%)
8	50.27	202	545	0.0027	660	0.18	15.5
10	78.54	198	514	0.00255	625	0.16	14.5
12	113.1	201	503	0.00252	615	0.13	14.5

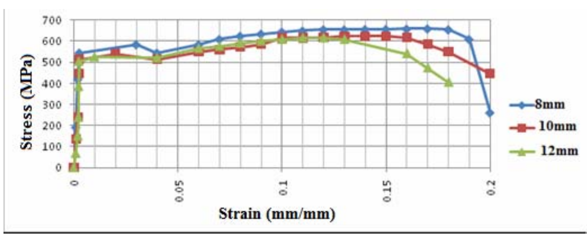

Figure-1. Stress-strain curves for steel reinforcement.

Mixing water

Ordinary potable water was used for mixing and curing all concrete mixes in this investigation.

Concrete mixes

Reference concrete mix for NSC was designed in accordance with ACI 211. 91 [13] for obtaining a minimum compressive strength of 40MPa at 28 days without any admixtures. The mix proportions are 1: 1.19: 1.8 by weight of cement with cement content 525 kg/m^3 and w/c ratio of 0.43 to obtain a slump of 100±5 mm. Several trail mixes were carried out to determine the optimum content of silica fume and the optimum dosage of HRWRA to have the same workability (slump 100 ±5) in order to achieve a mix with compressive strength higher than 70 MPa.

The results indicated that maximum compressive strength obtained was for mixes with 5% silica fume as addition by weight of cement and a dosage of HRWRA 2 liter/100kg of cement. Finally, discrete steel fibers with different aspect ratios (100 and 60) and volume fractions (0.5% and 0.75%) were added to the concrete mixes. Table-2 indicates the details of mix proportion used.

Mixing method

The dry constituents of the reference NSC mix were placed in the mixer and mixed for about 1.5 minutes. Then, the required quantity of water was added. The whole mix ingredients were mixed for about 2 minutes until a homogenous concrete mix was obtained.

For HPC, the required quantity of silica fume was mixed with the cement before the addition to the mixer to ensure homogeneous dispersion of silica fume, and then the dry constituents were mixed for about 1.5 minutes to attain a uniform dry mix. Seventy percent of the required amount of water was added to the mixer and mixed for about 1minute, while the remaining amount of the mixing water was added to the HRWRA and added gradually to the mix. The whole constituents were mixed for further 2 minutes. The same procedure of mixing was carried out for the fiber reinforced concrete mixes, except that the fibers were added by hand after all mix ingredient was thoroughly mixed, and the mixing was then continued for (2-3) minutes to obtain a uniform distribution of fibers throughout the concrete mix.

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Table-2. Details of trail mixes for various volume fraction, aspect ratios and types of fiber.

Mix proportions by weight	HRWRA (liter/100kg of cement)	Silica fume (% as addition by weight of cement)	W/C ratio	Type of fiber	aspect ratio	Fiber volume fraction	Slump (mm)	Slump reduction (%)	Compressive strength (MPa)		Increase in strength with respect to reference mix (%)	
									7days	28days	7days	28days
1: 1.19: 1.8	2	5	0.295	-	-	-	105	-	64.68	69.44	-	-
				Steel	100	0.5	75	28.5	67.85	73.5	4.901	5.846
				steel	100	0.75	65	38.1	68.5	75.25	5.9	8.36
				Steel	100	1	40	61.9	65.5	71.3	1.26	2.67
				Steel	60	0.75	85	19	69.8	78.86	7.91	13.56

Preparation of specimens

Fourteen reinforced concrete columns 100×100×1000 mm were prepared and divided into five series as indicated in Table-3. A mould was prepared from plywood, the edges of the mould were tie up with sufficient numbers of bolts to ensure the water tightness and the mould sides with the base were lightly coated by a mineral oil before casting the concrete.

Four longitudinal reinforcing deformed steel bars one at each corner of column specimen cross section with different diameter (8, 10 and 12) were used. Lateral ties with 8mm diameter were used for all columns specimens as shown in Figure-2. The concrete cover of 12.5 mm was provided in all columns and a cover of 20 mm was provided between the ends of longitudinal bars at the top and bottom surfaces of the specimens to prevent direct loading on the bars.

Control cylinder specimens of 150×300 mm were prepared from the same mix of non fibrous and fibrous concrete for each column specimen in order to determine compressive strength of the concrete. The control specimens were prepared, cured for 60 days then tested at the same age of the columns specimens.

Table-3. Details of the column specimens.

Test series	**Column symbols**	**Volume fraction of fibers V_f**(%)	**Aspect ratio of fibers L/d**	**Compressive strength of concrete** (MPa)	**Diameter of long. reinf. (**mm) (Percent of long. Reinf.) (ρ_l%)	**Spacing of lateral reinf.** (mm) (Percent of lateral reinf.) (ρ_s%)
1	NF-8N100	0	--	About 75	8 (2.01)	100 (2.79)
	0.5SF100-8N100	0.5	100			
	0.75SF100-8N100	0.75	100			
2	NF-10H100	0	--	About 75	10 (3.14)	100 (2.79)
	0.5SF100-10N100	0.5	100			
	0.75SF100-10N100	0.75	100			
3	NF-12H100	0	--	About 75	12 (4.52)	100 (2.79)
	0.5SF100-12H100	0.5	100			
	0.75SF100-12H100	0.75	100			
4	NF-8N100	0	--	About 50	8 (2.01)	100 (2.79)
	0.5SF100-8H100	0.5	100			
5	0.75SF60-8H100	0.75	60	About 75	8 (2.01)	100 (2.79)
	0.75SF60-8H75	0.75	60			75 (3.72)
	0.75SF60-8H50	0.75	60			50 (5.58)

