Strength and Behaviour of Fibrous High-Strength Concrete columns

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ABSTRACT.

The behaviour of high-strength fiber reinforced concrete columns was observed with a testing program of 7 columns, loaded eccentrically. The theory was analyzed by modifying the stress block diagram of concrete.

The experimental results show that using high-strength fiber reinforced concrete with fiber volume fraction of 1.0%, increased the column ultimate capacity up to 40% in addition to increasing its ductility and toughness, significantly.

The proposed theoretical analysis gave a good estimation of experimental results.
Keywords: Columns, Eccentricity, Fiber Reinforcement, High Strength Concrete, Ultimate Strength.

1. INTRODUCTION.

The use of High-Strength Concrete (HSC) in construction has steadily increased over the past years leading to the design of smaller sections. Several studies have demonstrated the economy of using HSC in high-rise buildings, as well as low to mid-rise once [1]. The increasing use of HSC has led to concern over the applicability of current design methods. These methods are primarily empirical and are developed from experimental data on specimens having compressive strengths below 40 MPa [1].

HSC may be brittle, and as the concrete strength increases the post-peak portion of the stress-strain diagram almost vanishes or descends steeply [2-6]. The decrease in ductility (which is defined as the ability of the specimen to undergo large deformations before failures occur) could be a serious drawback for HSC. Research indicates that steel fibers are more effective in increasing both strength and ductility of HSC than Normal-Strength Concrete (NSC) [7]. This result from the improved bond characteristics associated with the use of fibers in HSC compared with NSC.

The use of fibers to improve the characteristics of construction materials is very old [8-11]. Addition of fibers to concrete transforms it into a more ductile material. When concrete cracks, the randomly oriented fibers arrest the microcracking mechanism and limit crack propagation, thus improving strength and ductility.

The column is one of the most critical members of a framed structure, where failure is a catastrophic type and causes the whole storey to be destroyed. The rapidly increasing use of HSC in columns [1] means that its design is now of paramount importance in framed buildings. To dissipate the energy introduced by the loading, the column should be designed with sufficient strength and ductility to allow a plastic hinge to form in the beam outside the column.

2. RESEARCH SIGNIFICANCE.

High-Strength Concretes with and without fibers possess behaviour that are significantly different from NSC materials. This paper presents an experimental investigation of the behaviour of High-Strength Fiber Reinforced Concrete (HSFRC) Columns. Theoretical analysis including modification to the stress-strain diagram is also presented.

3. EXPERIMENTAL PROGRAM.

3.1 Materials.

All reinforcing bars were plain, having yield strengths of 388.4 MPa and 409.5 MPa for φ3mm and φ9.5mm bars, respectively. Ordinary Portland cement (type I) and Natural gravel with a maximum size of 9.5mm were used. Natural sand with a maximum size of 2.36mm was used. Mix proportion by weight was 1:1.24:1.86:0.285:0.05 for the cement, sand, gravel, water, and superplasticizer, respectively.

Steel fibers of 1100 MPa tensile strength was used. (i) Straight fibers 25.4mm long with 0.4mm diameter and aspect ratio (***L/D***) of 63.5; and (ii) hooked fibers 50mm long, with 0.5mm diameter and ***L/D*** of 100.

Using the superplasticizer (Melment L-10) resulted in uniform mixing of the concrete even with 1.5% fiber content there was no evidence of balling or segregation. Control specimens (150×300 mm size) and standard prisms (100×100×400 mm size) were used to obtain the fiber reinforced concrete compressive strength (f'_{cf}) and modulus of rupture (f_{rf}), respectively. Each point is the average of three test specimens.

All concrete was placed and internally vibrated in 2 layers. The columns were cast horizontally and together with the control specimens they were cured in continuously damp condition under plastic sheets for 28-days, followed by 1-day drying before testing.

3.2 Test specimens.

The variables in the 7 HSFRC tested columns were concrete compressive and tensile strength, type and volume fraction of steel fibers. Preliminary tests have shown that the cylinder strength of concrete before fiber was about 60 MPa, which has increased by up to 17% with fibers giving the values of f'_{cf} **Table (1)**. Two types of steel fibers (hooked and straight) were used with volume fraction ranging from 0.0 to 1.5%.

All columns had four 9.5mm longitudinal bars with 3mm closed ties at 100 mm c/c spacing. **Table (1)** and **Fig. (1)** present details of columns in this testing program.

Rollers were used to facilitate applying loads to the test specimens with a fixed eccentricity of 35mm for all of the 7 columns in this work. The small value of 35mm was chosen to ensure compression failure. Tests were made using Universal Testing Machine with a capacity of 2500kN. Dial gages were used to measure the middle deflection and shortening of the column, in addition, mechanical strain gages were used to measure tension and compression strains. **Fig. (2)** Shows the testing frame and location of the dial and strain gages.

4. EXPERIMENTAL RESULTS.

All test results are summarized in **Table (1)**, while graphical representations of these results are displayed in **Figs. (3-10)**. A total of 7 column specimens were tested in this investigation, all of them failed in compression type failure. **Fig. (11)** shows photographs of the 7 test specimens after failure.

