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Eccentrically Loaded Slender Columns Strengthened by High-Strength Fiber Reinforced Concrete Jackets.

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ECCENTRICALLY LOADED SLENDER COLUMNS STRENGTHENED BY HIGH-STRENGTH FIBER REINFORCED CONCRETE JACKETS

دراسة الأعمدة الرفيعة المحملة لا مركزيا والمقواة بقمصان من الخرسانة عالية المقاومة المحتوية على ألياف

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خلاصة:

يتناول هذا البحث دراسة لسلوك الأعمدة الرفيعة المصنعة من الخرسانة عالية المقاومة والمقواة بقمصان مصنعة من الخرسانة عالية المقاومة والتي كانت مقاومة الضغط المميزة لها تساوي ٨٩,٣ ميجاباسكال كما أنها تحتوي على ألياف من الصلب وذلك تحت تأثير أحمال لامركزية. وقد تم استخدام نتائج التجارب المعملية فى اختبار مدى إمكانية تطبيق متطلبات التصميم الواردة في الكود المصري والكود الأوربي والكود الأمريكي وذلك على الأعمدة التي تم تقويتها. وقد تضمن برنامج الاختبارات المعملية عدد ١١ عمود لها نسبة نحافة قبل تقويتها تتراوح من ١٧,٣ إلى ٦٩,٣. وأهم المتغيرات التي تم دراستها هي نسبة النحافة للأعمدة بعد تقويتها بالقمصان ونوع ألياف الصلب المستخدمة وقيمة اللامركزية للحمل ونسبة التسليح العرضي للأعمدة. وقد أوضحت النتائج أن الأعمدة النحيفة والقصيرة التي بها شروخ والمحملة لا مركزيا والتي تم تقويتها بقمصان من الخرسانة عالية المقاومة التي تحتوي على ألياف من الصلب نسبتها ١% بالحجم يمكن التعامل معها على أنها أعمدة جديدة. استخدام ألياف الصلب مع الخرسانة عالية المقاومة للقمصان تقلل إلى حد كبير الانهيار المبكر للغطاء الخرساني وتزيد الحمل الأقصى بالمقارنة بمثيلتها التي تم تقويتها بقمصان من الخرسانة عالية المقاومة التي لا تحتوي على ألياف. وقد تبين أن الطرق الثلاثة المختلفة لتصميم الأعمدة النحيفة و المستخدمة فى الكود المصري والكود الأوربي الثاني والكود الأمريكي يمكن استخدامها بأمان لتصميم الأعمدة النحيفة المقواة بقمصان من الخرسانة عالية المقاومة التي تحتوي على ألياف.

ABSTRACT

The behavior of eccentrically loaded cracked slender High-Strength Concrete (HSC) columns strengthened by HSC jackets contain steel fibers and have cube compressive strength equal to 89.3 MPa has been studied experimentally. The results of these tests have been used to examine the applicability of the design requirements of slender columns of the Eurocode 2 (EC-2), the Egyptian code (ECCS-2001) and ACI 318-99 building code, when applied to the strengthened columns. The testing program included 11 columns with slenderness ratios before strengthening ranged between 17.3 and 69.3. The main variables considered in this study were the slenderness ratio of the strengthened columns, the type of the steel fibers, the end eccentricity of the applied axial load and the transverse reinforcement ratio of the HSC jackets. The results showed that, eccentrically loaded slender and short cracked columns strengthened by HSC jackets contain 1% steel fibers (by volume) can be treated as new integral columns. Using a steel fibers into the HSC mix of the jackets reduces to a great extent the early cover spalling of the tested columns and increases the ultimate loads comparing with the same columns strengthened with HSC jackets without steel fibers. The three different methods for design of slender columns required by the ECCS-2001, EC-2 and ACI 318-99 building code can be safely used for design of slender columns strengthened by HSC jackets contain steel fibers.

Keywords: Column Jacket; High-Strength Concrete; Slenderness ratio; Codes; Strengthening.

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INTRODUCTION

Many methods and techniques are used for repair and strengthening of defective reinforced concrete columns [1-4]. Strengthening of defective columns by reinforced concrete jackets has been a widely used method of repair. This method is based on considering the jacket as an addition to the cross-section of the original column and also as an increased confinement to the interior section. Many factors involved in this method of repair among them the geometrical properties of the jacket section, the shape and amount of reinforcement of the jacket, the strength of the concrete and the reinforcement, the frictional characteristic of the surface of the original column and the load level in the original column before applying of the concrete jacket. The reported experimental tests of columns strengthened by reinforced concrete jackets in the available literature is limited [5, 6]. These tests have been conducted on the strength and behavior of short columns with reinforced concrete jackets constructed with Normal-Strength Concrete (NSC).

The use of High-Strength Concrete (HSC), which is growing rapidly, become attractive for tall building structures as well as for earthquake-resistant structures where a reduction of the mass and column sizes is of great importance [7]. The reduction in column sizes results in economic benefits. However, this leads also to an increase in the slenderness of columns and buckling of these columns may result in excessive cracking or defects. It should be noted that, although the design of reinforced concrete columns for buckling is by now a relatively well researched subject, a variety of design methods are in use and the design requirements of the Egyptian Code (ECCS-2001) [8], Eurocode 2 (EC-2) [9] and ACI 318-99 building code [10] differ markedly. In addition, the design equations given in theses codes for the design of slender columns contain empirical relationships derived from tests using NSC.

The maximum potentiality of HSC can not be realized fully in structures due to its relative brittleness and lack of ductility. This drawback can be overcome by addition of steel fibers in HSC mix. Steel fibers have some advantages over conventional stirrups [11-15]. First, the fibers are randomly distributed through the volume of the concrete at much closer spacing than can be obtained by the smallest reinforcing rods. Secondly, the first-crack tensile strength and the ultimate tensile strength are increased by the steel fibers. The first-crack strength is increased by the crack arrest mechanism of closely spaced wires. The ultimate tensile strength is increased because additional energy is needed to pull or to strip the fibers out of the concrete, if the fibers were not broken off during initial cracking.

The main objective of this investigation is to study the behavior of eccentrically loaded HSC cracked slender columns strengthened by HSC jackets contain steel fibers. The results of these tests were used to examine the applicability of the design methods used by the ECCS-2001, EC-2 and ACI 318-99 code when applied to the strengthened columns.

