

Influence of Fiber Dispersion on the Performance of Microfiber Reinforced Cement Composites

by Y. Akkaya, S. P. Shah, and M. Ghandehari

Synopsis: High performance fiber reinforced composites are identified by their enhanced elastic behavior, pseudo-strain hardening response and toughened post-peak response. If the composite is adequately reinforced by fibers, the bridging action will transfer the load and multiple cracking will occur. This study investigates the effect of dispersion of fibers on the multiple cracking behavior of fiber reinforced composites. Electronic Speckle Pattern Interferometry technique is used to record the location of crack initiation, sequence of the multiple cracking and corresponding cracking stresses. Microstructural parameters at each crack location are statistically quantified by the theory of point processes. The size of the fiber free areas and fiber clumping are calculated at the crack cross-sections. By using Linear Elastic Fracture Mechanics, fracture toughness of the matrix is calculated. A strong relation between the cracking stress and the fiber free areas in the composite is observed. It is shown that the toughness of the composite depends on the fiber clumping at the first crack cross-section.

Keywords: extrusion; fiber clumping; fiber dispersion; fiber-reinforced concrete; laser interferometry; multiple cracking

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INTRODUCTION

High performance fiber reinforced composites are characterized by enhanced elastic limit, strain hardening response and toughened post-peak response. Enhanced elastic behavior terminates when the first macrocrack in the matrix appears. When the load increases on the composite, a flaw spreads at the cross-section of the matrix. The mechanics of this spread depends on the size of the flaw, the properties of the fiber reinforcement and matrix toughness. If the composite is adequately reinforced, the bridging fibers will share the load and transfer it to the other parts of the composite. During the strain hardening response, the number of cracks increase and cracks widen very little. Multiple cracking occurs when the subsequent transferred load cracks the matrix again. Hence, the initial flaw size and the fiber dispersion play important role on the initiation of cracking and toughness(1-5). At the toughened post-peak response, the number of cracks remain essentially constant and crack width increases.

The performance of the composite depends on the fiber parameters such as type, length, diameter, volume, and dispersion of the fibers. Previously it is reported (6,7) that due to better dispersion shorter fiber composites have enhanced mechanical properties compared to longer fiber composites at the same volume addition.

Experimental results with 2, 4 and 6 mm long PVA fiber reinforced extruded cement composites indicated that short fiber composite failed with an enhanced post-peak behavior with multiple cracks. Microstructural studies confirmed that shorter fibers dispersed better than the longer fibers, and this in turn increased the toughness performance of the composites.

This study investigates the effect of fiber dispersion on the multiple cracking behavior of extruded fiber reinforced composites. Extrusion is a forming process, which forces a highly viscous, dough-like plastic mixture through a rigid opening (a die) having a desired cross-sectional geometry. The extrusion process provides flexibility of the product shape, high fiber content, alignment of fibers and a strong interfacial bond between fiber and matrix (5,6,8). Electronic Speckle Pattern Interferometry (ESPI) technique is used to observe multiple cracking of the material and to evaluate cracking stresses of the fiber reinforced composite. ESPI technique is a highly accurate

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displacement measurement method, which allows mapping of the crack propagation at the micro-scale.

Microstructural parameters at each crack location are quantified, and the size of the fiber free areas and the percentage of fiber clumping in the composite are calculated by point process statistics. Each fiber is idealized as a point in the analysis and the relationship between fiber distances is investigated. It is observed that the cracking stress decreases when the size of the fiber free areas in the composite increase. By using this relation, fracture toughness of the matrix is calculated by linear elastic fracture mechanics (LEFM). It is also observed that the toughness of the composite decreases when the fibers are clumped at the first crack cross-section of the composite.

MATERIALS AND EXPERIMENTAL TECHNIQUES

The extruded composite was made with 2 mm long by 14 μ m diameter PVA fibers. The basic mix design of the composite was (by volume) 45% portland cement, 12% silica fume, 3% fibers, 1% superplasticizer and 39% water, with a 0.29 water/cement ratio (by weight).

Specimens are produced by extrusion of the fresh composite. A ram type extruder mounted on an MTS machine is used to process the specimens under stroke control. A die with a rectangular cross-section of 25.4 mm x 4 mm is used during extrusion. Details of the extruder can be found in Shrinivasan et al. (9). After 24 hours from extrusion, specimens are placed in steam curing at 90°C for two days. Thereafter, they are kept in constant temperature and humidity of 20°C and 50 RH% for three days. Then the specimens are oven-dried at 105°C for 24 hours and tested in flexure after another 24 hours.

