EFFECT OF STEEL FIBERS ADDITION ON THE BEHAVIOR OF HIGH STRENGTH CONCRETE CIRCULAR SHORT COLUMNS

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ABSTRACT

This paper presents the effects of the addition of steel fibers on the behavior of concentrically loaded reinforced concrete circular short columns. An experimental investigation into the behavior of 24 short reinforced concrete columns with and without steel fibers was carried out. The columns had a circular section (200 mm diameter and 900 mm long). Test variables include concrete strength, spacing of spiral reinforcement, and inclusion of steel fibers. The axial stress and axial strains were obtained and used to evaluate the effects of the presence of steel fibers. It was found that the addition of steel fibers slightly improves the load carrying capacity of the tested columns whereas it significantly enhances the ductility of these specimens. Test results also indicated that for the same confinement parameter $\rho_s f_{sy}/f'_c$, specimens with steel fibers has higher strength enhancement compared to specimens without steel fibers. For confinement parameter of 0.1 the strength enhancement equal 1.7 for specimens with fibers while it is 1.4 for nonfibrous specimens. An empirical expression is proposed and used to predict the confined strength of columns. It was shown that the proposed formula predicts the load carrying capacity of the tested columns reasonably.

Keywords: short columns; compressive tests; confined concrete; high strength concrete; steel fibers.

تأثير إضافة ألياف الفولاذ على سلوك الأعمدة الخرسانية الدائرية عالية المقاومة و القصيرة

الخلاصة

يبين هذا البحث تأثير إضافة مادة ألياف الفولاذ على سلوك الأعمدة الخرسانية عالية المقاومة الدائرية و القصيرة و المفحوصة تحت تأثير أحمال انضغاط محورية. تضمن الجزء العملي من البحث تحضير وفحص 24 أنموذجاً لأعمدة خرسانية مسلحة دائرية المقطع بقطر 200 ملم و طول 900 ملم تحتوي نصفها على مادة ألياف الفولاذ. تشمل المتغيرات مقاومة الخرسانة للإنضغاط و المسافة الشاقولية لتوزيع حديد التسليح الحلزوني و وجود ألياف الفولاذ ضمن الخرسانة. أظهرت نتائج الفحص بأن ألياف الفولاذ المضافة حسنت بدرجة قليلة مقاومة التحمل القصوى للأعمدة في حين أن المطيلية ازدادت بشكل كبير مقارنة بالأعمدة غير الحاوية على ألياف الفولاذ. وكذلك وجد عند تثبيت معامل الحصر فأن تحسين المقاومه للأعمده الحاويه على الياف الفولاذ أعلى من تحسين المقاومه للأعمده غير الحاويه على الألياف. حيث وجد أن تحسين المقاومة بلغ 1,7 للأعمدة الحاوية على ألياف الفولاذ بينما بلغ 1,4 للأعمدة غير الحاوية على ألياف الفولاذ عندما يكون معامل الحصر مساوياً إلى 0,1. و كذلك فقد تم إقتراح علاقة تجريبية للتنبأ بالمقاومة القصوى لخرسانة الأعمدة المحصورة. و قد وجد أن المعادلة المقترحة تعطي نتائج جيدة مقارنة مع النتائج المختبرية.

INTRODUCTION

High strength concrete has been commonly used in reinforced concrete structures and particularly in columns. It leads to a reduction in column size due to higher load carrying capacity. It is well known that the mechanical behavior of high strength concrete is brittle under compression relative to lower strength concrete. Thus, confinement, ductility and deformability features of high strength concrete columns are important, especially under seismic loading conditions. Ductility of high strength reinforced concrete columns can be improved by effectively confining column core. It is widely accepted that concrete confinement can be provided by using small spacing of lateral ties and also adding steel fibers to concrete. The use of steel fiber composites in reinforced concrete structures has become very popular and found effective [Tokgoz, 2009].

High strength concrete with steel fibers does not fail even after high strain values under compression, thus the member behaves a ductile manner. Moreover, the addition of steel fibers into high strength concrete column not only improves ductility but also prevents premature cover spalling [Ganesan and Murthy 1990, Hsu et al. 1995, Foster and Attard 2001, and Foster 2001].

Ganesan and Murthy (1990) have presented an experimental investigation on the strength and behavior of reinforced concrete columns with and without steel fibers subjected to monotonic axial compression. The effect of steel fibers on the strength, confinement and ductility of reinforced concrete columns have been given, and an analytical model to describe the stress–strain behavior of confined steel fiber concrete has been proposed.

Hsu et al. (1995) have tested 14 square section slender high strength reinforced concrete columns with and without steel fibers to examine the experimental and theoretical load–deflection behavior of the columns under biaxially eccentric load. They found that the addition of steel fibers improved the ductility and confinement of high strength concrete columns. They also indicated that steel fibers do not affect the load carrying capacity of the columns significantly.