EXPERIMENTAL PROGRAM

Details of Original Columns

Eleven long and short HSC columns were prepared to represent the original columns. The present investigation is focused on jackets surrounding the full perimeter of the original columns which are normally used for repair of interior columns. All the original columns had the same square cross-section (100 mm x 100 mm) as shown in Fig. 1. The geometrical dimensions and details of reinforcement of the original columns are given in Table 1. According to the height of the columns H_{col}, the tested specimens were divided into four groups. It should be noted that according to the ECCS-2001 [8], columns can be considered as slender if the slenderness ratio λ_i is more than 50, where ($\lambda_i = H_{col}/i$) and i is the radius of gyration of the cross-section. According to the EC-2 [9] the second order effects should be taken into consideration for the reinforced concrete columns with λ_i is more than 25. According to the ACI 318-99 building code [10], columns can be considered as slender if λ_i is more than 24.

For the original columns, four different slenderness ratios λ_i were tested (λ_i = 17.3, 26.0, 52.0 and 69.3). The corresponding effective column heights (H_{col}) were 500 mm, 750 mm, 1500 mm and 2000 mm, respectively. The original columns were loaded with small end eccentricity (e) equal to 30 mm and the corresponding e/t ratio is 0.3. The same longitudinal reinforcement ratio was used for all the original columns (4 ϕ 10 with $\rho_l = 3.14\%$). Two different transverse reinforcement ratios were used for all the tested columns ($\rho_v = (A_v/bs) =$ 0.70% and 1.13%). This was in the form of stirrups of bar diameter (ϕ6) with spacing between stirrups (s=80 mm and 50 mm). It should be noted that, for earthquake resistant design, the ECCS-2001 requires that s should be the least of: 8 times the smaller longitudinal bar diameter (8x10mm= 80mm); 24 times the diameter of the stirrups (24x6 mm =144 mm); half the length of the shorter column dimensions (0.5x100 =50 mm) or 150 mm.

Details of Concrete Jackets

All the columns with the jackets had the same square cross-sections (170 mm x 170 mm). The longitudinal dimensions and the details of reinforcement of the conerete jackets are shown in Fig. 1 and Table 2. The same longitudinal reinforcement ratio was used for all the concrete jackets (4 ϕ 12 with ρ_l =1.57%). Two transverse reinforcement ratios (ρ_v=0.49% and 0.78%) were tested. These were in the form of closed stirrups of bar diameter (ϕ6) but with two different spacing between stirrups (s=50 mm and 80 mm, respectively). Two different types of steel fibers with different aspect ratios were tested (Type A and type B), but with the same fiber percent by volume (V_p =1.0%). In addition, two different conditions of end eccentricity were included in the test program (small eccentricity with e/t=0.33 and big eccentricity with e/t=0.70).

It should be noted that, after 28 days of casting, all the original columns were loaded with small eccentricity equal to 30 mm up to the initiation of failure load. It should be noted that the original columns failed by increasing the compression strain until crushing of the concrete in the compression zone before yielding of the steel [15]. After loading of the original columns, the specimens were enveloped by concrete jackets to obtain models for strengthened columns. For all the tested specimens, the surfaces of the original columns were roughened by cavities at a spacing ranging between 50-100 mm with average diameter of 20 mm and average depth of 10 mm.

The tests of the specimens were conducted in the loading frame and the test setup is shown in Fig. 2. The lateral deflections at the midheight of the columns were monitored by two dial gauges 0.01 mm accuracy. Electrical strain gauges of 120 Ohm resistance and 10 mm and 5 mm length were bonded to the longitudinal reinforcement and the transverse reinforcement within the central 100 mm of the specimens. The concrete strains in the midheight of the test units were measured using mechanical strain gauge over the central 200 mm gauge length.

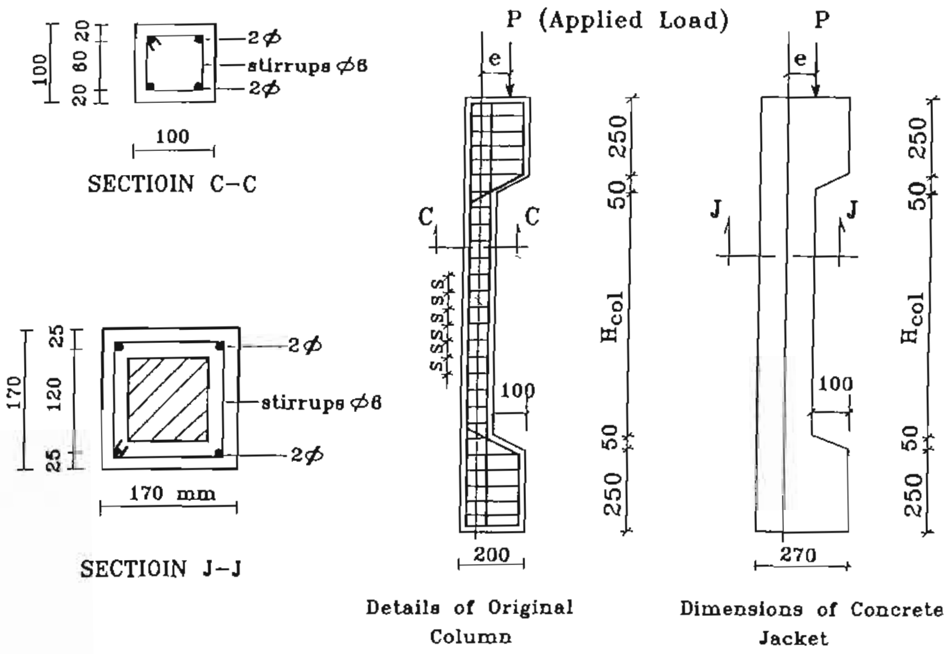

Fig. (1): Details of test specimens.