Firstly, the column tension cracks started in a similar way for all 7 specimens, at the middle of the tension face and increased towards the edges of the specimen. Secondly, the compression cracks appeared, for all specimens, there was a major dominant crack with a few additional cracks to its side, which increased in size causing compression failure eventually, as shown in **Fig. (11)**.

Because of the domination of compression type failure, there was no significant difference in crack pattern between specimens with and without fibers. However, the integrity at failure of SP-3, which had 1.0% volume fraction of hooked type fibers, was significantly better than that of SP-1, which was identical but without fibers. This indicates that repair after damage (retrofitting) could be easier if the columns with fibers is used.

4.1 Column middle deflection.

Figs. (3 and 4) shows the relationship between the applied load and column middle deflection for specimens with hooked fibers and specimens with straight fibers, respectively. From these figures, it is well shown that ductility is increased for specimens with fibers over reference specimen SP-1 without fibers. Nevertheless, the ductility of specimens with hooked type steel fibers (SP-2-3-4) is better than that of specimens with straight steel fibers (SP-5-6-7), and this can be related to the improved bond characteristics of hooked steel fibers in comparison to straight ones. The fiber content showed no significant variation in ductility among specimens with different fiber volume fraction (0.5, 1.0, 1.5%) for the same type of steel fibers.

4.2 Column shortening.

Figs. (5 and 6) shows the relationship between the applied load and column axial shortening for specimens with hooked fibers and specimens with straight fibers, respectively. From **Fig. (5)** it is well shown that addition of 1.0% hooked steel fibers to the columns (SP-3) increased the ductility of the specimen significantly, while **Fig. (6)** show that addition of 1.0% straight steel fibers to the columns (SP-6) increased only the toughness (which is defined as the area under the load-deflection curve) of the specimen but not ductility.

4.3 Tensile and compressive strain.

Figs. (7-10) show the relationship between the applied load and column tensile and compressive strain for specimens with hooked fibers and specimens with straight fibers, respectively.

From these figures it is shown that the addition of steel fibers increased the ductility of specimens rather than specimens without steel fibers significantly. The addition of 1.0% hooked steel fibers succeeded in increasing the ultimate strain at the middle of the column by 171% and 44% for tensile and compressive strain, respectively. While, the addition of 1.0% straight steel fibers succeeded in increasing the ultimate strain at the middle of the column by 67% and 44% for tensile and compressive strain, respectively. From these results, it is shown that the hooked steel fibers improved tensile strain significantly rather than straight ones, but there was no difference between hooked and straight steel fibers in improving compressive strain.

4.4 Ultimate strength.

One of the major problems in fiber reinforced concrete (FRC) is how to represent the effect of including fibers, in the concrete matrix, in estimating the strength of HSFRC columns.

The fiber factor (***F***) is the most reliable solution for this problem; ***F*** can be expressed as follows:

$$F = V_f . b_f . L / D$$

where:

V_f is the fiber volume fraction,
b_f is the fiber bond factor,
= 0.25 for straight fibers,
= 0.5 for hooked fibers,
L&D are the fiber length and diameter, respectively.

This nondimensional factor represents the effect of fiber powerfully, as it represents the fiber in its volume fraction, bond factor, and aspect ratio. Thus F can be used to represent any type and volume fraction of fibers.

Table (1) and **Fig. (12)** show the relationship between failure load and the fiber factor F, which represent the effect of addition of fibers to the column specimens. From this figure it is shown that the ultimate capacity of the columns is increased significantly with increasing the fiber factor F, but it also shows that there may be an upper limit to the usefulness of V_f for both straight and hooked fibers. Using 0.5% and 1.0% for V_f succeeded in strengthening the column. In contrast, using 1.5% for V_f led to a drop in strength compared to columns with 1.0% for V_f.

5. EVALUATION OF EXPERIMENTAL RESULTS.

5.1 Joint strength design equations.

The procedure for the analysis of concrete columns is well demonstrated in the ACI code [12], yet this method is primarily empirical and are developed from experimental data on specimens having compressive strengths below 40 MPa [1], with no steel fibers of course.

A modification to the ACI method is proposed here taking into account the effect of inclusion of steel fibers on the stress block diagram, based on the work firstly proposed by Wafa etal [13] to simulate the effect of inclusion of steel fibers on flexural strength of beams. **Fig. (13)** shows this modification by adding a stress block for the tension zone to account for the increase in the tensile resistance of concrete with fibers.