Foster and Attard (2001) have tested plain and steel fiber high strength concrete columns to examine the effects of steel fibers on strength and ductility. They have reported that cover spalling of the columns can be effectively prevented by adding steel fibers into the concrete, and the use of steel fibers improves the strength and ductility of high-strength concrete columns.

Foster (2001) has investigated the effect of steel fibers on the cover spalling and ductility of concrete columns. It has been concluded that steel fibers prevent premature cover spalling and improve ductility of high strength concrete columns. A model was developed to determine the quantity of steel fibers to ensure reasonable level of ductility.

Lima Junior and Giongo (2004) have investigated the experimental behavior of concentrically loaded steel fiber high strength concrete columns confined with low ratios of rectangular ties. They reported that steel fibers improve ductility and confinement and prevent premature concrete cover spalling of columns.

Campione et al. (2010) have tested 16 short, confined, reinforced concrete columns with and without steel fibers. The specimens were tested to failure at different strain rates under two loading schemes: concentric compression and eccentric compression with constant eccentricity. They concluded that the presence of steel fibers delay the spalling of concrete cover and increase the strain capacity and ductility; the eccentricity of the applied axial load cause substantial variation in the peak load, ultimate strength, and failure modes.

COLUMN DUCTILITY

Ductility is the ability of columns to deform without significant loss of strength. Since testing of columns was performed using load control procedure it was not possible to trace the response of the specimens after the peak load. For this reason the ductility of the tested columns were computed using the procedure developed by Foster and Attard (2001).

Foster and Attard (2001) used a technique involving an I_{10} ductility index which allows a comparison between the performance of fiber and non-fiber reinforced concrete columns. The method is based on the definition of flexural toughness as set out in ASTM C1018-97, where the level of ductility is defined using the I_{10} ductility index. The I_{10} factor "energy ductility" can be found by dividing the area under the load- deflection curve at 5.5 times the yield deflection by the area under the curve at yield deflection, as defined in Fig. 1.

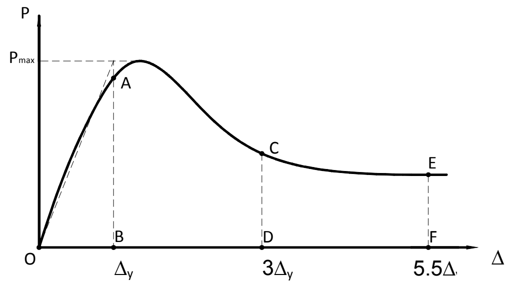


Fig. 1 Determination of column ductility [Foster 2001]

A number of authors [Shaikh and Uzumeri 1980, Martinez et al. 1984, Mander et al. 1988, and Razvi and Saatcioglu 1994] have indicated that ductility is a function of the confinement parameter $k_e\rho_s f_{sy}/f'_c$, where ρ_s is the lateral reinforcement volumetric ratio, f_{sy} is the yield strength of the spiral reinforcement, f'_c is the concrete compressive strength, and k_e is a confinement effectiveness coefficient, which represents the efficiency of tie reinforcement arrangement. Martinez et al. (1984) shows that for columns confined by spirals k_e can be calculated using the following expression:

$$k_e = \frac{\left(d_s - \frac{s'}{2}\right)}{1-\rho_{cc}} \qquad (1)$$

where

d_s : Diameter of the column core measured between the spiral centers,

s' : Clear vertical spacing between spiral bars,

ρ_{cc} : Ratio of longitudinal reinforcement area to concrete core area.

Foster and Attard (2001) suggested that the relationship between ductility and the confinement parameter $k_e\rho_s f_{sy}/f'_c$ for non-fiber spiral columns can be given by:

$$I_{10} = 1.9\,ln(1000 k_e \rho_s f_{sy}/f'_c) \qquad (2)$$

where

ρ_s : Volumetric ratio of spiral reinforcement,

f_{sy} : Yield strength of spiral steel.

Noting that the confinement parameter is related to the confining stress f_l which determined as $0.5\rho_s f_{sy}$, then Eq. (2) can be written as:

$$I_{10} = 1.9\,ln(1000 \times 2k_e f_l/f'_c) \qquad (3)$$

For columns with both spirals and steel fibers, Foster and Attard (2001) supposed that the confinement due to each component can be summed to give a total confining stress. That is:

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

Where f_{lf} is the lateral confinement stress provided by the steel fibers. Marti et al. (1999) assumed a constant bond shear τ_b along the length of the embedded fiber and found that the confining pressure applied to the section by the fibers is:

$$f_{lf} = \frac{3}{8}\alpha_f \rho_f \tau_b \qquad (5)$$

where

α_f : Fiber aspect ratio = l_f/d_f , where l_f and d_f is the fiber effective length and diameter, respectively,

τ_b : Fiber-concrete bond shear strength = $0.6(f'_c)^{2/3}$ as an approximation suggested by Marti et al. (1999), in MPa.