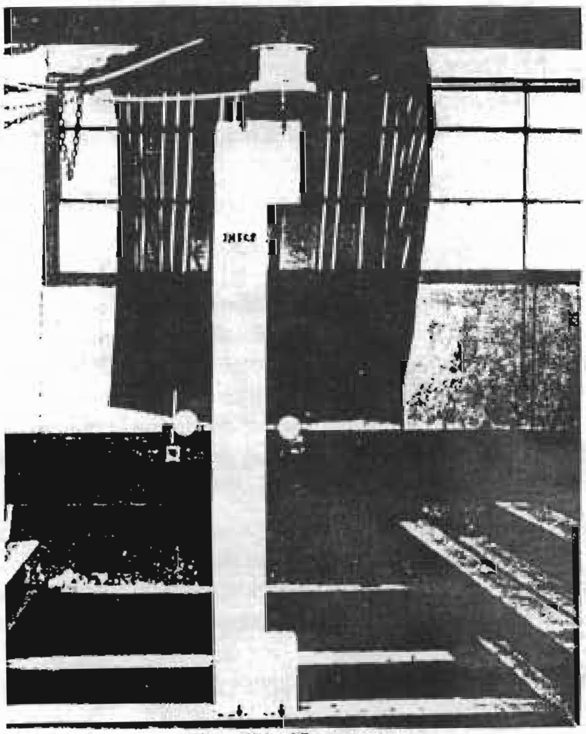

Fig. (2): Test setup.

Table (1): Details of the original columns.

Group	Column	f_{cu} (MPa)	f_c' (MPa)	H_{col} (mm)	λ_i	e (mm)	e/t	Longit. bars	Spacing of stirrups S (mm)	Stirrups Ratio (ρ_v %)
1	**JHFC1**	85.1	75.8	500	17.3	30	0.30	4ϕ10	80	0.70
	JHFC2	85.1	75.8	500	17.3	30	0.30	4ϕ10	50	1.13
	JHFC3	85.1	75.8	500	17.3	30	0.30	4ϕ10	80	0.70
2	**JHFC4**	85.1	75.8	750	26.0	30	0.30	4ϕ10	50	1.13
	JHFC5	85.1	75.8	750	26.0	30	0.30	4ϕ10	50	1.13
3	**JHFC6**	85.1	75.8	1500	52.0	30	0.30	4ϕ10	80	0.70
	JHFC7	85.1	75.8	1500	52.0	30	0.30	4ϕ10	50	1.13
	JHFC8	85.1	75.8	1500	52.0	30	0.30	4ϕ10	80	0.70
4	**JHFC9**	85.1	75.8	2000	69.3	30	0.30	4ϕ10	50	0.70
	JHFC10	85.1	75.8	2000	69.3	30	0.30	4ϕ10	80	1.13
	JHFC11	85.1	75.8	2000	69.3	30	0.30	4ϕ10	50	0.70

Table (2): Details of the strengthened columns and the concrete jackets.

Group	Column	f_{cu} (MPa)	f_c' (MPa)	H_{col} (mm)	λ_{ij}	λ_{bj}	e (mm)	e/t	Long. Bars	Stirrups Ratio (ρ_{vj}%)	Fiber percent (V_f%)	Fiber Type
1	**JHFC1**	85.1	75.8	500	10.2	2.9	119.0	0.70	4ϕ12	0.49	-	-
	JHFC2	89.3	80.1	500	10.2	2.9	56.1	0.33	4ϕ12	0.49	1.0	A
	JHFC3	89.3	80.1	500	10.2	2.9	119.0	0.70	4ϕ12	0.49	1.0	A
2	**JHFC4**	89.3	80.1	750	15.3	4.4	119.0	0.70	4ϕ12	0.49	1.0	A
	JHFC5	89.3	80.1	750	15.3	4.4	119.0	0.70	4ϕ12	0.49	1.0	B
3	**JHFC6**	89.3	80.1	1500	30.6	8.8	119.0	0.70	4ϕ12	0.49	1.0	A
	JHFC7	89.3	80.1	1500	30.6	8.8	119.0	0.70	4ϕ12	0.78	1.0	A
	JHFC8	89.3	80.1	1500	30.6	8.8	119.0	0.70	4ϕ12	0.49	1.0	B
4	**JHFC9**	85.1	75.8	2000	40.8	11.8	56.1	0.33	4ϕ12	0.49	-	-
	JHFC10	89.3	80.1	2000	40.8	11.8	119.0	0.70	4ϕ12	0.49	1.0	A
	JHFC11	89.3	80.1	2000	40.8	11.8	56.1	0.33	4ϕ12	0.49	1.0	A

Materials

The HSC concrete mix used in this study was the same for the original columns and the concrete jackets. The mix includes ordinary portland cement in conjunction with 13 mm diameter gravel. The fine aggregate was natural sand with a fineness modulus of 2.80. The mix proportions by weight per 1.0 m^3 was: Cement 475 kg, Gravel 1180 kg, Sand 580 kg and water 125 kg. Light gray locally produced silica fume with a specific gravity of 2.15 was used with 15 percent by weight of cement. A superplasticizer with 3 percent by weight of cement was added and enough mixing time was allowed to produce uniform mix of concrete without any segregation.

The actual characteristic compressive strength f_{cu} of the HSC mix after 28 days based on an average of three cube specimens (150x150x150 mm) without and with steel fibers are given in Table 2, in addition to the cylinder compressive strength f_c' (based on the average of three cylinders 150x300 mm).

As expected, the addition of steel fibers to the concrete mix considerably enhances the flexural strength f_r and the splitting cylinder tensile strength f_{sp}. The splitting cylinder tensile strength f_{sp} (based on 150x300 mm cylinder) and the flexural strength f_r (based on 100x100x500 mm beams) of the HSC mix without steel fibers were equal to 4.85 MPa and 8.35 MPa, respectively. For the HSC mix containing steel fiber percent by volume equal to 1%, the splitting cylinder tensile strength was equal to 7.35 MPa, while the flexural strength was equal to 10.8 MPa.

The original columns and the HSC jackets were cast in forms made of wood with smooth hard varnish surfaces. The forms were removed after 48 hours from casting and columns were moistured continuously with water for 26 days. Then, the specimens were painted white in order to facilitate crack observation. The specimens were tested after 28 to 30 days from the day of casting.

The main longitudinal reinforcement of each of the original columns and the concrete jackets consists of four deformed high-grade bars with diameter 10 mm or 12 mm and the yield strength of these bars f_y were equal to 397.0 and 382.0 MPa, respectively, while the maximum strength were 581.0 and 602.0 MPa, respectively. The recorded strain at the initiation of yield of these bars were 1890 and 1820 $\mu\varepsilon$, respectively. The transverse reinforcement comprised 6 mm diameter mild steel bars with yield strength equal to 262.0 MPa and maximum strength equal to 376.0 MPa.