The following algorithm illustrates the modified method proposed in this research:

assume c

$$a = \beta_1 \times c$$

$$\varepsilon_{sf} = \frac{\tau_f}{E_{sf}}$$

$$e = \left[\varepsilon_{sf} + \varepsilon_c\right] \cdot c / \varepsilon_c$$

$$\varepsilon_{sb} = \varepsilon_c \cdot \frac{d - c}{c}$$

$$f_s = E_s \cdot \varepsilon_{sb} = \left(E_s \cdot \varepsilon_c \cdot \frac{d - c}{c} \right)$$

$$\bar{\varepsilon}_{sb} = \varepsilon_c \cdot \frac{c - \bar{d}}{c}$$

$$\bar{f}_s = \bar{E}_s \cdot \bar{\varepsilon}_{sb} = \left(\bar{E}_s \cdot \varepsilon_c \cdot \frac{c - \bar{d}}{c} \right) \leq f_y$$

$$C_c = 0.85 \times f'_{cf} \times a \times b$$

$$T_c = f_{rf} \times b \times (h - e)$$

$$T_s = A_s \times f_s$$

$$C_s = \bar{A}_s \times \left(\bar{f}_s - \left(0.85 \times f'_{cf}\right)\right)$$

$$P_n = \left(C_c + C_s - T_c - T_s\right)$$

$$M_n = C_s \times \left(c - \bar{d}\right) + C_c\left(c - \left(a/2\right)\right) + T_c \times \left(\left(\left(h - e\right)/2\right) + \left(e - c\right)\right) + T_s \times \left(d - c\right)$$

$$\bar{e} = \left(M_n / P_n\right)$$

$$e_{cc} = \bar{e} + \left(\left(h/2\right) - c\right)$$

If $e_{cc} \leq 35mm$ *increase c*
end

A computer program was established to execute the upper algorithm and replace the usual interaction diagram used in designing columns (i.e. solving the cubic equation resulting from equilibrium analysis of the column section); the results are illustrated in **Table (2).**

5.2 Comparison of design methods.

Table (2) compares between the experimental results, ACI method, and the proposed algorithm. From this table it is clear that the proposed algorithm gives a good and conservative estimation in comparison to the ACI method which gave very conservative results. **Fig. (14)** illustrates this fact schematically by comparing the safety ratio (**R**) of the experimental results to the design ACI and the proposed methods.

Where

$$R = P_{nEXP} / P_{nDES}$$

P_{nEXP} is the failure load
P_{nDES} is the value of load calculated from ACI method or the Proposed one.

6. CONCLUSIONS.

Based on the seven HSFRC columns tests, the following conclusions can be drawn:

1- HSC columns without fibers exhibited sudden modes of failure. Therefore, it is not recommended to construct HSC columns without fibers unless the designer ensures that the failure would be outside the column by considering all the design variables including overstrength of the adjoining members [14].
2- The addition of fibers to the columns helped to keep significantly better integrity of specimens at failure load, in contrast with columns without fibers.
3- The addition of steel fibers to HSC columns had led to a more ductile behaviour of the member during testing.
4- **Table (1)** shows that there may be an upper limit to the usefulness of V_f for both straight and hooked fibers. Using 0.5% and 1.0% for V_f succeeded in strengthening the column. In contrast, using 1.5% for V_f led to a drop in strength compared to columns with1.0% for V_f. All 7 HSC columns could be safely designed using the proposed method.
5- Current existing ACI design method was found to be very conservative for HSC column design, with and without fibers.
6- Based on this work, using steel fibers for HSC columns with an upper limit of 1.0% volume fraction has proved to be successful in improving both the ductility and strength of columns.

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8. NOTATION.

c	Is the distance from extreme compression fiber to the neutral axis of the section
a	Is the height of compressive stress block in the concrete section
e	Is the distance from extreme compression fiber to the proposed tension block distribution
β_1	Is a factor equals 0.65 in this work
ε_c	The maximum externally measured concrete compression strain
ε_{sf}	Is the tensile strain of fibers
τ_f	Is the fiber pullout bond strength and equals 2.75 MPa for straight fibers and 4.75 MPa for hooked fibers.
E_{sf}	Is the modulus of elasticity of steel fibers, in this work it is taken equal to 200×10^3 MPa
ε_{sb}	Is the strain in tensile bars
$\bar{\varepsilon}_{sb}$	Is the strain in compression bars
E_{sb}	Is the modulus of elasticity of tensile bars, MPa
$\bar{E}_{sb}$	Is the modulus of elasticity of compression bars, MPa
f_s	Is the stress in tensile bars, MPa
$\bar{f}_s$	Is the stress in compression bars, MPa
A_s	Is the area of tensile bars, mm^2
$\bar{A}_s$	Is the area of compression bars, mm^2

f_{rf}	Is the modulus of rupture of HSFRC, MPa
C_c	Is the concrete compression force
T_c	Is the concrete tensile force
C_s	Is the steel bars compression force
T_s	Is the steel bars tensile force
P_n	Is the column nominal axial force, kN
M_n	Is the column nominal moment, kN.m
$\bar{e}$	The eccentricity from the applied force to the neutral axis, mm
e_{cc}	The eccentricity from the applied force to the plastic center, mm

Table (1): Experimental results.