EXPERIMENTAL PROGRAM

Specimens

A total of 24 short column specimens had an overall height of 900 mm with circular cross-section of 200 mm diameter were loaded concentrically. Half of the specimens were cast with the inclusion of steel fibers and the other half was without steel fibers. The concrete mix of fiber-reinforced samples contains 0.5% by volume (approx. 1.67% by weight) of end hooked steel fibers.

The column specimens were divided into three main groups depending on the concrete strength; low, normal and high strength concrete. Each group was subdivided into two subgroups depending on the inclusion of fibers. More details are given in reference [Al-Shamma, 2011].

Four columns were tested for each subgroup. The first column was plain without reinforcement, is used to establish the properties of unconfined concrete and to account for the difference between concrete strength in the column and that based on testing of concrete cylinder. The second column was reinforced with six longitudinal steel bars of 10 mm diameter only. The third column was reinforced with six longitudinal steel bars of 10 mm diameter and spirals of 6 mm diameter with spiral pitch equals to 50 mm. The fourth column was reinforced with the same reinforcement as the third column except for spiral pitch which was 100 mm. Fig. 2 shows reinforcement details of columns with spiral steel.

In all specimens the ratio of the gross area of the column cross section A_g to the core area A_c measured to the outside of the lateral reinforcement was 1.3. The columns were prepared for vertical casting using local PVC tubes cut to match the column height and then cut vertically to facilitate the removing of the mold later. Three clamps were used to support the mold throughout the casting process. The base of molds was constructed by cutting plywood panel into square pieces with dimensions 300×300 mm. Then, the fabricated steel cages were placed in the molds. The assembled steel cages for the tested columns are shown in Plate 1.

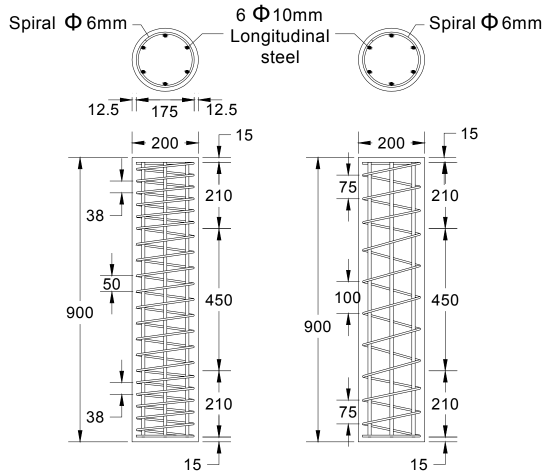


Fig. 2 Reinforcement details of spiral columns

Column specimens are identified with a series of numbers and letters. For example, column specimen LSC-F-100 is a column with low strength concrete, fiber reinforced, and 100 mm pitch of spiral reinforcement. Table 1 shows various properties of longitudinal and transverse reinforcement of the tested columns. The concrete strength f'_c given in Table 1 represents standard cylinder compressive strength obtained at the day of test of the specimens, ρ_l, s, and ρ_s are respectively longitudinal steel ratio, spacing and volumetric ratio of spiral steel. The volumetric ratio of spiral reinforcement, ρ_s varied between 0.6% and 1.3%.

Table 1 Details of test specimens

Column ID	Volumetric Ratio of Steel Fibers	Concrete Compressive Strength	Longitudinal Reinforcement (Deformed Bars)		Lateral Reinforcement (Plain Bars)		
	ρ_f (%)	f'_c (MPa)	No. & Size (mm)	(%)ρ_l	Size (mm)	Pitch, s (mm)	ρ_s (%)
LSC-F-50	0.5	28.71	10φ6	1.5	6	50	1.3
LSC-F-100					6	100	0.6
LSC-F-L					--	--	--
LSC-F-P			--	--	--	--	--
NSC-F-50		41.36	10φ6	1.5	6	50	1.3
NSC-F-100					6	100	0.6
NSC-F-L					--	--	--
NSC-F-P			--	--	--	--	--
HSC-F-50		64.46	10φ6	1.5	6	50	1.3
HSC-F-100					6	100	0.6
HSC-F-L					--	--	--
HSC-F-P			--	--	--	--	--
LSC-N-50	--	27.68	10φ6	1.5	6	50	1.3
LSC-N-100					6	100	0.6
LSC-N-L					--	--	--
LSC-N-P			--	--	--	--	--
NSC-N-50		40.59	10φ6	1.5	6	50	1.3
NSC-N-100					6	100	0.6
NSC-N-L					--	--	--
NSC-N-P			--	--	--	--	--
HSC-N-50		59.13	10φ6	1.5	6	50	1.3
HSC-N-100					6	100	0.6
HSC-N-L					--	--	--
HSC-N-P			--	--	--	--	--

Table 2 Longitudinal and transverse reinforcing bar properties*

Bar Size (mm)	Tensile Strength (MPa)	Yield Strength (MPa)	Elongation (%)
6	418	317	23.2
10	590	495	15.3

* Test results for each type of reinforcement are average values of three coupons.