The steel fibers used in the present study was hooked-ended straight fiber of a type available in the Egyptian market (called HAREX type). Two types of steel fibers (here will be referred to as Type A and Type B) were used. The length of the fibers were 31.5 and 24.3, for Type A and Type B, respectively, while the equivalent diameters were equal to 0.64 mm and 0.76 mm, respectively. The aspect ratio of Type A and Type B were equal to 49.2 and 32.0, respectively. The minimum tensile strength of the fibers of Type A and Type B were 492 MPa and 534 MPa, respectively. Mixing and placing of steel fibers was according to the recommendations of ACI Committee 544 [16].

ANALYSIS OF TEST RESULTS

Failure Modes and Ultimate Loads

The recorded ultimate loads P_{ujexp} and the corresponding lateral deflections at midheight of the tested strengthened columns δ_{ujexp} are given in Table 3. Specimen JHFC2 was tested up to the maximum load of the loading frame and final failure of the specimen was not obtained. The recorded ultimate loads of the tested strengthened columns in this program were compared in Table 3 with the calculated ultimate loads of the original HSC columns (P_{uo}). It should be noted that, the total area of the cross-section of the strengthened column is 2.89 times the area of the original columns. It can be seen that, short columns strengthened using HSC jackets with and without steel fibers increased the ultimate load by more than three times the calculated ultimate load of the original column. Columns JHFC1, JHFC3, JHFC4 and JHFC5 showed an increase of the experimental ultimate load comparing with that of the original columns by 351%, 348%, 324% and 313%, respectively. This increase in the ultimate load was relatively reduced for the strengthened slender columns as can be seen for the specimens of group 3 and group 4 in Table 3.

The addition of steel fibers to the HSC mix of the jackets enhances the ultimate load of the tested specimens. Short column JHFC3 with steel fibers in the concrete jacket showed an increase in the experimental load over that of the column JHFC1 (without steel fibers) and also slender column JHFC11 showed an increase in the experimental load over that of the column JHFC9. Increasing the aspect ratio of the steel fibers slightly increased the ultimate load but did not affect the failure mode of the strengthened columns (as can be seen from the comparison between specimens JHFC4 and JHFC5 and also specimens JHFC8 and JHFC6).

Generally, all the tested short columns (group 1 and 2) failed at or near to the midheight while most of the slender columns (group 3 and 4) failed out of the middle third. The failure mode of the columns depends mainly on the eccentricity of the applied axial load. It should be noted that, the calculated balanced eccentricity (e_{bal}) of the tested strengthened columns was equal to 56.9 mm (assuming the design ultimate concrete strain equal to 0.003). The tested specimens with big eccentricity (e/t=0.70) failed by yielding of the longitudinal bars in the tension side of the jacketed columns, followed by a shift of the neutral axis toward the compression side until crushing of the concrete in the compression side of the section. This can be seen from the photographs of specimen JHFC6 in Fig. 3. In this case, the initial crack in the column is flexural and the cracking load was about 10%-15% of the ultimate load. As can be seen from the photographs, the cracking behavior of the column JHFC6 is similar to that of flexural cracks of beams (cracks in the tension side of the column). Slender specimens (JHFC9 and JHFC11 with λ_i=40.8) tested with small eccentricity (e/t=0.33) failed due to increasing the tensile steel strain at the midheight up to yielding of the longitudinal reinforcement before the compression strain reaching the crushing value.

For the strengthened specimen without steel fibers and tested with e/t=0.7 (JHFC1), the concrete cover, first, spalled off in the compression side and at later stages, the spalling of the cover extended to the side faces of the column as can be seen from Fig. 3. In contrast, for specimen JHFC3 tested with the same eccentricity and contain steel fibers of 1% by volume, the cover remained intact throughout the test, well beyond the peak load. The damage of the concrete jacket of the strengthened columns without steel fibers and tested with small eccentricity was considerably more severe than that of the columns subjected to big eccentricity. This can be clearly observed from the photographs of the failure modes shown in Fig. 3. It can be seen also from these photographs that, the concrete jacket of specimen JHFC11(which contains steel fibers) still intact at the maximum ultimate load while the jacket of specimen JHFC9 (without steel fibers) was severely damaged. In addition, buckling of the longitudinal bars in the compression zone of the tested short and long specimens that contained steel fibers was not observed in contrast to the similar specimens without steel fibers. This indicate that, the transverse reinforcement of columns required by the ECCS-2001 can be safely used for HSC jackets contain steel fibers of 1% by volume.

Recorded Steel Strains

The behavior of the tested specimens can be explained also from the readings of the strain recorded in the longitudinal reinforcement of the column jackets during the tests of some specimens as shown in Fig. 4. It should be noted that, the plotted recorded strains were only for the values up to the beginning of the yield of the longitudinal bars and after yielding, the strains increased with a very fast rate without a noticeable increase in the recorded applied ultimate load. For the specimens tested with big eccentricity (e/t=0.70), the tensile and compressive strains increased gradually but the tensile strains increased with a higher rate. It can be seen from Fig. 4 that, yielding of the longitudinal reinforcement of the jackets of these specimens occurred always sooner at low values of compressive strains in the compression zone. Increasing the slenderness ratio resulted in an increase in the recorded tensile strains in the longitudinal reinforcement as can be seen for specimen JHFC6 and specimen JHFC10.

It can be seen also from Fig. 4 that, for the short specimen JHFC2 tested with small eccentricity (e/t=0.33), the recorded compressive strain increased gradually with increasing the applied load up to crushing of the concrete in the compression zone, while the tensile steel strains was far from yielding. For the long specimen JHFC11, because of the additional moment due to buckling of the column, the yielding of the longitudinal reinforcement in the tension side reached at low levels of compressive strains.

Table (3): Summary of test results.

Column	V_f %	e/t	λ_{ij}	δ_{ujexp} (mm)	P_{ujexp} (kN)	P_{uo} (kN)	$\frac{P_{ujexp}}{P_{no}}$
JHFC1	0.0	0.70	10.2	2.82	419.4	125.7	3.51
JHFC2	1.0	0.33	10.2	> 2.85	> 750.0	316.0	> 2.37
JHFC3	1.0	0.70	10.2	3.31	436.9	125.7	3.48
JHFC4	1.0	0.70	15.3	3.14	407.8	125.7	3.24
JHFC5	1.0	0.70	15.3	3.32	393.2	125.7	3.13
JHFC6	1.0	0.70	30.6	4.47	408.7	125.7	3.25
JHFC7	1.0	0.70	30.6	4.69	348.6	125.7	2.77
JHFC8	1.0	0.70	30.6	4.80	326.2	125.7	2.60
JHFC9	0.0	0.33	40.8	4.15	694.2	316.0	2.20
JHFC10	1.0	0.70	40.8	6.53	305.8	125.7	2.43
JHFC11	1.0	0.33	40.8	4.96	734.0	316.0	2.32

Fig. (3): Photographs of the failure modes of some of the tested columns.