Specimen	V_f %	f'_{cf} (MPa)	f_{rf} (MPa)	P_n (kN)
SP-1	0.0	61	7.2	260.6
SP-2	0.5H	63.5	9.5	309.9
SP-3	1.0H	68.2	11.5	366.8
SP-4	1.5H	70.5	12.75	344.7
SP-5	0.5S	61.3	7.95	295.5
SP-6	1.0S	62.0	8.55	316.5
SP-7	1.5S	63.4	9.1	304.9

H&S represents Hooked and Straight steel fibers, respectively.

Table (2): Comparison between design methods.

Specimen	V_f %	P_n (kN)		
		Experimental	*ACI Method*	*Modified Method*
SP-1	0.0	260.6	235.98	252.05
SP-2	0.5H	309.9	243.56	267.3
SP-3	1.0H	366.8	255.5	282.2
SP-4	1.5H	344.7	267.5	298.0
SP-5	0.5S	295.9	240.0	260.6
SP-6	1.0S	316.5	243.8	263.4
SP-7	1.5S	304.9	248.1	266.8

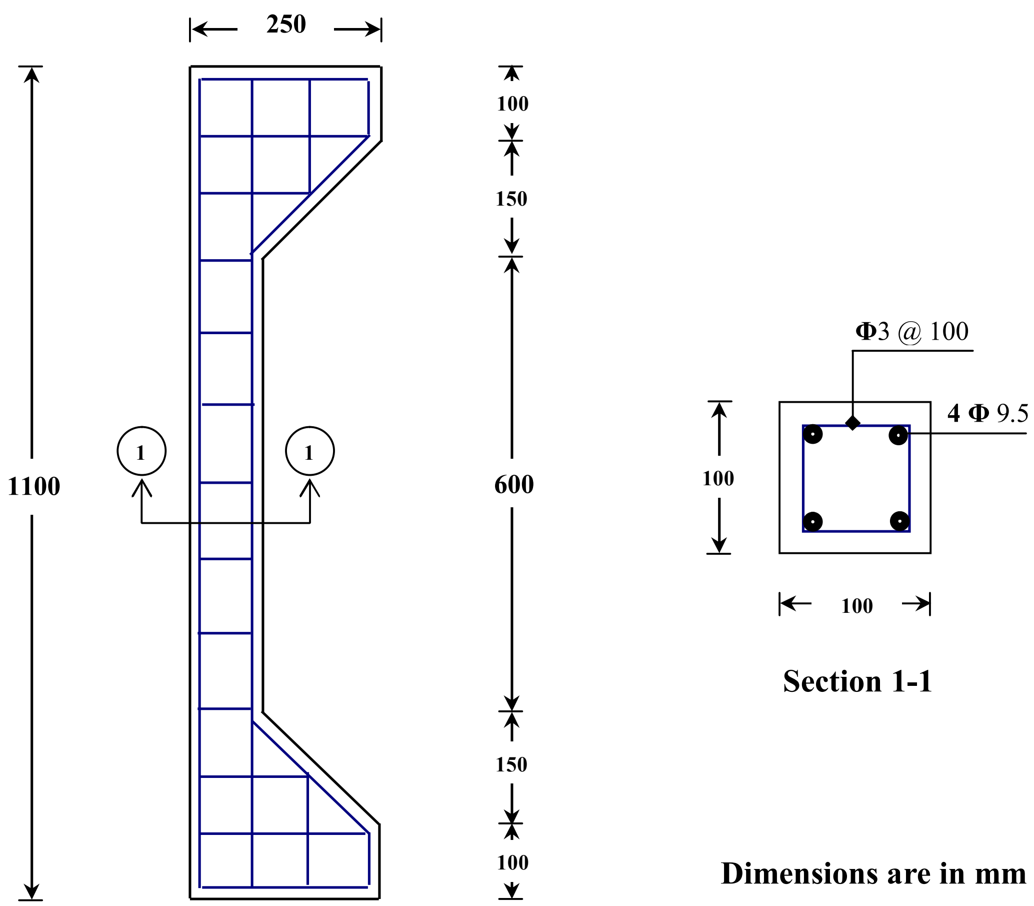

Figure (1): Test specimen details.