Plate 1 Assembled reinforcement cages for columns

Material Properties

The materials consisted of Ordinary portland cement (Type I) produced by United Cement Company (TASLUJA), maximum size of 10 mm local gravel brought from Al-Nibaee region, well graded natural sand obtained from Al-Ukhaider region, tap water for mixing and curing, silica fume produced by Sika Corporation and TufFlow SP603 superplasticizer admixture to maintain good workability of the mix. 10 mm diameter deformed steel bars and 6 mm diameter plain steel bars are used for longitudinal and spiral reinforcement, respectively. The main data obtained for testing bars are shown in Table 2.

Fibrous concrete was obtained by adding hooked end steel fibers in fresh concrete. The steel fibers are Dramix® ZC 50/50 type and manufactured by Bekaert Corporation (Plate 2). The fiber had a length of 50 mm, a diameter of 0.5 mm, and a tensile strength (declared by the manufacturer) of 1000 MPa (Fig. 3). The fiber percentage adopted was 0.5% by volume of concrete, corresponding to 40 kg/m^3.

Plate 2 Dramix® ZC 50/50

Fig. 3 Steel fiber dimensions end hooked steel fibers

Three target concrete strengths, 21 MPa, 42 MPa, and 63 MPa, are considered in this study and obtained using several laboratory trial batches. Low water-cement ratio w/c and high cement content were used to achieve the strength level required. The corresponding water to cementitious material ratios w/c was 0.5, 0.4, and 0.3 for LSC, NSC and HSC respectively. Different mix designs for the three target concrete strengths were used for six batches of concrete. Table 3 shows details of concrete mix design for each of the targeted strengths.

Material	Target Strengths (MPa)		
	21	42	63
	Mix1	Mix2	Mix3
Cement (Type I) (kg/m^3)	400	450	500
Silica Fume (kg/m^3)	-	-	40
Sand (kg/m^3)	600	680	700
Gravel (kg/m^3)	1200	1150	1100
Water (kg/m^3)	200	180	162
Superplasticizer (SP603) (L/100 kg)*	-	1.5	2.5
w/c	0.5	0.4	0.3

* Liter per 100 kg of cementitious materials (Cement + Silica Fume).

Table 3 Details of the concrete mix design

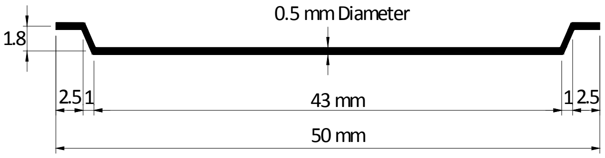


Table 4 Mechanical properties of hardened concrete

Mix design ID	Volumetric Ratio of Fibers (%)	f'_c (MPa)	f_{ct} (MPa)	E_c (GPa)	f_r (MPa)
LSC (Mix 1)	0	26.37	4	22.34	4
NSC (Mix 2)		38.66	5.30	28.94	4.71
HSC (Mix 3)		56.32	6.65	36.91	5.26
LSC (Mix 1)	0.5	27.35	4.59	24.53	5.4
NSC (Mix 2)		39.93	5.77	31.83	7.1
HSC (Mix 3)		61.39	7.14	36.17	7.52

Twelve standard cylinders 150×300 mm and two standard prisms 100×100×500 mm with each batch were tested to determine the mechanical properties of hardened concrete. In Table 4 values of concrete compressive strength f'_c, modulus of elasticity E_c, splitting tensile strength f_{ct}, and modulus of rupture f_r, at an age of 28 days are given for both ordinary and fibrous concrete. The stress-strain curves of the reinforcing bars and the concretes are shown in Figs. 4 and 5, respectively.

Columns Casting

A total of six batches of concrete were used to cast the columns. All columns were cast vertically to simulate typical construction practice of columns. Concrete was discharged in the columns directly from 0.1 m^3 capacity horizontal pan mixer in approximately 2 or 3 lifts. The mixing time was about 5-8 minutes; the dry constituents were well mixed for about 2 minutes to insure proper distribution of the ingredients and to disperse any agglomeration of the fine materials in the pan. For batches with superplasticizers, the added water was premixed with superplasticizer and mixed for 3 minutes. Batches contain steel fibers, the fibers were added carefully (to prevent balling) and mixed for another 3 minutes. An electric table vibrator was used to consolidate the concrete and to remove air bubbles at each of the lifts. The column formworks were stripped and de-molded approximately two days after casting and covered by fresh water inside the curing tanks at ambient temperature in the laboratory for about four weeks. After that the specimens were removed from the water and left to dry under laboratory conditions until testing.