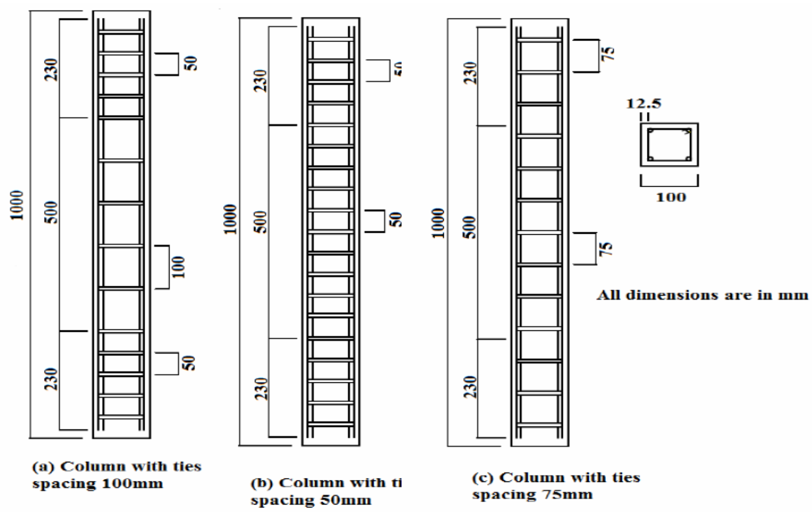

Figure-2. Details of columns reinforcement.

Test program

Universal testing machine with maximum capacity of 2500 kN was used for testing the reinforced concrete columns, as shown in Figure-3. The tested columns were white painted to facilitate detection of cracks. Demec points were used to measure the strains of concrete. They were mounted in three locations along the concrete surface within the effective span of the tested specimen, as shown in Figure-4. In each location, two demec points were mounted at spacing of 100 mm to measure the longitudinal compressive strain and two demecs were horizontally mounted at spacing of 50 mm to measure lateral strains. The column specimens were externally confined by 5 mm thick and 100 mm height steel collars at both ends of the specimen to avoid premature failure at the end regions of the columns throughout the test. The columns gross deflection was measured by a dial gauge of (0.01mm/div) Sensitivity, fixed with the two collars by a steel frame, as shown in Figure-4.

Figure-3. Testing machine for column specimens.

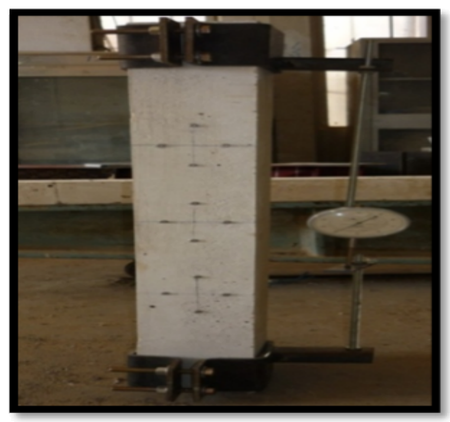

Figure-4. Location of the dial gauges and demec points.

Maximum strength capacity of column specimens

Table-4 shows the results of the experimental maximum strength capacity for all column specimens at age 60 days. The effect of the different variables on the results can be discussed as follows:

a) Effect of concrete compressive strength

The maximum strength capacity results for normal strength and HPC columns with and without steel fibers reinforced with different longitudinal reinforcement ratio are indicated in Table-4. It can be concluded that the peak strength of concrete columns significantly increases as the compressive strength of concrete increases. For non fibrous HPC columns (column No. 3), the percentage increase is about 98% relative to non fibrous NSC columns (column No.1). The percentage increase in peak stress of fiber reinforced columns over non fibrous concrete in the case of HPC is comparatively less marked compared to the NSC columns with the same fiber content and aspect ratio. The addition of 0.5% steel fibers with aspect ratio 100 to NSC (column No. 2) causes an increase in column peak strength of about 54% relative to non

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fibrous NSC columns (column No.1), while the percentage increases for HPC columns containing 0.5% steel fibers (column No.6) is about 23% in comparison with non fibrous HPC columns (column No.3). This suggests that HPC requires more fibers to acquire the same percentage increase in strength and ductility as that provided by normal strength fibrous concrete. Thus, the same percentage of fibers had different effects on the behavior of two concrete mixes with different concrete strengths.

Table-4. Maximum strength capacity for tested column specimens.

Column No.	**Column symbols**	**Type of concrete**	**Type of fibers**	**Compressive strength of concrete** (f'c) (MPa)*	**Long. Rein. Ratio** ρ_l(%)	**Volumetric ratio of ties** ρ_s(%)	**Volume fraction of fibers** V_f(%)	**Aspect ratio of fibers L/d**	**Maximum load of columns Po** (kN)
1	NF-8N100	NSC	-	29.390	2.01	2.79	-	-	260
2	0.5SF100-8N100		Steel fibers	41.465	2.01	2.79	0.5	100	400
3	NF-8H100	HPC	-	48.150	2.01	2.79	-	-	515
4	NF-10H100			48.150	3.14	2.79	-	-	481
5	NF-12H100			48.150	4.52	2.79	-	-	550
6	0.5SF100-8H100		Steel fibers	52.730	2.01	2.79	0.5	100	635
7	0.5SF100-10H100			52.730	3.14	2.79	0.5	100	560
8	0.5SF100-12H100			52.730	4.52	2.79	0.5	100	620
9	0.75SF100-8H100			56.019	2.01	2.79	0.75	100	625
10	0.75SF100-10H100			56.019	3.14	2.79	0.75	100	575
11	0.75SF100-12H100			56.019	4.52	2.79	0.75	100	670
12	0.75SF60-8H100			63.800	2.01	2.79	0.75	60	731
13	0.75SF60-8H50			63.800	2.01	5.58	0.75	60	810
14	0.75SF60-8H75			63.800	2.01	3.72	0.75	60	770

*cylinder compressive strength (150×300) at age 28 days.