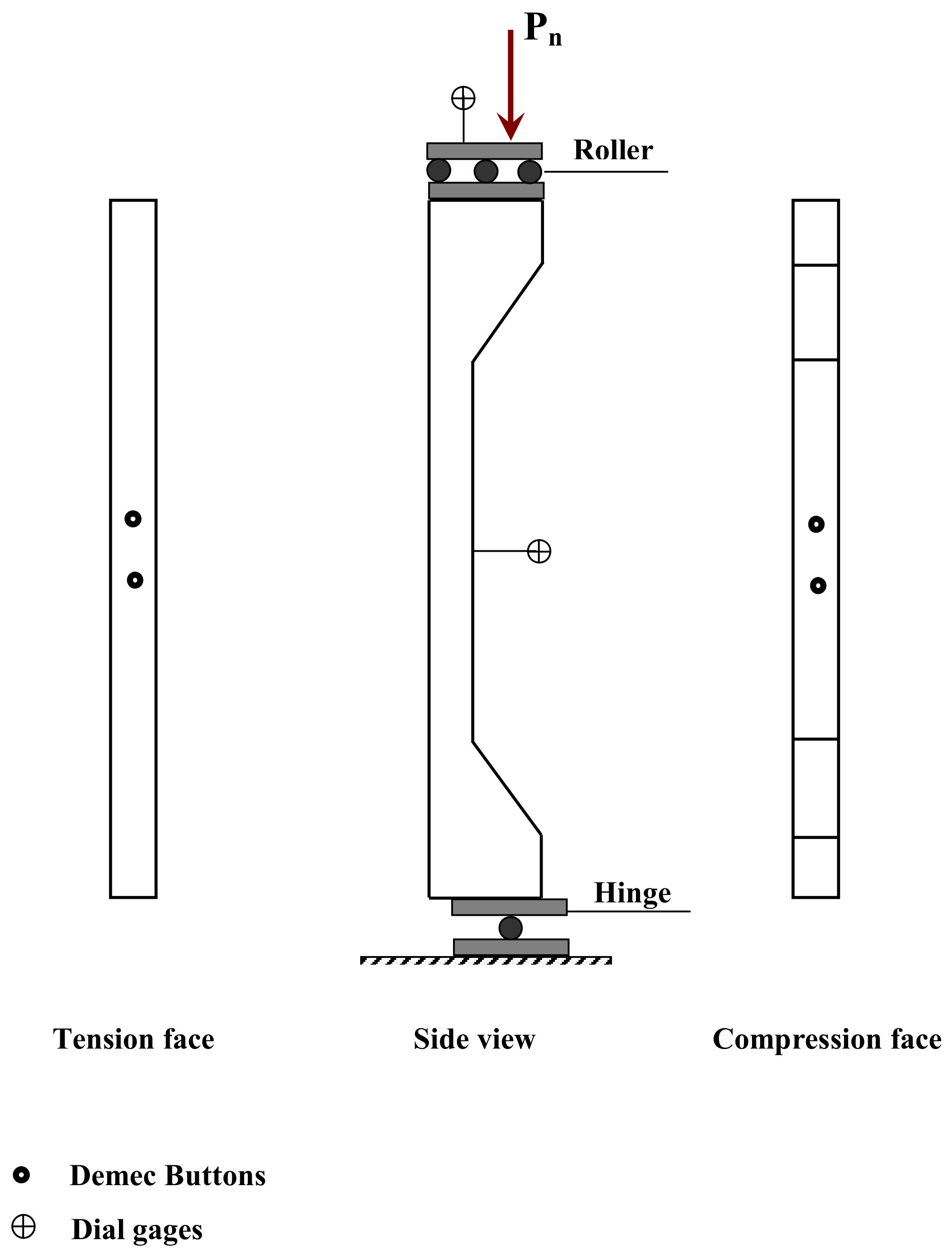

Figure (2): Test frame details.

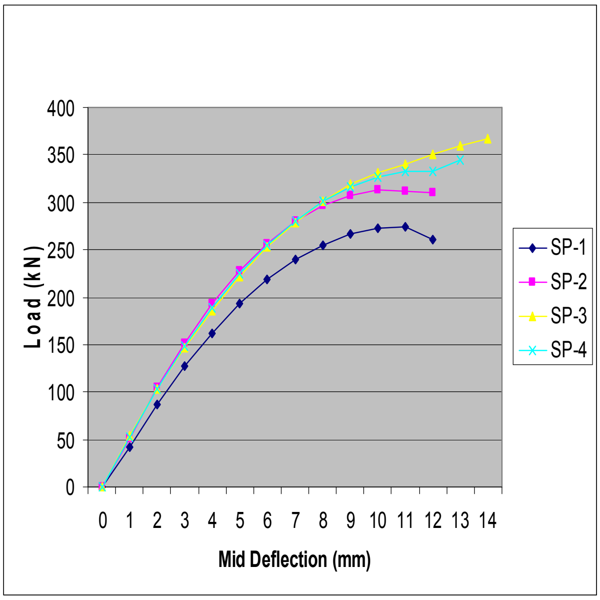

Figure (3): Load-mid deflection relationship.

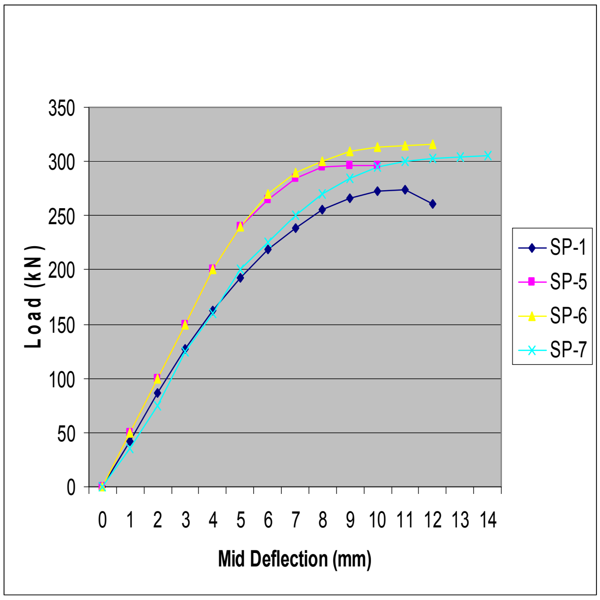

Figure (4): Load-mid deflection relationship.