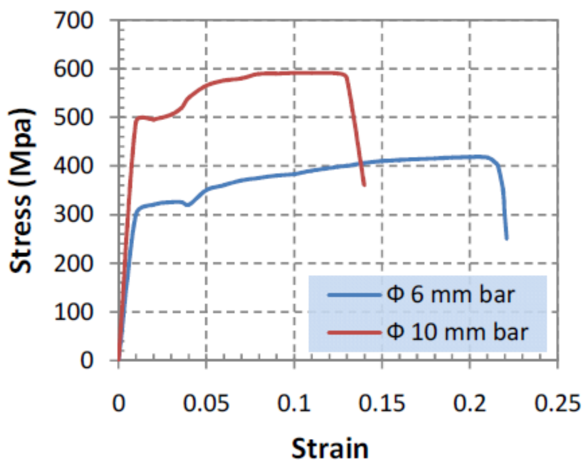


Fig. 4 stress-strain curves of reinforcing bars

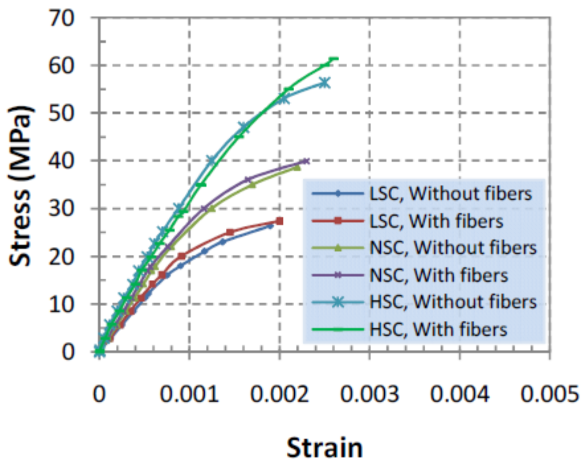


Fig. 5 stress-strain curves of concrete

Fig. 6 Test set-up, distribution of demec points and location of dial gauge

Instrumentation and Testing Procedure

The columns gross axial shortening was measured using dial gauge of 0.01mm/div. sensitivity, located at the bottom surface of testing machine. Also, Demec points are used to measure the surface strains of concrete. They were mounted along the concrete surfaces within the test region of the tested samples. Two demec gauge points were mounted at spacing of 100 mm at the column mid-height along the column vertical axis to measure longitudinal compressive strains at two opposite directions of the column.

A 3000 kN capacity compression testing machine (MFL SYSTEME) located at Structures Laboratory of Al-Mustansiriya University, was used to apply the compression load monotonically until failure. Details of test set-up, distribution of demec points and location of dial gauge are shown in Fig. 6.

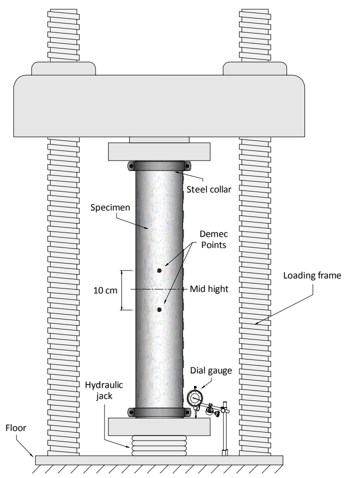


Plate 3 Failure mode of fibrous and nonfibrous columns

FAILURE MODE OF COLUMN SPECIMENS

During testing of column specimens up to failure, it was observed that failure of unconfined columns is by the formation of the longitudinal cracks at certain location followed by cracks which propagated to other sides. It was also noticed that for unconfined high strength concrete specimens a sudden type of failure occurs, unless steel fibers were used. When columns cast with steel fibers the failure was ductile, small crack width and much less cover spalling was observed. It was also observed that, addition of steel fibers to high strength concrete columns resulted in small crack width, and crack propagation was gradual and premature concrete cover spalling of the columns was delayed. It could be noticed that the addition of steel fibers resulted in bulging of the central part of the columns relative to concrete columns without steel fibers, as shown in Plate 3.

For fiber reinforced columns, it can be seen that the steel fibers played a significant role in preventing concrete cover from early spalling. This is because steel fibers crosses the cracks and preventing from further widening and allowing other cracks to form at other locations. It seems that the presence of steel fibers is highly effective at load stages after peak where it was also observed then that the column bulging continues to increase without loss of column integrity which resulted in increasing the tensile stresses in the spirals leading to rupture of some of these spirals, as shown in Plate 4.

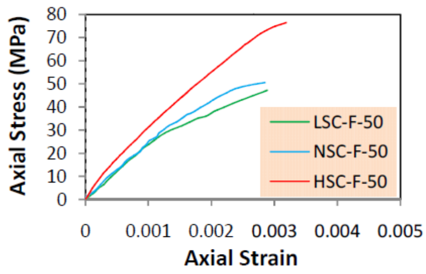
STRESS-STRAIN RELATIONSHIP OF THE TESTED COLUMNS

The stress strain curve of the column specimens consists of two main parts; ascending and descending.