The axial stress-strain relationships for fibrous (steel fiber with 0.5% volume fraction and aspect ratio100) and non fibrous normal and HPC columns reinforced with the same volumetric ratio of longitudinal and lateral reinforcement are shown in Figure-5. It can be seen that the inclusion of steel fibers in reinforced normal strength and HPC columns increases the deformability of the specimens. It can also be observed that at the same applied stress, non fibrous and fibrous HPC columns have lower deformability relative to non fibrous and fibrous NSC columns. This indicates that the effectiveness of confinement decreases as the concrete strength increases. This means that higher strength concrete columns require more confinement than normal strength concrete columns.

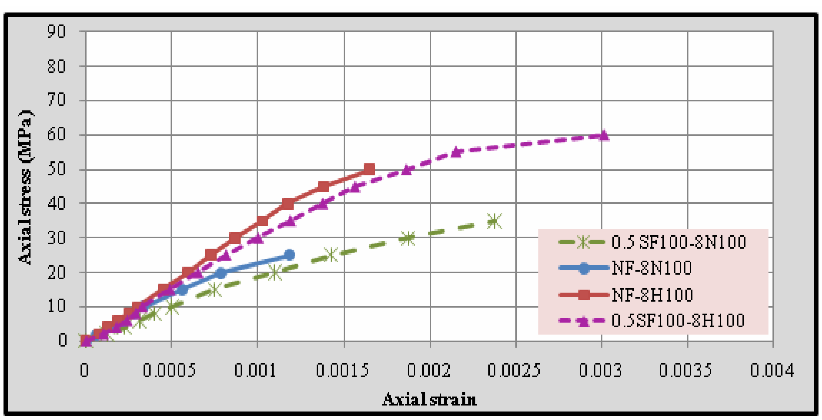

Figure-5. Effect of compressive strength of concrete on the behavior of column specimens.

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b) Effect of volumetric ratio of longitudinal reinforcement

The results in Table-4 indicated that the volumetric ratios of longitudinal reinforcement is slightly affected the peak strength of columns as shown in Figure-6. Column specimens with volumetric ratios of longitudinal reinforcement of 3.14% and 4.52% behaved better than that with longitudinal reinforcement of volumetric ratio 2.01%. This is due to buckling of the longitudinal reinforcement happened subsequently at maximum load and which is critical for column with volumetric ratio of longitudinal reinforcement 2.01% as shown in Figure-7.

The strain and ductility of column specimens are increased as the volumetric ratio of longitudinal reinforcement and volume fraction of fibers increased as shown in Figure-8. The presence of fibers can, however, arrest the propagation of macro-cracks for a longer period of time, and their gradual pull-out improves the ductility, especially at post-peak region [14].

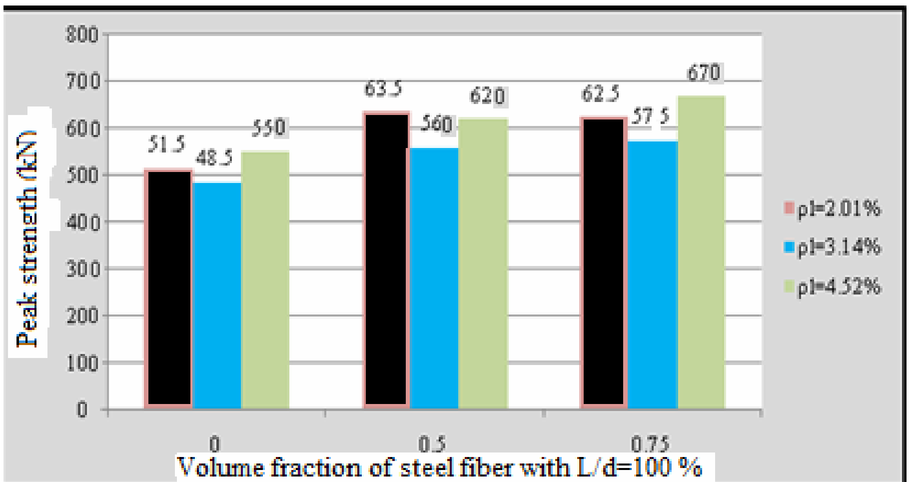

Figure-6. Effect of longitudinal reinforcement ratio for HPC columns on the peak strength capacity of columns.