Axial Force-Lateral Deflection Relations

Figure 5 shows the relationship between the lateral deflections at midheight of some of the tested strengthened columns with the applied axial load. Generally, small lateral deflections were measured in the short columns with $\lambda_i = 10.2$ and 15.3, while the long columns with $\lambda_i = 30.6$ and 40.8 were subject to relatively large lateral deflections at the midheight of the column as shown in Fig. 5a and b. The value of the eccentricity of the applied axial load was the main factor affecting the lateral deflection of the specimens. It can be seen from Fig. 5a and b, that the columns tested with smaller eccentricity (JHFC11 with e/t=0.33) showed smaller values of lateral deflections compared with the same columns tested at bigger eccentricity (JHFC10 with e/t=0.70).

The columns of the higher load eccentricity showed greater deflections at ultimate load. At the end of the tests of these specimens (during the last 20 kN), it was observed that the specimens cracked and deformed significantly prior to failure. After reaching the maximum load, the columns continued to deform as an indication of a ductile behavior. The addition of steel fibers to the HSC jackets has a considerable effect on the load-lateral deflection curve. As shown in Table 3, for the different values of slenderness ratio, specimens contain steel fiber percent of 1% by volume (specimens JHFC3 and JHFC11) showed a considerable increase in the lateral deflection (especially at the ultimate load) comparing with the same specimens without steel fibers in the jacket (specimens JHFC1 and JHFC9). This indicates that, the existence of the steel fibers in the HSC mix of the column jackets considerably enhances the ductility of the strengthened columns.

The test program included two transverse reinforcement ratios of the HSC column jackets (ρ_{vj} =0.49% and ρ_{vj} =0.78 %), which were in the form of the same stirrup bar diameter (ϕ 6) but with two different stirrups spacing (s = 80 mm and 50 mm). The applied load-lateral deflection relationships for the specimen JHFC6 with transverse reinforcement ratio of the jacket ρ_{vj} equal to 0.49% was approximately the same as that of the similar specimen JHFC7 which have transverse reinforcement ratio of the jacket equal to 0.78%. This indicates that the transverse reinforcement ratio has negligible effect on the lateral deflection of the tested specimens. As shown in Table 3, the lateral deflections at ultimate loads (δ_{ujexp}) for the columns containing steel fibers show a little decrease comparing with that of the same columns without steel fibers.

The type of the steel fibers used with the HSC mix of the column jacket had negligible effect on the lateral deflection of the tested specimens. The applied load-lateral deflection curves of the same specimens but with different type of steel fibers were approximately similar (specimens JHFC4 and JHFC5 and specimens JHFC6 and JHFC8). However, some differences were observed in the lateral deflections at ultimate loads for columns with identical jackets but with different types of steel fibers. As can be seen from Table 3, short and long specimens with jackets containing steel fibers of Type A (aspect ratio=48.4) showed lateral deflections at ultimate loads (δ_{ujexp}) less than the same specimens but with jackets containing steel fibers of Type B (aspect ratio=32.1).

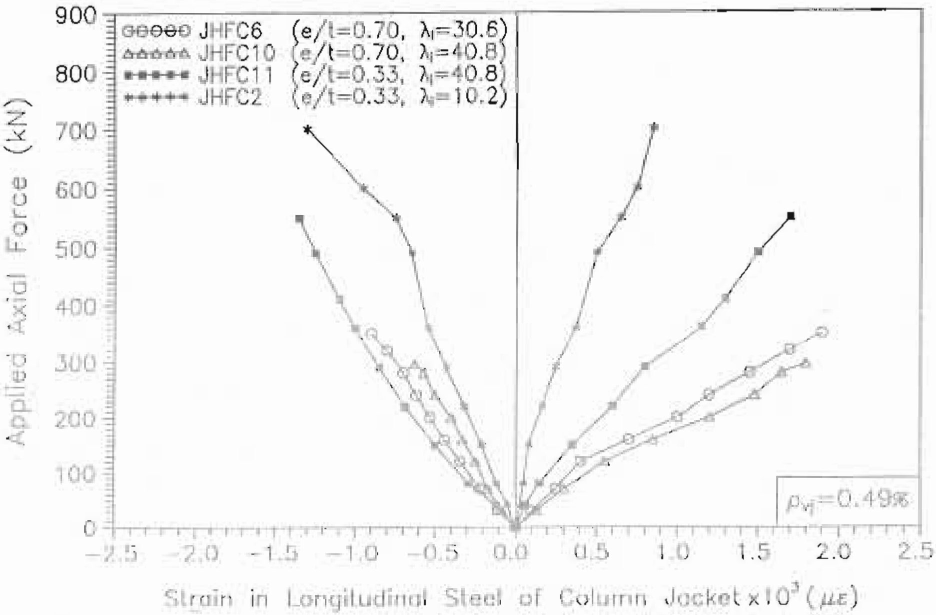

Fig. (4): Strains in the longitudinal bars of some of the tested specimens.

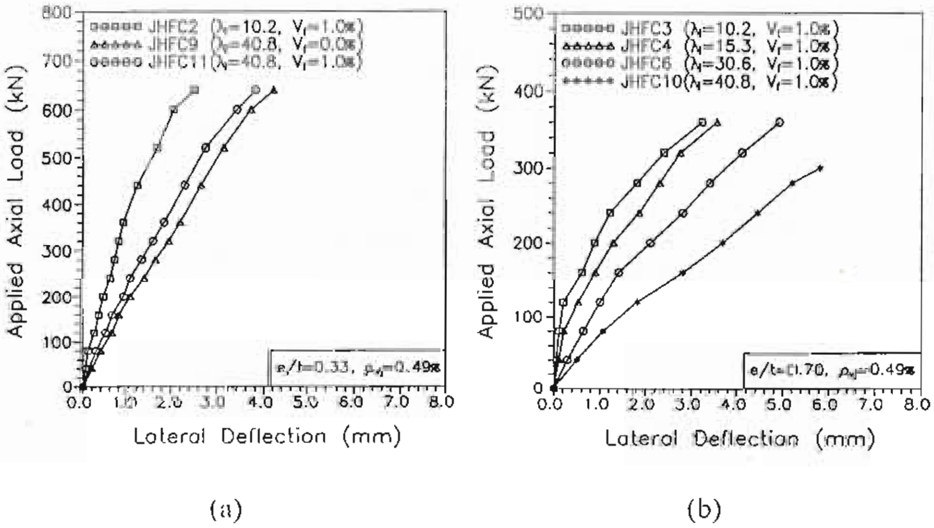

Fig. (5): Applied axial load-midheight lateral deflection relations.