Flexural behavior of the composites is studied to evaluate the mechanical performance. A closed-loop, four-point flexure test is performed by controlling the displacement measured by an LVDT mounted on the specimen (Figure 1). Specimens are loaded at a rate of 3×10^{-5} mm/sec. The gauge length was 38.1 mm. The resulting load versus displacement data is recorded. Five identical specimens are tested. Flexure tests are monitored with ESPI technique to record the location of crack initiation, and sequence of the multiple cracking.

The ESPI experimental setup includes an interferometer, a loading unit, image acquisition and processing units, an analog/digital interface and an image storage and display (Figure 1). The optical phase-shifts are controlled by piezo-electric transducers. A 45 mW Helium-neon laser is used as the light source. The test is monitored by a CCD camera to record the speckle patterns. The scattered images of the reference and deformed states of the composite and load – displacement values are recorded. The speckle pattern fringes are formed by the electronic subtraction of reference image and deformed image.

Microstructural observations at the cross-sections of crack locations are made by

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a Scanning Electron Microscope (S.E.M.), in order to understand the mechanisms involved in the composite failure. The specimens are cut at the fracture plane along the specimen width (Figure 2). Polished specimens are prepared in order to study fiber dispersion and fiber free areas in the composite. Statistical analysis is employed to quantify the fiber dispersion on the polished cross-section. The relation between microstructural parameters and the composite cracking is evaluated.

ELECTRONIC SPECKLE PATTERN INTERFEROMETRY TECHNIQUE

ESPI technique is a highly accurate displacement measurement method, which allows mapping of the crack propagation at the micro-scale. The sensitivity of the technique is in the order of a fraction of wavelength of the monochromatic light. Speckle effect forms when a rough object is illuminated with a laser light. The scattered light from the object presents a granular appearance. Each point on the object surface absorbs and re-emits the light as a source of spherical waves. The complex amplitude of the scattered light at any point in space is given by the sum of the amplitudes of the contribution from each point on the object surface. Since the surface roughness is typically random, the intensity of the scattered light at any point also varies randomly, leading to the speckle effect. The random speckle pattern contains information about the surface shape of the object. Speckle interferometry utilizes this phenomenon to obtain the relative deformation due to loading of a structure. This is done by comparing (correlating) the random speckle pattern at two different loaded states of the object. If the random variation in phase and amplitude of the speckle patterns are not too drastic, the speckle patterns in the original and displaced states are correlated and can be processed to obtain correlation fringes. Excessive displacement can be a source of de-correlation. To overcome this effect, the displacement process was segmented into intervals and a significant change in the random phase and amplitude of the speckle pattern was prevented. Details about the technique and fringe formation can be found in Ghandehari et.al.(10,11).

FIBER DISPERSION ANALYSIS

Fiber dispersion is important in the initiation and sequence of the cracking. As the fibers clump and the size and number of matrix areas which are not supported by fibers increase, the initiation of a crack requires less energy and once the crack forms, it can advance easily through the fiber free areas in the matrix.

Point process statistics are calculated in order to quantify the dispersion of fibers. The procedure is described in (12,13). SEM pictures of the polished crack surfaces are digitized, each fiber is marked and taken as a point in the image analysis program. Then the coordinates of each fiber are used to calculate the distances between fiber pairs. These distances are used to calculate the statistical point process functions. An area of 10 mm² on the cross-sectional surface, where the cracking initiated, is used for analysis. Further information about specimen preparation can be found in Akkaya et. al. (6).

Two statistical functions are calculated to quantitatively describe the fiber

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dispersion. The K-function is a standard measure of the expected number of fibers within a certain distance of a given fiber location (Figure 3a). The K-function is calculated to describe the tendency of fibers to clump. The F-function is the distribution of distances between arbitrary points to the nearest fiber, which can be used to describe the size of the fiber free areas (Figure 3b). The F-function gives the probability of any point in the cross-section being at least a distance r away of the nearest fiber.