Figure-7. Buckling of longitudinal reinforcement for columns with volumetric ratio of longitudinal reinforcement (ρ_l = 2.01%).

c) Effect of volumetric ratio of lateral reinforcement (Spacing of ties)

The results from Table-4 demonstrate that the maximum strength capacity slightly increases as the volumetric ratio of ties increases for HPC columns containing steel fibers. The percentage increase for steel fibers HPC columns is about 5% and 11% for volumetric ratio of ties 3.75% and 5.58%, respectively relative to volumetric ratio 2.79%. This is because that smaller tie spacing increases the confined concrete area, resulting in higher confinement efficiency, and the tie spacing controls the lateral buckling of the longitudinal bars [3]. The effect of volumetric ratio of ties for HPC columns containing steel fibers (V_f =0.75% and aspect ratio 60) was shown in Figure-9. It can also be observed that the volumetric ratio of lateral reinforcement slightly affects the ascending part of the stress-strain relationship.

d) Effect of steel fiber content (Volume Fraction) and aspect ratio

The effect of fibers with different contents and aspect ratios on the maximum strength of HPC columns was presented in Figure-10 and Figure-11. Generally, the addition of fibers to HPC columns increases the maximum strength with respect to non fibrous HPC columns specimens. The increase of steel fiber volume fraction from 0.5% to 0.75% with aspect ratio 100 causes a slight decrease in maximum strength of HPC columns. This may be due to the fact that the increase in volume fraction

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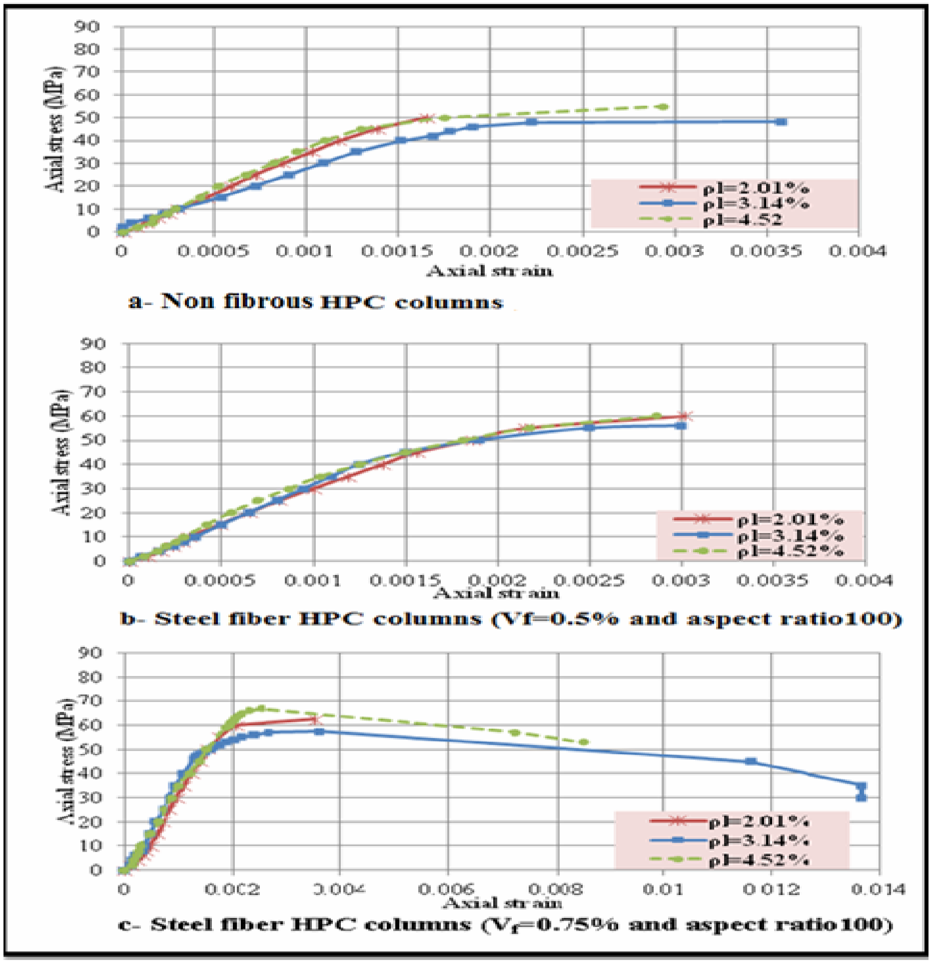
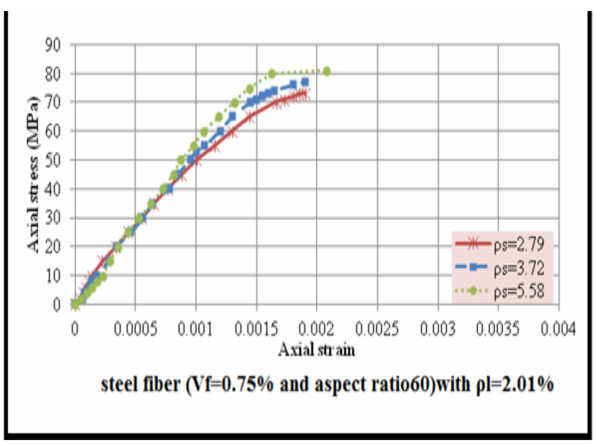


a- Non fibrous HPC columns

b- Steel fiber HPC columns (Vf=0.5% and aspect ratio100)

c- Steel fiber HPC columns (V_f=0.75% and aspect ratio100)

Figure-8. Effect of volumetric ratio of longitudinal reinforcement.



steel fiber (Vf=0.75% and aspect ratio60)with ρl=2.01%

Figure-9. Effect of spacing between ties on HPC fiber reinforced concrete of columns.