CODES PROVISIONS FOR DESIGN OF SLENDER COLUMNS

ECCS-2001

According to this code, a braced rectangular column is designed as short column if

$$\lambda_b = H_e/b \quad \leq 15 \tag{1}$$

or

$$\lambda_i = H_e/i \quad \leq 50 \tag{2}$$

and

$$H_e = \beta . H_{col} \tag{3}$$

where H_e is the effective column buckling height, b is the column dimension perpendicular to the axis of bending, i is the radius of gyration of the column cross section (equal to 0.289 times the overall depth of rectangular columns), H_{col} is the unsupported height of the column from the top of floor to the bottom of the floor above and β is the effective length factor which depends on the end conditions of the column and can be determined as given in the code (for a braced frame $\beta \leq 1.0$).

If the column slenderness ratio exceeds this limit, the column will buckle prior to reaching its limit state of material failure. The effect of buckling can be taken in design by an additional moment (M_{add}) induced by the deflection of the column's buckled shape at the section being considered and can be calculated as follows:

$$M_{add} = P . \delta \tag{4}$$

where P is the applied ultimate axial load and δ is the induced deflection due to buckling which can be calculated from the following expression:

$$\delta = \frac{\lambda_b^2 . b}{2000} \tag{5}$$

The induced deflection δ can be calculated also from the following general form:

$$\delta = \frac{\lambda_i^2 . t'}{30000} \tag{6}$$

where t' is the column dimension in the direction of buckling. According to this code, for rectangular cross section λ_b should not be taken more than 30 or $\lambda_i \leq 100$.

EC-2

According to this code, isolated columns in non-sway structures need not be checked for second order effects (including geometrical imperfections) if the slenderness ratio (λ_i) is less than or equal to the value of (λ_{crit}) given by the following equation:

$$\lambda_{crit} = 25\,(2 - e_{o1}/e_{o2}) \tag{7}$$

where e_{o1} and e_{o2} are the actual eccentricity of the applied axial loads at the ends of the column (called the first order eccentricity) and it is assumed that $|e_{o1}| \leq |e_{o2}|$.

The second order effect due to buckling of the column can be calculated using the Model Column Method. This method can be applied for columns with $\lambda_i < 140$ and the first order eccentricity $e \geq 0.1$. According to this method, the second order eccentricity (δ) of such a column may be calculated as follows:

$$\delta = \frac{K_1 H_e^2}{10}\,(1/r) \tag{8}$$

and

$$K_1 = (\lambda_i/20 - 0.75) \qquad \text{for} \quad 15 \leq \lambda_i \leq 35 \tag{9.a}$$

$$K_1 = 1.0 \qquad \text{for} \quad \lambda_i > 35 \tag{9.b}$$

where the curvature ($1/r$) can be calculated from the following equation:

$$1/r = \frac{2.K_2.\varepsilon_{yd}}{0.9\,d} \tag{10}$$

where ε_{yd} is the design yield strain of steel reinforcement ($\varepsilon_{yd} = f_{yd}/E_s$) and d is the effective depth of the cross-section in the expected direction of stability failure. The coefficient K_2 in Eq. 10 takes account of the decrease of the curvature with increasing the axial force and is defined by the following equation:

$$K_2 = (P_{ud} - P_{sd})\,/\,(P_{ud} - P_b) \qquad \leq 1.0 \tag{11}$$

$$P_{ud} = 0.85 f_c'\,(A_c - A_s) + f_{yd}\,A_s \tag{12}$$

where P_{ud} is the design ultimate capacity of the section subjected to axial load only, P_{sd} is the actual design axial force and P_{bal} is the axial load which, when applied to a section, maximizes its ultimate moment capacity. For symmetrical reinforced rectangular sections, P_b may be taken as $(0.4\,f_c'\,A_c)$, where f_c' is the design cylinder compressive strength of concrete, f_{yd} is the design yield strength of longitudinal reinforcement, A_c is the total area of the column cross-section and A_s is the total area of the longitudinal reinforcement of the column. It will be always conservative to assume that K_2 equal to 1.0.

ACI 318-99

In nonsway frames it shall be permitted to ignore slenderness effects for compression members that satisfy:

$$\lambda_i = \left(\frac{k.H_{col}}{i}\right) \leq 34 - 12\left(\frac{M_1}{M_2}\right) \tag{13}$$

where k is the effective length factor which depends on the end restrains of the column and can be determined by means of the Jackson and Moreland Alignment Charts given in the code (for a braced frame $k \leq 1.0$). The ratio of the moments ($M1/M2$) at the two ends of the column in a braced frame will generally be taken between +0.5 and –0.5.

For design of slender columns subjected to concentric or eccentric axial load, this code recommends the Moment Magnifier method (MM method). This method can be applied for columns with $\lambda_i \leq 100$. Let the ultimate column load and the larger end moment, from a first-order elastic frame analysis, be P_u and $M_u = P_u \, . \, e$. It should be noted that, the design ultimate axial load according to the ACI code is given as follows:

$$P_u = \varphi P_n \tag{14}$$

where φ is the strength reduction factor which, for tied columns, varies linearly as the nominal axial load capacity of the column cross section, P_n , varies from P_a to zero where P_a is equal to the smaller of the balanced axial load P_{bal} or $(0.143 f_c' A_c)$. If P_n is greater than or equal to P_a, the factor φ should be taken equal to 0.7, while if P_n between P_a and zero, φ should be taken equal to (0.9- 0.2 P_n/P_a).