The K-function is the normalized number of fibers within a distance r of a given fiber. Increase of K-function implies that fibers are clumping since the number of fibers at a certain distance from a given fiber is increasing. For the estimator of K-function the following formula is used (13):

$$K(r) = \frac{\text{number of fibers within } r \text{ of a given fiber}}{(\text{number of fibers})^2} \times \left(\frac{\text{Area of region}}{\text{of interest}} \right) \quad (1)$$

Where, r is the distance from the fiber. In order to analyze if the fiber dispersion of the composites is perfectly random or if they show any clumping behavior, reference random data points (Poisson distribution) are used. This random dispersion is compared to the fiber dispersion of the crack locations. For a complete spatial random dispersion K-function would be πr^2 (Figure 4). K-functions on the upper side of random dispersion would indicate clumping, where K-functions on the lower side of random dispersion would indicate a regular dispersion. Percentage of clumping in the composite with respect to the random dispersion can be calculated by,

$$\% \text{ Clumping} = \left(\frac{C}{3.14} - 1 \right) \times 100 \quad (2)$$

Where, C is the constant term in Cr^2 , the curve fit to the K-function of the fiber dispersion at the crack cross-section (14).

Estimation of the F-function is achieved by laying down a regular grid of points on the observed region (4961 points were used), and then finding the nearest fiber to each grid point. The largest of these arbitrary point-nearest fiber distances is the radius of the maximum fiber free area (Figure 3b). The following estimator is used for F-function (13):

$$F(r) = \frac{\text{number of grid points whose nearest fiber is less than } r \text{ away}}{\text{total number of grid points}} \quad (3)$$

Since the compliment of this function would indicate the fraction of the total fiber free area, $1-F(r)$ is calculated. High values of $1-F(r)$ indicate the increase in the fiber free areas. If the fibers exhibited complete spatial randomness, then the F-function would take the form, $1-\exp(-\pi\lambda r^2)$, where λ is the number of fibers in the observation area (12).

LINEAR ELASTIC FRACTURE MECHANICS:

Fiber reinforced cement composites contain preexisting flaws and defects. It is the size of these flaws that determines the critical load required for crack initiation. Far-field stresses, well away from the location of the flaw, will be equal to the applied stress, σ_a . However, the presence of the flaw along the cross-section perpendicular to the applied load would cause stress concentration at the tip of the flaw. Here, the flaw is considered to be in an infinite plane, traction-free and subjected to uniaxial tensile stress. According to the principles of linear elastic fracture mechanics (15), stresses at the flaw tip are proportional to a factor called the Stress Intensity Factor, K_I , which can be derived by basic principles of elasticity. A general expression for K_I would be,

$$K_I = \sigma_a \sqrt{\pi a} f\left(\frac{a}{b}\right) \quad (4)$$

Where, a is the flaw size and $f(a/b)$ accounts for the geometry of the structure. Various K_I solutions for different geometry can be found in fracture mechanics books (16, 17, 18). K_I increases proportionally with the applied load and at a critical value, K_{IC} , the failure occurs. LEFM is applicable as long as the initial flaw size can be considered unchanged. In this study, LEFM is used to calculate the fracture toughness at the crack initiation.

For an edge cracked plate subjected to pure bending, K_I solution is given as (17),

$$f\left(\frac{a}{W}\right) = \frac{K_I B W^{3/2}}{M} = \frac{6 \sqrt{2 \tan\left(\frac{\pi a}{2W}\right)}}{\cos\left(\frac{\pi a}{2W}\right)} \left[0.923 + 0.199 \left\{ 1 - \sin\left(\frac{\pi a}{2W}\right) \right\}^4 \right] \quad (5)$$

Where, B is the specimen thickness, W is the specimen width, a is the flaw size, M is the moment. Fracture toughness K_{IC} can be calculated by Equation 5, by using the moment and flaw size at which the composite cracking occurs.

TEST RESULTS AND ANALYSIS

Figure 5 gives the 4-point bending test results of the identical composites. Cracking stresses, maximum stresses and corresponding displacements are given in Table 1. ESPI pictures of cracks of one of the samples are presented in Figure 6a. This figure presents the crack initiation locations and corresponding stresses and displacements. For this composite, a total of six cracks have been observed and examined. Crack locations can be identified by discontinuities (Figure 6b) in the fringes of ESPI pictures. The number of cracks examined in the other specimens can be found in Table 1.

Later, each specimen is separated from the main crack location, polished and SEM picture of the cross-section is taken. Then the specimen is polished until all the other crack cross-sections are exposed and SEM pictures taken. Figure 7 presents the

pictures of the first and sixth cracks, used in the image analysis program to obtain the coordinates of each fiber. From this, the distances between fiber pairs and the point process statistics are calculated.