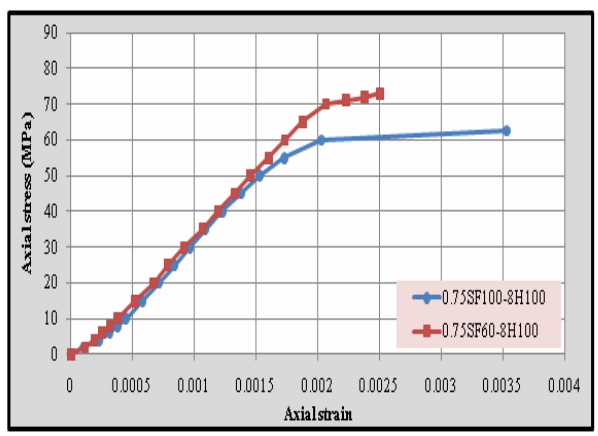


Figure-10. Effect of aspect ratio of steel fiber on the HPC columns.

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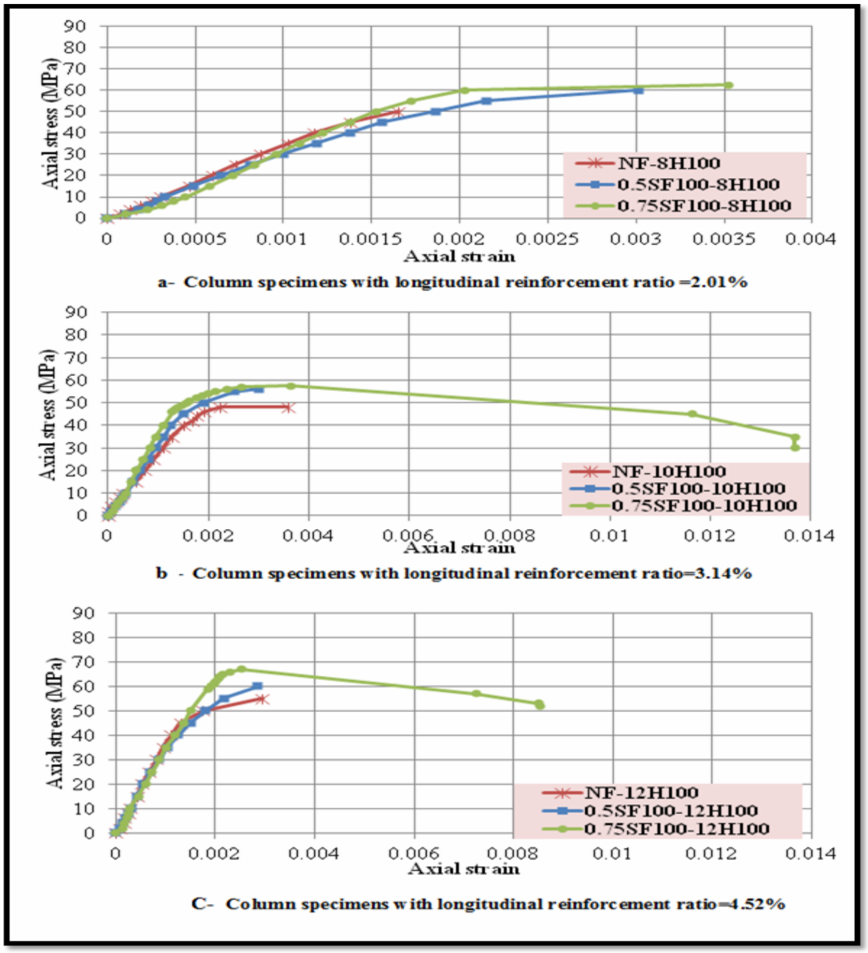

Figure-11. Effect of various contents of steel fiber on the stress-strain relationship of HPC columns.

of fibers leads to a reduction in the compatibility of the mix, thus increasing the entrapped air, and this is followed by a decrease in strength and density [16]. The results also demonstrate that at a constant steel fiber volume fraction, the percentage increase in peak strength decreases with the increase in the aspect ratio from 60 to 100. The percentage increase in peak strength for HPC columns containing 0.75% steel fibers is about 42% and 21% for specimens with short fibers (aspect ratio 60) and long fibers (aspect ratio 100), respectively. This is probably due to the fact that short fibers become active earlier than longer fibers to control the initiation and propagation of initial micro-cracks, as well as under the increasing loads, once the cracks become quite wide in this stage, the short fibers begin to pullout of the matrix, and their crack bridging capability is relatively diminished as compared to longer fibers. For a given volume fraction of fibers added to the matrix, the short fibers are greater in number and, therefore, much closer together. Thus, short fibers influence to a greater degree the early part of matrix cracking, thereby enhancing the strength of the composite as compared to longer fibers [17].

Figure-11 showed that the fiber volume fractions slightly affect the ascending part of the stress-strain relationship. Column specimens containing 0.75% steel fibers with aspect ratio 100 have a stress-strain curve extend beyond the peak load when the volumetric ratio of longitudinal reinforcement is 3.14% and 4.52% (Ø10 and Ø12), which means that the ductility of these columns significantly improved.

Axial strain of concrete

The axial strain of concrete for column specimens was measured at three locations (top, middle and bottom) along the length of column specimens using strain readout unit (demec point). Each value of strain is the average of two readings. Some results were plotted in Figure-12 for different values of stress prior to the occurrence of cracks, since the measurement of strain after the presence of cracks leads to scattered readings due to cracking of concrete. It can be concluded that the addition of fibers enhances the deformability for both NSC and HPC columns at a specified axial load.