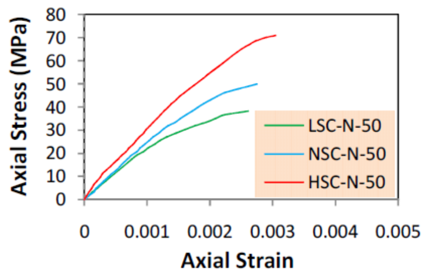
Plate 4 Spiral rupture occurs before complete spalling of concrete cover (HSC-F-100)

divided into two distinct stages of behavior. The first stage is characterized by an approximately linear relationship between stress and strain. This behavior continued until the initiation of longitudinal cracks. The second stage begins when the concrete cover starts to spall out. At this stage the stiffness of the column decreased as indicated by the reduced slope of the stress-axial strain curves. The final stage occurred when the longitudinal reinforcement starts to bend and the core of concrete carries all the axial loading while the concrete cover is completely spall out at failure regions.

In Fig. 7 the relation between stress and strain of columns for various types of concrete are presented. A stiffer response in the pre-cracking stage is noted for columns with higher concrete compressive strength. Also, the ascending part of the curves for HSC specimens is nearly linear in comparison to both NSC and LSC specimens which shows a nonlinear response.



(a) Reinforced with 0.5% steel fibers



(b) Non-fiber concrete

Fig. 7 Effect of concrete strength on the stress-strain curves of column specimens

TEST RESULTS AND DISCUSSION

Axial Load Capacity of the Tested Columns

Fig. 8 shows the ultimate load capacity for all the tested columns. For fiber reinforced concrete columns, no significant variation in the peak load was observed compared to that of ordinary concrete columns.

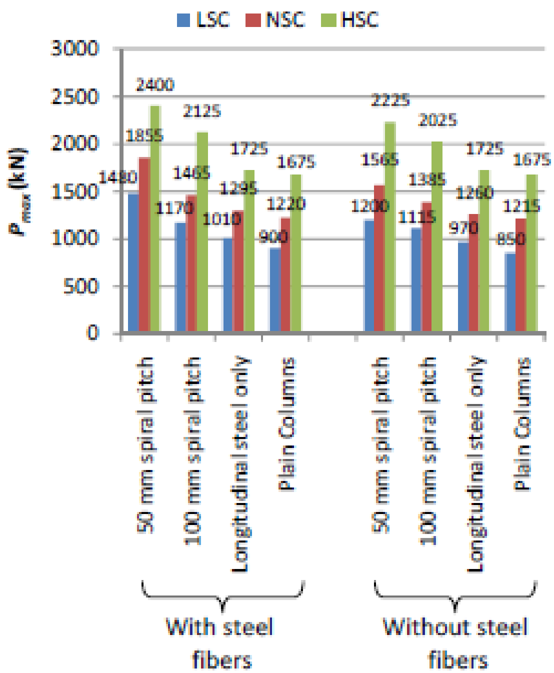


Fig. 8 Ultimate experimental load capacities P_{max} for tested columns

From Fig. 8 it can be seen that for nonfibrous columns, reducing the spacing of spiral steel from 100 mm to 50 mm increased the load capacity by about 8%, 13%, and 10% for LSC, NSC, and HSC respectively. Whereas for fibrous columns the load capacity increased by about 27%for LSC and NSC and 13% for HSC. It was also observed that the addition of steel fibers to concrete does not enhance the load capacity of unconfined columns, while little increase in load capacity is observed for confined columns. For columns with 50 mm spacing spirals, the increase in capacity is about 23%, 19%, and 8% for LSC, NSC, and HSC respectively. Test results showed that increasing concrete strength from 26 MPa to 60 MPa the load capacity of columns with 50 mm spacing spirals increased by about 85% or nonfibrous columns and 62% for fibrous columns.

Confined Concrete Strength by Spiral Reinforcement

Kim (2007) have indicated that the confined concrete strength f'_{cc} of columns can be determined by subtracting the calculated load carried by longitudinal reinforcement from the measured maximum load Pmax of the column, divided by the area of concrete core as follows:

$$f'_{cc} = \frac{P_{max} - (A_s f_y)}{A_c - A_s} \qquad (6)$$

where
f'_{cc} : Confined concrete strength of column,
P_{max}: Measured maximum load of column,
A_s : Total area of longitudinal reinforcement,
f_y : Yield strength of longitudinal steel,
A_c : Area of core measured to outside diameter of spiral steel.

In this paper Eq. (6) is used to calculate the confined concrete strength of the tested columns. The ratio of the confined strength of the tested reinforced concrete columns to the unconfined strength of similar column f'_{cc}/f'_{co} represents the strength enhancement of concrete. This enhancement of concrete strength was shown to be affected by the confinement parameter $\rho_s f_{sy}/f'_c$ as illustrated in Fig. 9.