Result of the K-function of specimen S1 is given in Figure 8. Similar results are found for other specimens. Crack cross-sections with evenly dispersed fibers can be identified by their closeness to the random dispersion. As it can be seen from the figure, first, second and third crack cross-sections have higher K-function values, meaning more number of fibers can be found around a fiber. Cross-sections with a higher degree of fiber clumping would crack before cross-sections with evenly dispersed fibers. Fourth, fifth and sixth cracks exhibited fiber dispersions similar to the random dispersion. Figure 9a presents the result of (1-F)-function. At the first crack location, as it can be seen from the figure, number of fiber free areas observed are more than the subsequent cracks. These fiber free areas act as defects in the composite and decrease the mechanical performance. Figure 9b presents the comparison of the largest fiber free area size. The largest fiber free area is found at the first crack location. Size of the largest fiber free areas found at the crack locations of all specimens can be found in Table 1.

K_{IC} values of the matrices are calculated by Equation 5. Experimental values of the cracking loads are used for the calculation of the moments. The radius of the fiber free areas are used as the flaw size. K_{IC} value is calculated for each crack of the specimen and their averages are used to calculate the K_{IC} value of the specimen. The average K_{IC} value is calculated from the data for specimens S1 to S5 as $26 \text{ MPa mm}^{1/2}$ (Figure 10), and is comparable to published values (19, 20, 21). Detailed statistical analysis of this comparison can be found in (14). From these results, it is reasonable to conclude that the procedure used to measure the size of the fiber free areas is reasonably accurate and that the linear elastic fracture mechanics assumptions are a good approximation for the cracking process.

The cracking load versus defect size values and Equation 5 (with $K_{IC}=26 \text{ MPa mm}^{1/2}$) are plotted in Figure 11. In both the theoretical model and the data, load increases as the size of fiber free areas decrease.

Toughness of the composite is expressed by the ratio of displacement at the maximum stress to the displacement at the first cracking stress, evaluated from the 4-point flexure test (Table 1). A variation in toughness can be seen among the identical specimens. Microstructural parameters are used to explain the different behavior among the samples. Percentage of the fiber clumping at the first crack location is calculated by Equation 2 (Table 1). A close relationship between the fiber dispersion at the first crack location and composite toughness is seen in Figure 12. SEM pictures of fiber dispersion at the first crack locations of the specimens with highest and lowest toughness values are also compared on the figure. An efficient fiber bridging and transfer of the load to the other parts of the composite can be achieved if there is a better fiber dispersion and less fiber clumps at the first crack location. Better fiber dispersion, in turn, increases the toughness of the composite.

CONCLUSION

Microstructural parameters such as the size of the fiber free areas and the fiber dispersion in the composite are associated with the mechanical performance and the sequential multiple cracking of the fiber reinforced cement based composites. These parameters can be quantified by using the statistical approach described in this study. It is seen that fiber free areas reduce the cracking stress of the composite by acting as defects in the material. The load at which sequential cracks form depends on the size of the fiber free areas at the composite cross-section.

K_{IC} of the extruded matrix is calculated as $26 \text{ MPa mm}^{1/2}$, and is in accordance with the values reported for cement paste in the literature. This indicates that the procedure used to measure the size of the fiber free areas is reasonably accurate and the linear elastic fracture mechanics assumptions are a good approximation for this cracking process. Fiber dispersion affects the toughness of the composites by its role in transferring the load to the other parts of the specimen. An effective crack bridging and increase in the toughness of the composite can be achieved if the fiber dispersion is better at the first crack location.

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Table 1. Cracking stresses, maximum stresses and corresponding displacements of the composites.

Specimen number	Crack number	Cracking stress (MPa)	Max. fiber-free area size (μm)	Displ. at first crack (μm)	Max. stress (MPa)	Displ. at max. stress (μm)	Toughness ratio	Clumping at first crack (%)
S1	1	25.5	381	16.3	36.0	88.6	5.4	47
	2	28.1	271					
	3	29.3	226					
	4	29.3	223					
	5	29.7	219					
	6	31.1	209					
S2	1	26.2	393	15.5	30.2	54.0	3.5	56
	2	27.1	316					
	3	28.0	284					
	4	29.2	232					
S3	1	26.8	400	20.7	37.4	102.7	5.0	48
	2	28.2	310					
	3	35.3	219					
S4	1	30.2	342	26.0	37.2	83.4	3.2	60
	2	33.1	284					
S5	1	25.6	335	20.4	34.5	193.0	9.5	11
	2	32.8	209					