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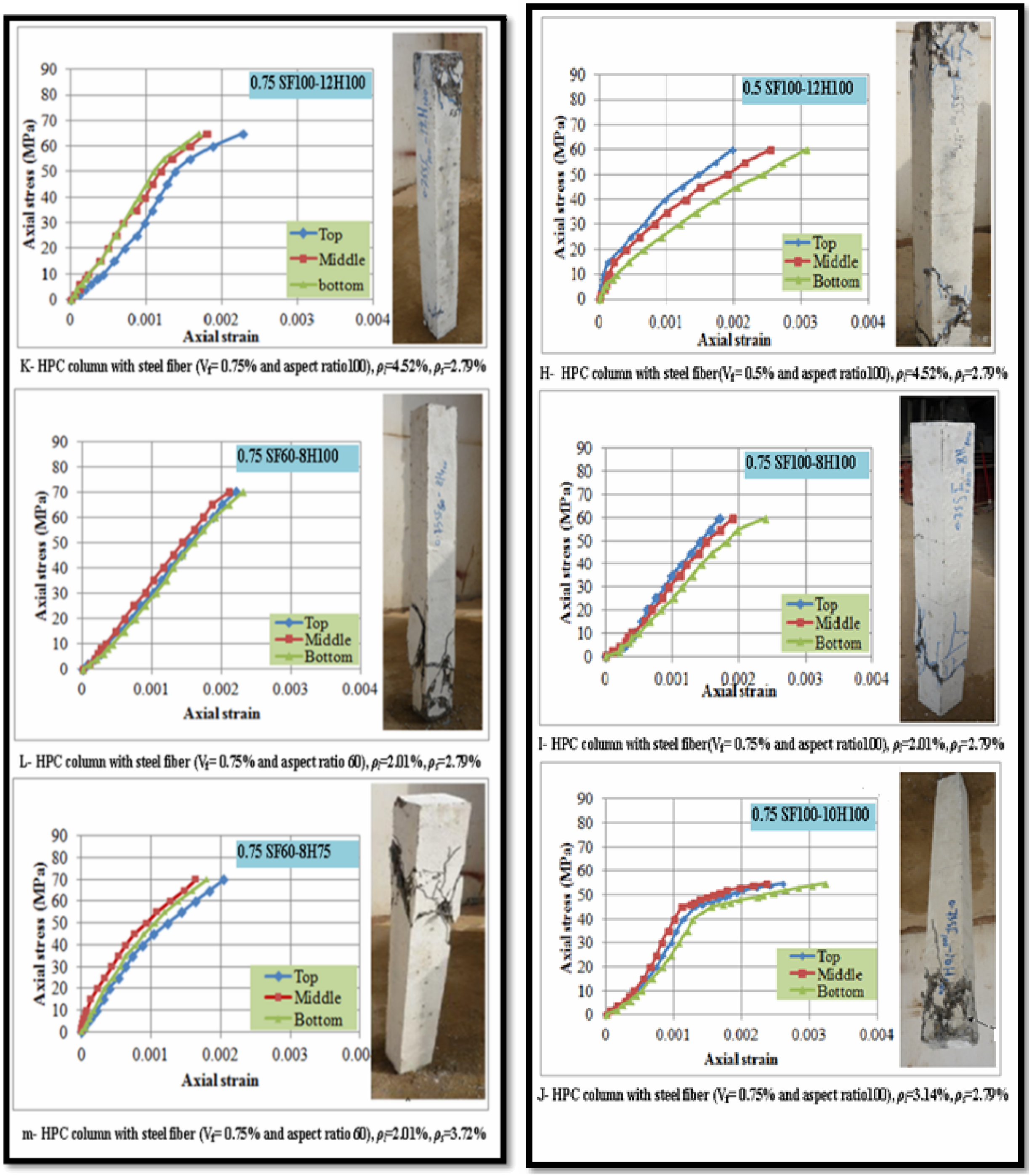

Figure-12. Stress-strain relationship for various column specimens.

Observed failure modes of column specimens

During the test of column specimens, it can be observed that most column specimens showed very similar behavior at early loading stages. The first set of cracks appeared on column faces at approximately 70% of the failure load, flexural and diagonal cracks occurred on the column side. The cracks width increased before the peak load was reached. Generally, three different modes of failure were observed:

a) Inclined cracks started near the ends (either top or bottom) increased in width with loading and extended longitudinally along the column causing the failure of column specimen.

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b) Cracks started near the edge horizontally and become wider as the load increased and then cause crushing of concrete.
c) Bulging of the column at location along its length followed by buckling of longitudinal reinforcement, especially for columns with volumetric ratio of longitudinal reinforcement ρ_l = 2.01%, as shown in Figure-13 and Figure-14.

Figure-13. Failure mode of NSC with and without fiber and (Vf = 0.5% and L/d = 100) with 2.79% of ρ_s and 2.01% of ρ_l.

Figure-14. Failure mode of HPC columns (Vf = 0.75% and L/d = 100) with 2.79% of ρs and different ratio of ρ_l.