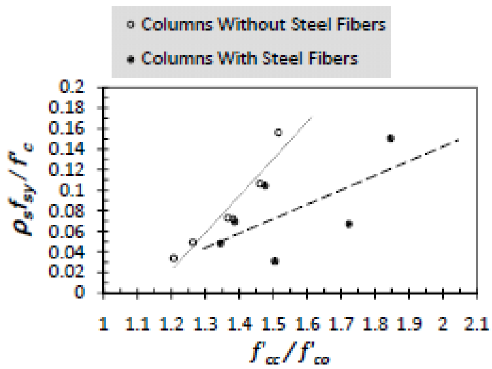


Fig. 9 Relationship between confinement parameter $\rho_s f_{sy}/f'_c$ and strength enhancement f'_{cc}/f'_{co}

Fig. 9 indicates that by increasing the confinement parameter $\rho_s f_{sy}/f'_c$ the strength enhancement f'_{cc}/f'_{co} considerably increased. Therefore, if the same level of strength enhancement f'_{cc}/f'_{co} is desired, columns with higher concrete strength should be reinforced with more lateral steel than those with lower concrete strength. Thus, the lateral confinement pressure required for high-strength concrete may be significantly higher than that for normal strength concrete. Saatcioglu and Razvi (1998) have also indicated that this requirement is usually met by using higher grades of spiral steel rather than increasing the volumetric ratio of reinforcement to avoid congestion of the column cage.

Fig. 9 also indicates that for the same confinement parameter $\rho_s f_{sy}/f'_c$ specimens with steel fibers posses higher strength enhancement compared to specimens without steel fibers. For confinement parameter of 0.1 the strength enhancement equal 1.7 for specimens with fibers while it is 1.4 for nonfibrous specimens. (i.e. an increase of about 20% relative to nonfibrous concrete columns).

An empirical expression was developed for the confined concrete strength f'_{cc} using regression analysis of 42 columns based on current test data and data of other researchers [Razvi 1995, Li 1994]. The obtained expression is a function of unconfined concrete strength and confining stresses provided by the spiral steel and steel fibers, as follows:

$$f'_{cc} = f'_{co} + 6.9(f_l + f_{lf})^{0.8} \qquad (7)$$

where

f'_{cc} : Axial compressive strength of confined concrete,

f'_{co} : Axial compressive strength of unconfined concrete,

f_l : Lateral confining stress provided by the spiral steel = $0.5\rho_s f_{sy}$,

f_{lf} : Lateral confining stress provided by steel fibers, Eq. (5).

The numbers 6.9 and 0.8 shown in Eq. (7) are constants obtained from nonlinear multiple stepwise regression analysis using Data Fit 9.0 software.

Comparison between the strength obtained using Eq. (7) and the experimental strength values based on Kim's Eq. (6) are presented in Fig. 10. The results indicate good agreement between the analytical and experimental strength values.

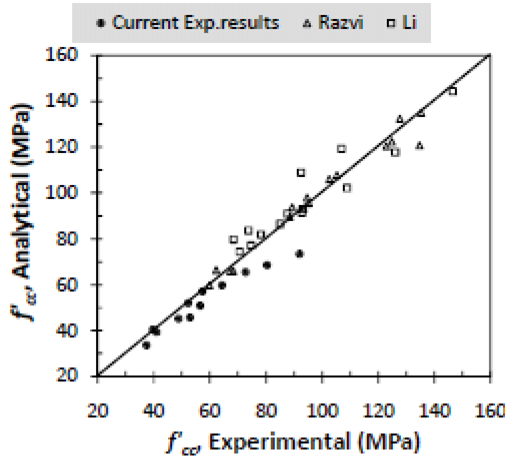


Fig. 10 Comparison between analytical confined strength using Eq. (7) and experimental strength values using Eq. (6)

Ductility of the Tested Columns

Since testing of columns was performed using load control procedure it was not possible to trace the response of the specimens after the peak load. For this reason the ductility of the tested columns were computed using the procedure developed by Foster and Attard (2001). It was observed that the addition of steel fibers significantly enhances the ductility of the tested columns. The tested columns with steel fibers were found to be able to deform lateral without loss of strength and the final failure was precipitated by large numbers of cracks and high lateral deformation.

For the above reasons an attempt has been made to separate the effect of spiral reinforcement contribution from steel fibers contribution in the equation developed by Foster and Attard (2001). This has been made by separating the equation into two parts and add them algebraically, as shown below:

$$(I_{10})_{Spiral} = 1.9\,ln(1000 \times 2k_e f_l/f'_c) \qquad (8)$$

$$(I_{10})_{Fibers} = 1.9\,ln(1000 \times 2f_{lf}/f'_c) \qquad (9)$$

$$I_{10} = (I_{10})_{Spiral} + (I_{10})_{Fibers} \qquad (10)$$

$$I_{10} = 1.9\left[ln(1000\times 2k_e f_l/f'_c) + ln(1000\times 2f_{lf}/f'_c)\right] \quad (11)$$

The ductility index I_{10} for various specimens was computed using Eq. (11) and the results are presented in Fig. 11.