The load and the moment to be used in the design of the section are P_u and ($\delta_{ns} M_u$) where δ_{ns} is the Moment Magnification Factor which is given by the following equation:

$$\delta_{ns} = \frac{C_m}{1-(P_u/0.75P_c)} \geq 1.0 \tag{15}$$

in which, C_m is the equivalent moment factor and is given by the following expression:

$$C_m = 0.6 + 0.4\frac{M_1}{M_2} \geq 0.4 \tag{16}$$

and the elastic buckling load P_c is given as follows:

$$P_c = \frac{\pi^2 EI}{(k\,H_{col})^2} \tag{17}$$

where EI is the flexural rigidity of column section. The value of EI shall be taken equal to one of the values given by the following equations:

$$EI_1 = \frac{0.4 E_c I_g}{1+\beta_d} \tag{18}$$

$$EI_2 = \frac{0.2E_cI_g + E_sI_{se}}{1+\beta_d} \qquad (19)$$

The modulus of elasticity of concrete is given by the following equation:

$$E_{c1} = 4730\sqrt{f_c'} \qquad (20)$$

in which I_g is the moment of inertia of the gross concrete section about the centroidal axis ignoring the reinforcement, E_s is the elastic modulus of steel, I_{se} is the moment of inertia of the reinforcement about the centroidal axis of the column cross-section and β_d is the concrete creep factor. In this study, the creep of concrete was neglected. It should be noted that, ACI Committee 363 [7] recommended the use of the following equation for calculating the modulus of elasticity of HSC:

$$E_{c2} = 3320\sqrt{f_c'} + 6900 \qquad \text{for } 21 < f_c' < 83 \text{ MPa} \qquad (21)$$

COMPARISON OF TEST RESULTS WITH CODES PROVISIONS

The recorded experimental ultimate axial loads of the strengthened columns (P_{ujexp}) are compared with the predicted values (P_{uACI}) using the Moment Magnifier method required by ACI 318-99 code and are given in Table (4). The ultimate loads were calculated for the tested columns using the flexural rigidity of the column section EI_1 (Eq. 18) and the modulus of elasticity for E_{C2} (Eq. 21). It should be noted that, in calculating the flexural rigidity EI_1, the creep of concrete in the form of the concrete creep factor β_d was ignored. The capacity reduction factor φ was adopted as unity in calculating P_{uACI}. It can be seen from Table 4, that the recorded experimental ultimate loads were more than the predicted values for all the strengthened columns. The ratio (P_{ujexp} /P_{uACI}) was conservative for all the tested slenderness ratios and for the different applied eccentricities (with a mean equal to 1.376).

The predicted values of the ultimate lateral deflection at the midheight of the tested columns due to the second order effect (δ_u) using the equations of ECCS-2001 and EC-2 are given in Table 4. It should be noted that according to the method of ECCS-2001 the values of δ_u can be calculated using Eq. 5 or Eq. 6. However, the values calculated using Eq. 5 is more than that of Eq. 6, and hence these values were used in calculating P_{uECCS}. In calculating δ_u according to the method of EC-2, the coefficient K_2 in Eq. 10 was taken equal to 1.0. A comparison between the recorded experimental ultimate axial load of the strengthened columns (P_{ujexp}) with the predicted values (P_u) using the design methods of these two codes are given in Table 4. The values of P_u was calculated from the equilibrium between the external forces (with the applied eccentricity e= 56.1 mm or 119 mm in addition to δ_u as calculated for each method) and the internal forces of the section. It should be noted that, a rectangular stress block of maximum stress equal to ($0.85f_c'$) and the ultimate concrete strain equal to 0.003 was used in calculation of the values of P_u for the two methods. It should be noted that according to ECCS-2001, columns can be considered as slender if λ_i > 50, while according to EC-2 the second order effects can be neglected for the columns with $\lambda_i \leq 25$. According to ACI 318-99 code, columns can be considered as slender if $\lambda_i > 24$. So, for the three codes the specimens of group 3 and group 4 were considered as slender columns and second order effects were taken into consideration.

The recorded experimental ultimate load (P_{ujexp}) for the tested strengthened slender and short columns showed to be more than that predicted by the ECCS-2001 and EC-2. The values predicted by the ECCS-2001 method were generally conservative for the eleven columns (with a mean equal to 1.313). The predictions of the model column method adopted in EC-2 were also conservative for ten columns from the tested eleven columns with a mean equal to 1.238 which is slightly less than that of the ECCS-2001. Only, the tested slender column JHFC9 without steel fibers in the HSC jacket (λ_i= 40.8) showed to be slightly unconservative for the EC-2 design method. For the two codes, the conservatism for the slender columns (group 3 and 4) considerably decreased with increasing the applied eccentricity.

Comparing the results of the specimens strengthened by HSC jackets without steel fibers with the similar specimens strengthened by HSC jackets contain steel fibers (V_f=1.0%) showed that the level of conservatism of the three codes considerably increased for the jackets with steel fibers. This means that, the methods of ACI 318-99, ECCS-2001 and EC-2 can be safely used for predicting the ultimate axial load of the slender columns strengthened using HSC jackets contain steel fibers with 1% by volume. Increasing the aspect ratio of the steel fibers enhanced the factor of safety.

Table (4): Comparison of test results with the predictions of ECCS-2001, ACI 318-99 building code and EC-2.

Column	P_{ujexp} (kN)	P_{uACI} (kN)	$\frac{P_{ujexp}}{P_{uACI}}$	ECCS-2001 δ_{u1} (mm)	ECCS-2001 δ_{u2} (mm)	ECCS-2001 P_{u1} (kN)	$\frac{P_{ujexp}}{P_{uECCS}}$	EC-2 δ_u (mm)	EC-2 P_u (kN)	$\frac{P_{ujexp}}{P_{uEC-2}}$
JHFC1	419.4	284.8	1.473	0.74	0.59	284.8	1.473	0.00	284.8	1.473
JHFC2	>750	853.0	>1.00	0.74	0.59	853.0	>1.00	0.00	853.0	>1.00
JHFC3	436.9	284.8	1.534	0.74	0.59	284.8	1.534	0.00	284.8	1.534
JHFC4	407.8	284.8	1.432	1.65	1.33	284.8	1.432	0.01	284.8	1.135
JHFC5	393.2	284.8	1.381	1.65	1.33	284.8	1.381	0.01	284.8	1.112
JHFC6	408.7	261.3	1.564	6.61	5.31	256.0	1.596	4.34	265.3	1.541
JHFC7	348.6	261.3	1.334	6.61	5.31	256.0	1.362	4.34	265.3	1.314
JHFC8	326.2	261.3	1.248	6.61	5.31	256.0	1.274	4.34	265.3	1.230
JHFC9	694.2	489.0	1.420	11.76	9.43	682.0	1.018	9.88	707.0	0.982
JHFC10	305.8	243.6	1.252	11.76	9.43	236.6	1.292	9.88	243.4	1.256
JHFC11	734.0	489.0	1.501	11.76	9.43	682.0	1.076	9.88	707.0	1.038

CONCLUSIONS

From the results of this study on the behavior of eccentrically loaded slender HSC columns strengthened by HSC jackets contain steel fibers and have cube compressive strength of 89.3 MPa, the following can be concluded:

1. Eccentrically loaded slender and short cracked columns strengthened by HSC jackets contain 1% steel fibers can be treated as new integral columns cross-sections.