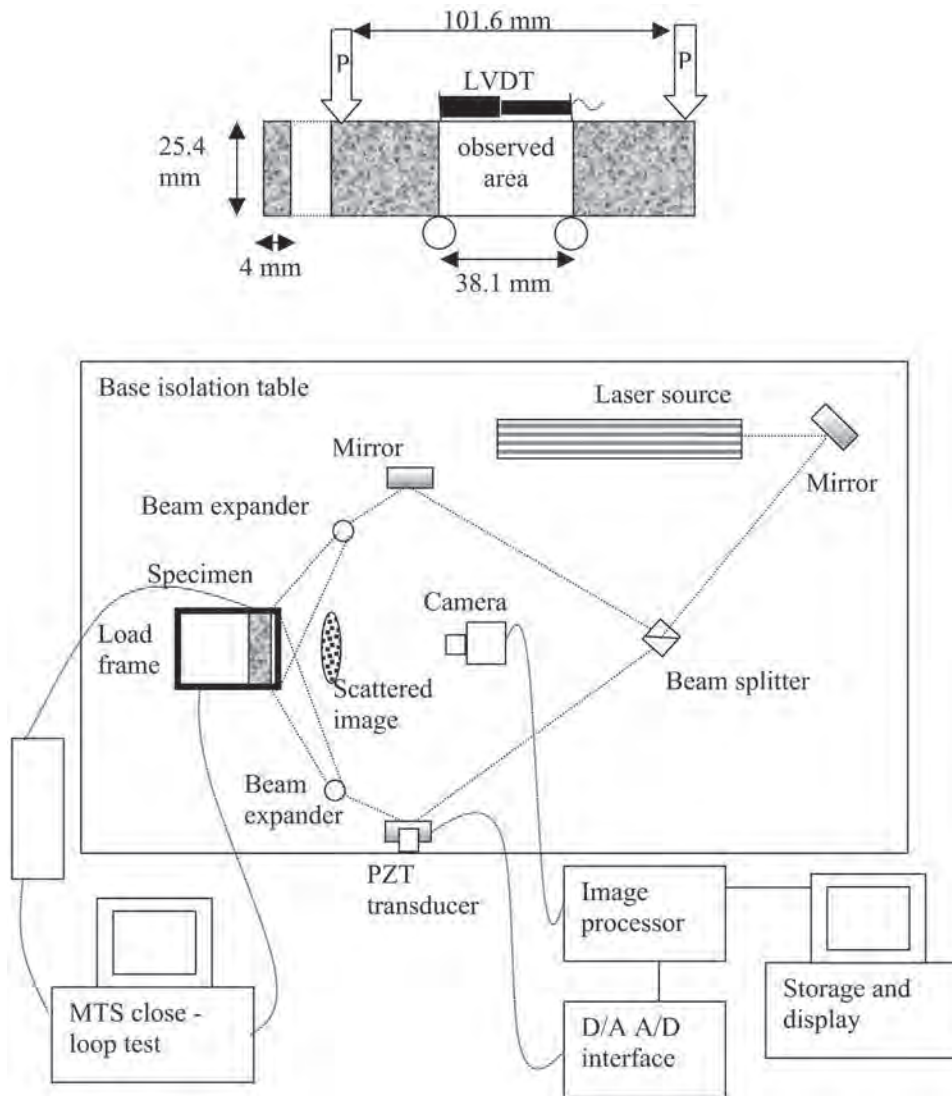


Figure 1—Dimensions of the test specimen and experimental set-up of ESPI technique.

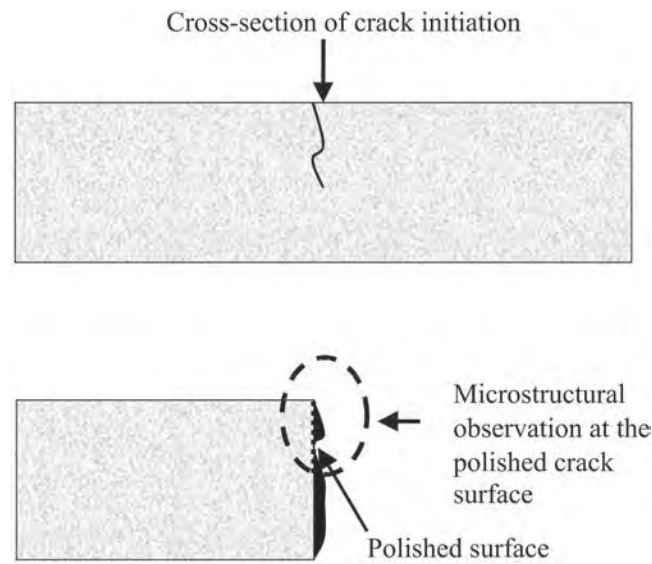


Figure 2—Location of the polished crack cross-section of the composite.