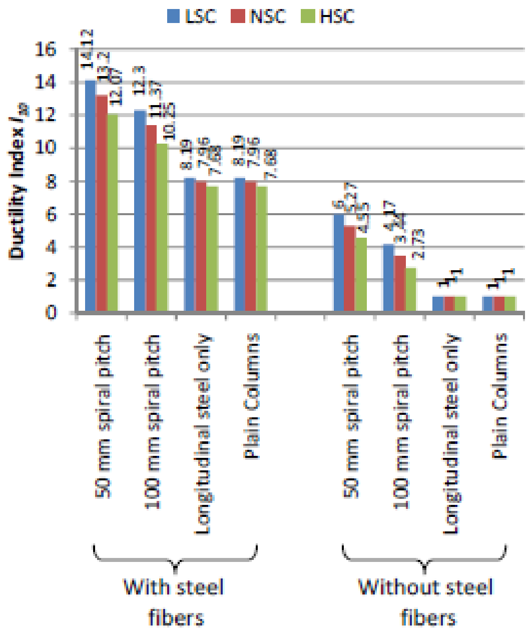


Fig. 11 Computed ductility Index I_{10} for columns using Eq. (9)

For columns confined by 50 mm spacing spirals, the addition of 0.5% by volume of steel fibers increases the ductility index I_{10} by about 135%, 150%, and 165% for LSC, NSC, and HSC respectively. Fig. 11 shows that for nonfibrous columns, increasing the confining stress by reducing the pitch of spirals from 100 mm to 50 mm cause an increase in column ductility index I_{10} by about 44%, 53%, and 67% for LSC, NSC, and HSC respectively. Test results showed that increasing concrete strength from 26 MPa to 60 MPa for columns with 50 mm spacing spirals resulted in reduction of ductility index I_{10} by about 24% for nonfibrous columns and 15% for fibrous columns.

CONCLUSIONS

1. Experimental tests show that columns without confinement failed shortly after the formation of vertical cracks, while columns confined with closely spaced spirals or having steel fibers shows a gradual bulging before failure takes place.
2. It was observed that the addition of steel fibers to concrete does not enhance the load capacity of unconfined columns, while little increase in load capacity is observed for confined columns. For columns with 50 mm spacing spirals, the increase in capacity is about 23%, 19%, and 8% for LSC, NSC, and HSC respectively.
3. The addition of steel fibers significantly enhances the ductility of the tested columns. For columns confined by 50 mm spacing spirals, the addition of 0.5% by volume of steel fibers increases the ductility index I_{10} by about 135%, 150%, and 165% for LSC, NSC, and HSC respectively.
4. Test results indicate that for the same confinement parameter $\rho_s f_{sy}/f'_c$, specimens with steel fibers has higher strength enhancement f'_{cc}/f'_{co} compared to specimens without steel fibers. For confinement parameter of 0.1 the strength enhancement equal 1.7 for specimens with fibers while it is 1.4 for nonfibrous specimens.
5. The proposed strength of confined columns given in Eq. (7), could predict the confined strength (f'_{cc}) of short concrete columns with good degree of accuracy.

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NOTATION

A_c	Area of the column core: the area enclosed inside the perimeter of the centre lines of spirals or ties
A_g	Gross sectional area of the column
d_f	Fiber diameter
d_s	Diameter of the column core measured between the spiral centers
E_c	Modules of elasticity of concrete
f'_c	Cylinder compressive strength of

	concrete
f'_{cc}	Axial compressive strength of confined concrete
f'_{co}	Plain concrete strength in a member
f_{ct}	Splitting tensile strength of concrete
f_l	Lateral confinement stress provided by the spiral steel
f_{lf}	Lateral confinement stress provided by the steel fibers
f_r	Modulus of rupture of concrete
f_{sy}	Yield strength of spiral steel.
f_y	Yield strength of longitudinal reinforcement
I_{10}	Ductility index
k_e	Confinement effectiveness coefficient ($k_e \leq 1$)
l_f	Fiber effective length
P_{max}	Measured maximum load of column
s	Spacing or pitch of spiral
s'	Clear spacing between spiral or hoop bars
α_f	Fiber aspect ratio = l_f/d_f
Δ_y	Deflection at yield stress
Δ_{85}	Deflection at 85 % of the maximum axial load on the descending branch of the axial load - deflection curve
ρ_{cc}	Ratio of longitudinal reinforcement area to concrete core area
ρ_f	Volumetric ratio of steel fibers
ρ_l	Ratio of longitudinal reinforcement area to concrete gross area
ρ_s	Volumetric ratio of spiral reinforcement
τ_b	Fiber-Concrete bond shear strength