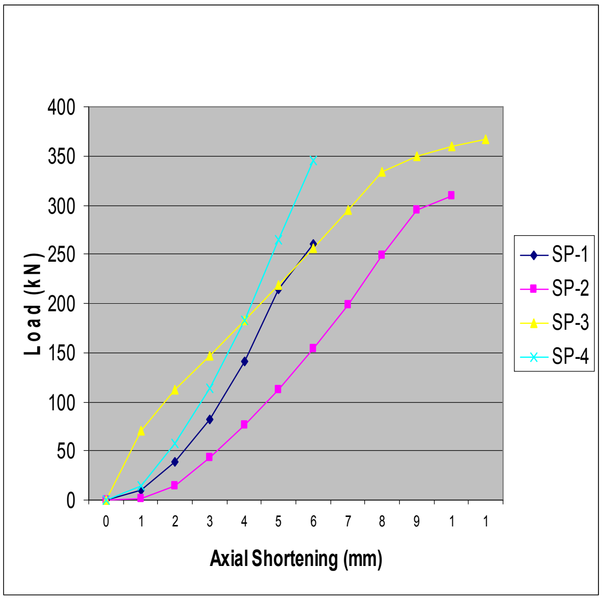

Figure (5): Load-axial shortening relationship.

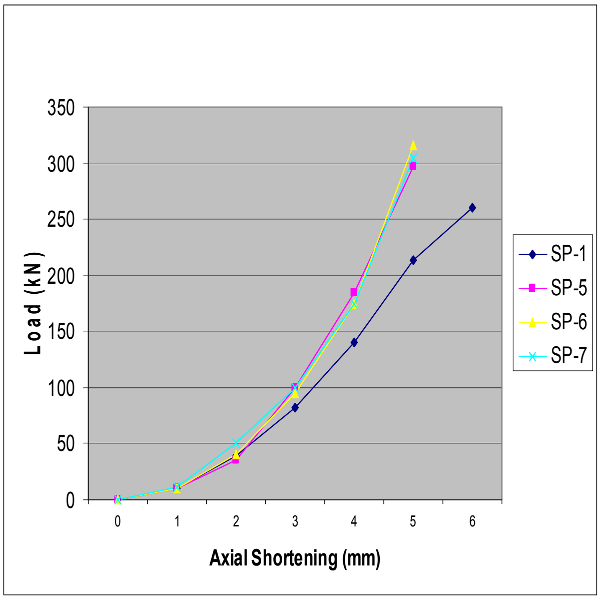

Figure (6): Load-axial shortening relationship.

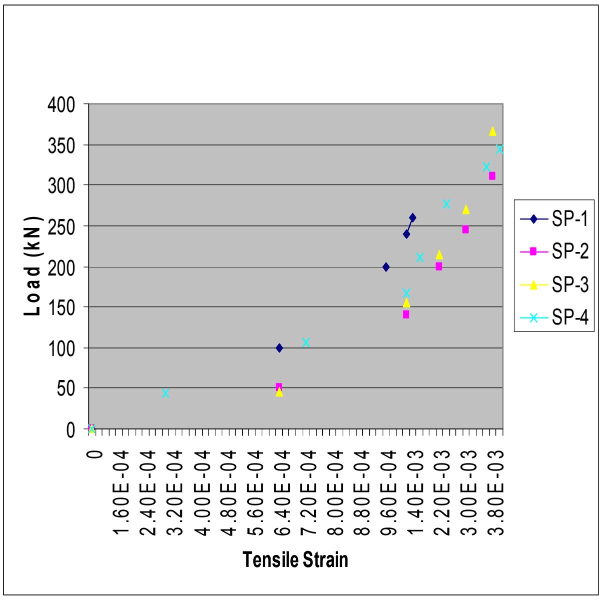

Figure (7): Load-tensile strain relationship.

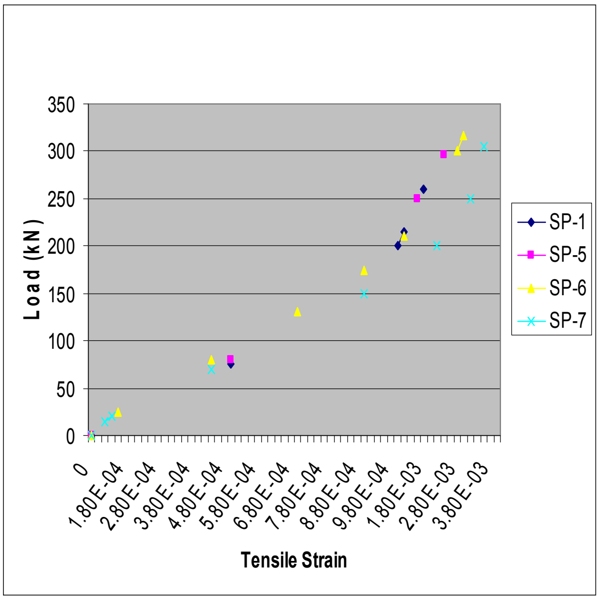

Figure (8): Load-tensile strain relationship.