Column ductility

Ductility is the ability of material to withstand plastic deformation without rupture bend-ability and crush-ability. The pressure exerted by the transverse reinforcement can only be applied to the concrete core area where the confining stress has fully developed due to 3D arching action, as shown in Figure-15, this concept of an effectively confined concrete area was developed by Sheikh and Uzumeri [18]. It was calculated by applying a reduction factor to the nominal area of confined concrete delimited by the perimeter of the hoops in cylindrical columns or outer ties in prismatic columns. This reduction factor (K_e) represents the ratio of the smallest effectively confined concrete area, midway between two layers of transverse bars, to the nominal area of confined concrete, and it is designated as the confinement effectiveness coefficient. This coefficient is determined as following for prismatic columns [19]:

$$Ke = \frac{\left[1-\frac{\sum w_i^2}{6c_x c_y}\right]\left[1-\frac{s'}{2c_x}\right]\left[1-\frac{s'}{2c_y}\right]}{1-\rho_c}\ (prismatic\ columns) \tag{1}$$

Where

Σwi^2 = sum of the squares of the clear horizontal spacings (wi) between the longitudinal steel bars along the perimeter of the cross-section.
s'= clear vertical spacing between two layers of transverse bars of diameter dh.
C = lateral dimension of the column core (cx and CY apply to rectangular sections).
ρ_c = area ratio of the longitudinal reinforcement to the column core section.

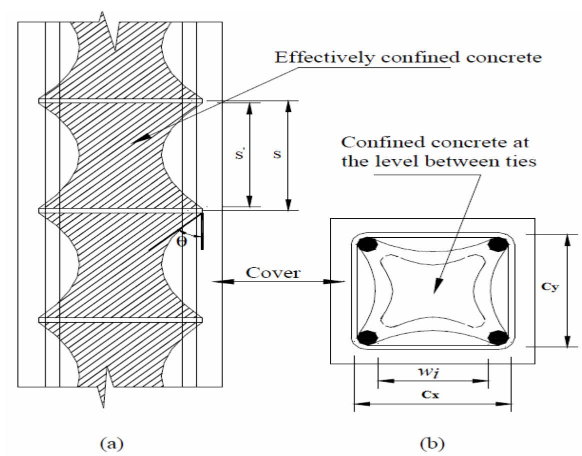

Figure-15. Illustration of arching action in confined concrete columns.

Since testing of columns was performed using a load control procedure, it was not possible to trace the response of the specimens beyond the peak load. For this reason, the ductility of the tested columns was computed using the procedure developed by Foster and Attard [20]. For non-fiber tied columns, it was suggested that the relationship between ductility and the confinement parameter ($k_e \rho_s f_{yt}/ f'_c$) can be given by [21]:

$$I_{10} = 1.9 \ln (1000\, k_e\, \rho_s\, f_{yt} / f'c) \tag{2}$$

Where

k_e = the confinement effectiveness coefficient, as calculated by Equation(1).

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ρ_s = the lateral reinforcement volumetric ratio, (%)
f_{yt} = the yield strength of the ties, (MPa)
F'_c = cylinder compressive strength, (MPa)

Noting that for square or circular columns the confinement parameter [21] is related to the confining stress by $f_l = 0.5\, \rho_s f_{yt}$, then Equation (2) can be written as [22]:

$$I_{10} = 1.9\ln\left(1000 \times 2k_e f_l / f'c\right) \tag{3}$$

For columns with both ties and fibers, Foster hypothesized that the confinement due to each component can be assumed to give a total confining stress as:

$$I_{10} = 1.9\ln\left[1000\left(\left(\frac{2k_e f_l}{f'_c}\right) + \left(\frac{2f_{lf}}{f'_c}\right)\right)\right] \tag{4}$$

Where

K_e = confinement effectiveness coefficient calculated using Equation (1).
f_l = lateral confinement stress in concrete core provided by the tie steel can be computed as: $f_l = 2\, A_{st}\, f_{yt} / d_c\, s$
A_{st} = area of the tie steel (mm^2)
f_{yt} = yield strength of tie steel (MPa)
d_c = cross - sectional dimension of column core measured center to center of transverse reinforcement,(mm)
S = spacing between ties or spiral, (mm)
ρ_s = lateral reinforcement volumetric ratio, (%)
$f_{l\,f}$ = lateral confinement stress provided by steel fibers computed as: $f_{lf} = \frac{3}{8}\alpha_f \rho_f \tau_b$
α_f: fiber aspect ratio = (l_f / d_f), where l_f and d_f are the fiber effective length and diameter, respectively.

ρ_f = volumetric ratio of fibers, calculated as following: $\rho_f = V_f * \gamma_c / \gamma_s$
V_f = volume fraction of fiber, (%), γ_c= density of the concrete, (kg/m^3), γ_s = density of the steel fibers, (kg/m^3), τ_b: fiber-concrete bond shear strength, in (MPa), calculated as following: $\tau_b = 0.6(f_c)^{2/3}$

An attempt has been made by Al-Shamma [23] to separate the effect of tie reinforcement contribution from steel fibers contribution in Eq. (4). This has been made by separating the equation into two terms and adding them algebraically, as shown below:

$$(I_{10})_{Tie} = 1.9\ln\left(1000\frac{\times 2k_e f_l}{f'_c}\right) \tag{5}$$

$$(I_{10})_{Fibers} = 1.9\ln\left(1000 \times \frac{2f_{lf}}{f'_c}\right) \tag{6}$$

$$I_{10} = (I_{10})_{Tie} + (I_{10})_{Fibers} \tag{7}$$

$$I_{10} = 1.9\left[\ln\left(1000\frac{\times 2k_e f_l}{f'_c}\right) + \ln\left(1000 \times \frac{2f_{lf}}{f'_c}\right)\right] \tag{8}$$

The ductility index I_{10} for various specimens tested in this investigation was computed using Equation (4) and Equation (8). The results are presented in Table-5 and Figure-16. It can be seen that the ductility index I_{10} calculated using Equation (8) increases significantly for steel fiber NSC and HPC columns, while for non fibrous NSC and HPC columns the values of I_{10} calculated using Equation(4) are the same as that calculated by Equation (8). The results also show that I_{10} calculated by Equation (8), increase significantly for steel fiber HPC and NSC columns in comparison with non fibrous HPC and NSC columns.