2. Using a steel fibers of 1% (by volume) into the HSC mix of the jackets reduces to a great extent the early cover spalling of the strengthened columns and increases the ultimate loads comparing with the same columns strengthened with HSC jackets without steel fibers

3. The failure mode of the strengthened columns depends mainly on the magnitude of the eccentricity of the applied axial load. Strengthened short and long columns tested with big eccentricity failed by typical flexural manner. Strengthened columns have slenderness ratio more than 30 failed due to increasing the tensile steel strain at the midheight up to yielding of the longitudinal reinforcement of the jacket before the compression strain reached the crushing value.

4. Increasing the aspect ratio of the steel fibers used with the HSC jackets slightly increased the ultimate load of the strengthened columns, while increasing the transverse reinforcement ratio of the HSC jackets contain 1% steel fibers over the design value slightly increased the ultimate load.

5. The minimum transverse reinforcement required by the ECCS-2001 for Normal-Strength Concrete short and slender columns can be safely used for HSC jackets contain steel fibers of 1% by volume.

6. The methods required by the ECCS-2001, EC-2 and ACI 318-99 building code for design of slender columns can be safely used for design of HSC slender columns strengthened by HSC jackets contain steel fibers.

REFERENCES

1. **Aboutaha, R. S., Engelhordt, M. D., Jirsa, J. O., and Kreger, M. E., (1995),** "Seismic Strengthening of Reinforced Concrete Columns with Inadequate Lap Splices", IABSE Symposium - Extending the Life Span of Structures, San Francisco, August, 1995, Vol. 1, pp. 343-348.
2. **Haroun, M. A. (1998),** "Ductility Enhancement in Composite-Jacketed Bridge Columns," Proceedings of the First International Conference on Civil Engineering, Department of Civil Engineering, Faculty of Engineering, Helwan University, 24-26 March 1998, Cairo, Egypt, pp. 389-399.
3. **Fahmy, E. H., Shaheen, Y. B., and Korany, Y. (1998),** "Application of Ferrocement Laminates in Repairing Reinforced Concrete Columns", The Arab Conference on Repair and Rehabilitation of Structures, Ministry of Housing, Utilities and Urban Communities, Housing and Building Research Center, Cairo, 16-19 September 1998, Egypt, pp. 410-421.

4. **Mahfouz, I., Sarkani, S., and Rizk, T. (1998),** " Repairing and Retrofitting of Circular and Square Reinforced Concrete Columns Using Fiber Reinforced Plastic (FRP)", The Arab Conference on Repair and Rehabilitation of Structures, Ministry of Housing, Utilities and Urban Communities, Housing and Building Research Center, Cairo, 16-19 September 1998, Egypt, pp. 437-451.

5. **Abdel-Rahman, A. (1985),** "Strengthening of Columns by Concrete Jackets", Proceedings of the First Egyptian Structural Engineering Conference, Structural Eng. Department, Faculty of Engineering, Cairo University, 22-25 April 1985, Cairo, Egypt, Vol. 5, pp. V1-V17.

6. **Abdel Hameed, U. M. (1999),** " Behavior of partially Strengthened Reinforced Concrete Columns Under Axial Loads," Master Thesis in Structural Engineering, Faculty of Engineering, Cairo University, Cairo, Egypt, 1999.

7. **ACI Committee 363 (1992),** " State-of-the-Art Report on High-Strength Concrete ", ACI 363R-92, American Concrete Institute, Farmington Hills, Mich., 1992, 55 p.

8. **Ministry of Housing, Infrastructures and Urban Communities (2001),** " Egyptian Code for Design and Construction of Reinforced Concrete Structures (ECCS 203-2001)," Cairo, Egypt, 2001.

9. **Commission of the European Communities (1990),** " Eurocode No. 2: Design of Concrete Structures, " Part 1: General Rules and Rules for Buildings, October, 1990.

10. **ACI Committee 318 (1999),** " Building Code Requirements for Reinforced Concrete and Commentary (ACI 318-99)," American Concrete Institute, Farmington Hills, Mich., 1999.

11. **ACI Committee 544 (1988),** " Design Consideration for Steel Fiber Reinforced Concrete," ACI Structural Journal, V. 85, No. 5, September-October 1988, pp. 563-580.

12. **Tawfik, S. Y., Mahdy, A. S., El-Raouf, H., and Zaki, Y. I. (1994),** " Fibers Closing Effects on Flexural Cracking", Second Alexandria Conference on Structural and Geotechnical Engineering, Faculty of Engineering, Alexandria University, Alexandria, Egypt, 18-20 April 1994, pp. 43-54.

13. **Foster, S. J., Liu, J., and Sheikh, S. A. (1998),** "Cover Spalling in HSC Columns Loaded in Concentric Compression," Journal of Structural Engineering, ASCE, Vol. 124, No. 12, December 1998, pp. 1431-1437.

14. **Foster, S. J., and Attard, M. M. (2001),** "On Behavior of High-Strength Concrete Columns: Cover Spalling, Steel Fibers and Ductility," ACI Structural Journal, Vol. 98, No. 4, July-August 2001, pp. 583-589.

15. **Yousef, A. M. (2001),** "Behavior of Eccentrically Loaded Slender High-Strength Concrete Columns," Mansoura Engineering Journal, Faculty of Engineering, El-Mansoura University, Vol. 26, No. 3, September 2001, pp. C1-C18.

16. **ACI Committee 544 (1993),** "Guide for Specifying Proportioning, Mixing, Placing, and Finishing Steel Fiber Reinforced Concrete," ACI Materials Journal, Vol. 90, No.1, Jan.-Feb. 1993, pp. 94-101.