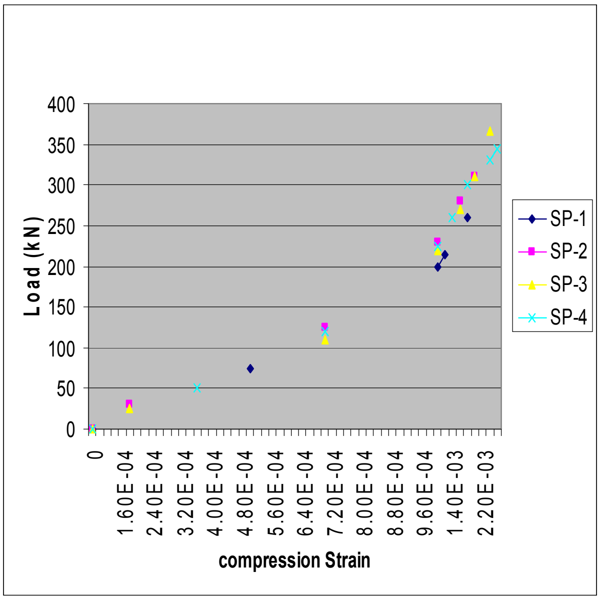

Figure (9): Load-compression strain relationship

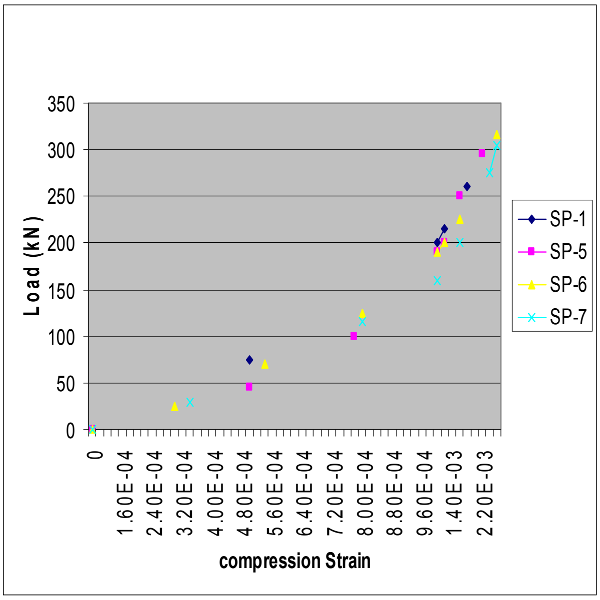

Figure (10): Load-compression strain relationship.

Figure (11): Specimens after failure.

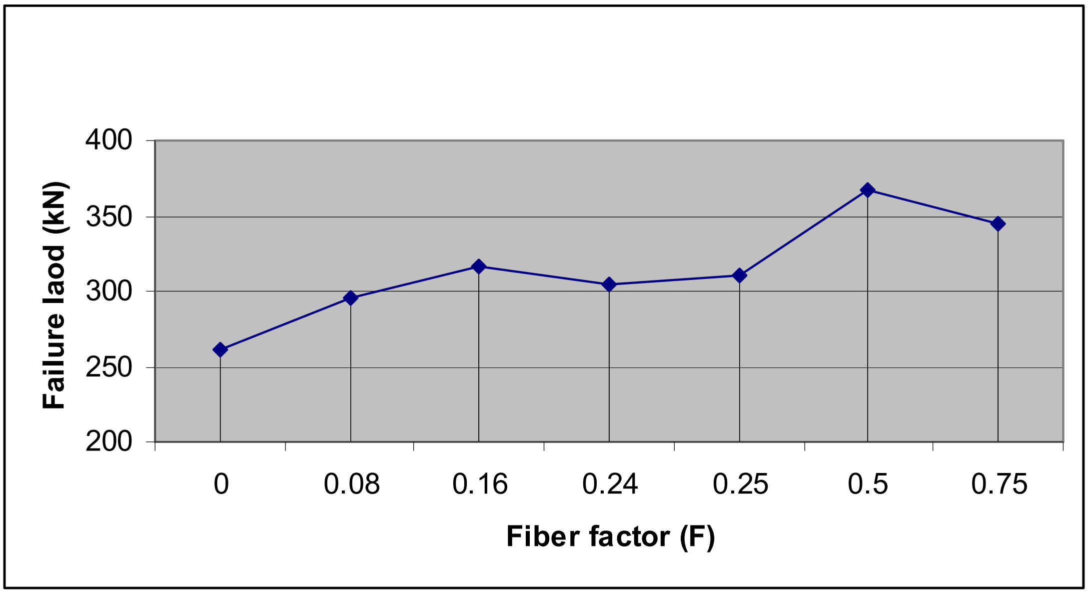

Figure (12): Failure load- fiber factor relationship.