Table-5. Predicted ductility index I_{10} for tested column specimens.

Column symbols	**f'c*** (MPa)	**Confinement effectiveness coefficient**	**Confining stress applied by ties**	**Confining stress applied by fibers**	**Ductility index**	
		Ke	f_L (MPa)	f_{Lf}**(MPa)**	I_{10}	
		Eq. (1)			**Eq.(8)**	**Eq.(4)**
NF-8N100	29.390	0.0936	7.6096	-	7.3747	7.374713
0.5Sf100-8N100	41.465	0.0936	7.6096	0.4120	12.4007	7.587712
NF-8H100	48.150	0.0936	7.6096	-	6.4367	6.436746
NF-10H100	48.150	0.1011	7.1767	-	6.4712	6.471199
NF-12H100	48.150	0.1085	7.0231	-	6.5637	6.563693
0.5sf100-8H100	52.730	0.0936	7.6096	0.4837	11.7918	7.248347
0.5sf100-10H100	52.730	0.1011	7.1767	0.4837	11.8263	7.268946
0.5sf100-12H100	52.730	0.1085	7.0231	0.4837	11.9188	7.324986
0.75sf100-8H100	56.019	0.0936	7.6096	0.7553	12.4089	7.522249
0.75sf100-10H100	56.019	0.1011	7.1767	0.7553	12.4434	7.539052
0.75sf100-12H100	56.019	0.1085	7.0231	0.7553	12.5359	7.584935
0.75sf60-8H100	63.800	0.0936	7.6096	0.4943	11.1089	6.903033
0.75sf60-8H50	63.800	0.3603	15.2191	0.4943	14.9860	9.943141
0.75sf60-8H75	63.800	0.2053	10.1461	0.4943	13.1472	8.344831

*cylinder compressive strength (150×300) at age 28 days.

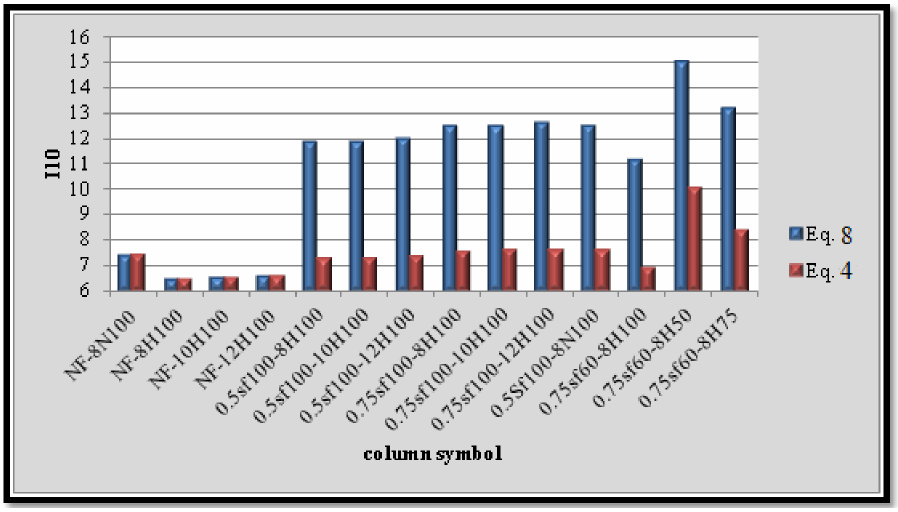

Figure-16. Comparison between values of $I_{10\ calculated}$ by Equation (4) and Equation (8) for the tested column specimens.

CONCLUSIONS

Considerable experimental work has been made to study the behavior of high performance fiber reinforced short concrete columns with different aspect ratios and volume fractions of steel fibers with different volumetric ratios of longitudinal and lateral reinforcement. From the experimental results presented in this research, the following conclusions can be drawn:

a) The addition of fibers to NSC and HPC columns increases the maximum strength. The percentage increase in peak strength at constant steel fiber volume fraction is slightly decreased with the increase in fiber aspect ratio. The percentage increase for HPC columns containing 0.75% steel fibers is about 42% and 21% for specimens with short fibers (aspect ratio 60)and long fibers(aspect ratio 100), respectively.
b) The value of peak strength of fiber reinforced columns over non fibrous columns in the case of HPC is comparatively less marked compared to the NSC columns with same fiber content and aspect ratio. This suggests that the HPC requires more fibers to acquire the same percentage increase in strength and ductility as that provided by normal strength fibrous concrete.
c) The volumetric ratio of longitudinal and transverse reinforcement (ties) slightly affects the peak strength of columns.
d) At the same applied load, non fibrous and fibrous HPC columns have lower deformability relative to non fibrous and fibrous NSC columns; this means that higher strength concrete columns require more confinement than normal strength concrete columns.
e) The deformability of steel fiber high performance concrete columns after concrete cracking increases as the fibers aspect ratio is increased.
f) The stress-strain relationship extends beyond the peak load for HPC columns containing long steel fiber (aspect ratio 100) with volume fraction 0.75% and reinforced with longitudinal bars with diameters 10 mm and 12 mm (volumetric ratio of longitudinal reinforcement 3.14% and 4.52%).
g) The volumetric ratio of lateral reinforcement slightly affects the ascending part of the stress-strain relationship in HPC.

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