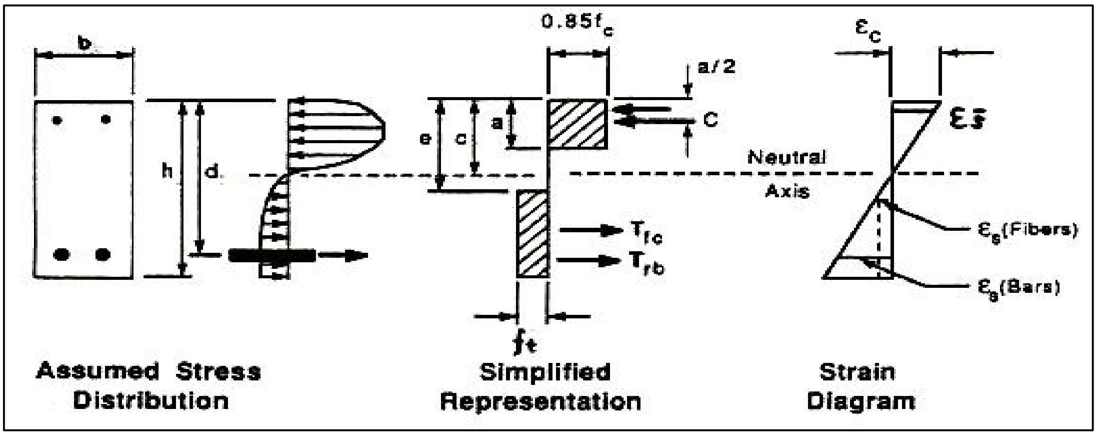

Figure (13): The modified stress block diagram for the proposed method.

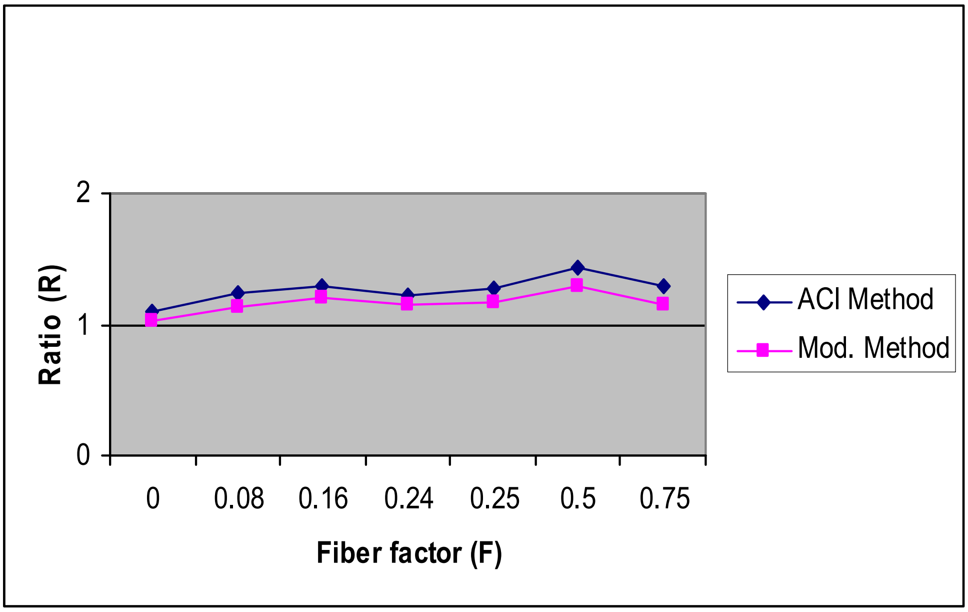

Figure (14): The relation ship between the ratio (R) and fiber factor (F).

مقاومة وتصرف الأعمدة المصنّعة من الخرسانة عالية المقاومة والمسلحة بالألياف

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الخلاصـــة

تمت مراقبة تصرف الأعمدة المصنعة من الخرسانة عالية المقاومة والمسلحة بالأياف من خلال برنامج فحص لسبعة نماذج محملة لا مركزياً. وقد تم التحليل النظري من خلال تحديث منحني الإجهاد- الإنفعال للأعضاء الخرسانية. أظهرت النتائج العملية أن استخدام الخرسانة عالية المقاومة والمسلحة بالألياف الحديدية بنسبة حجمية تساوي 1%، قد أدى إلى زيادة التحمل الأقصى للأعمدة بنسبة 40% إضافةً إلى تحسين المطيلية (المرونة) والجساءة لنماذج الفحص.

إن التحليل النظري المقترح قد جاء مطابقاً بنسبة جيدة للنتائج العملية.