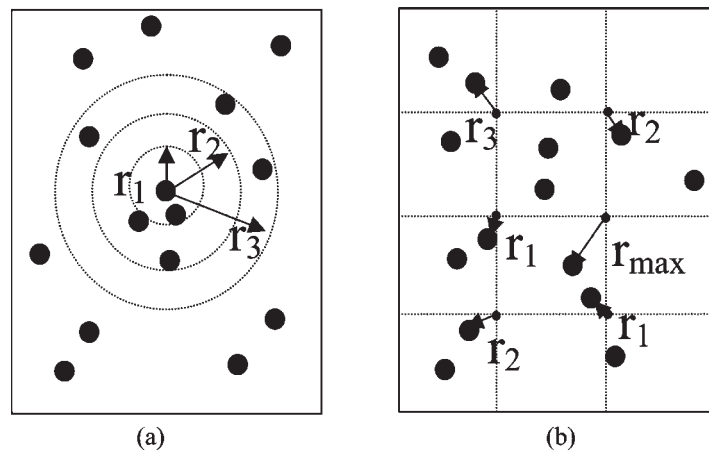


Figure 3—Calculation of (a) K-function (b) F-function.

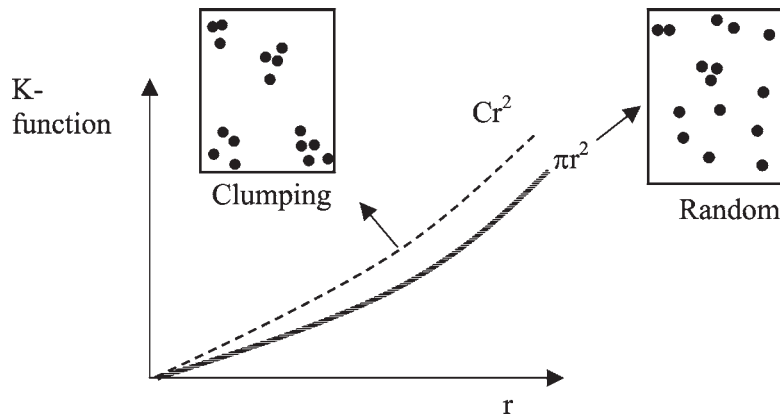


Figure 4—Comparison of clumping and random dispersion.

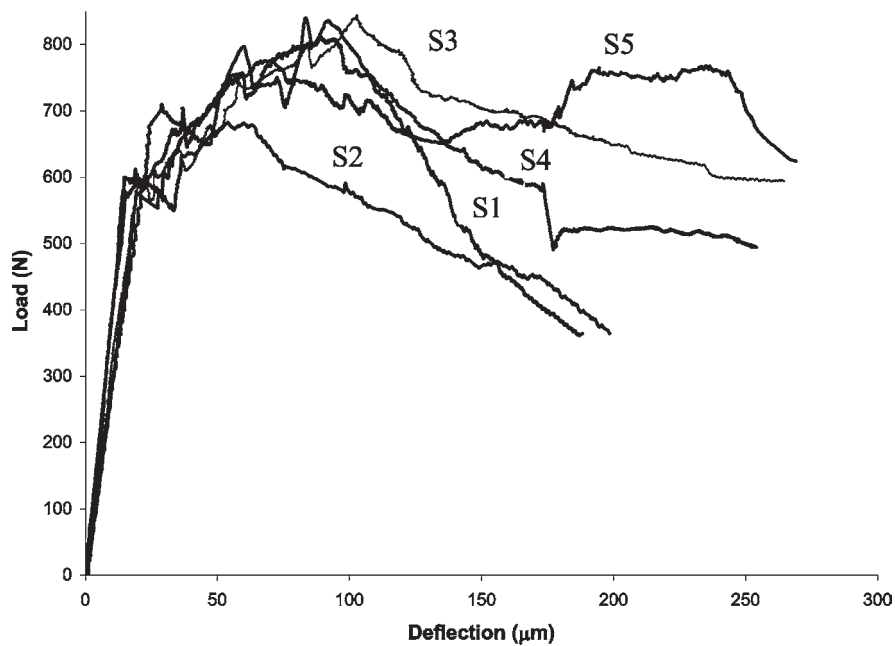


Figure 5—4-point flexure test results of the identical composites.

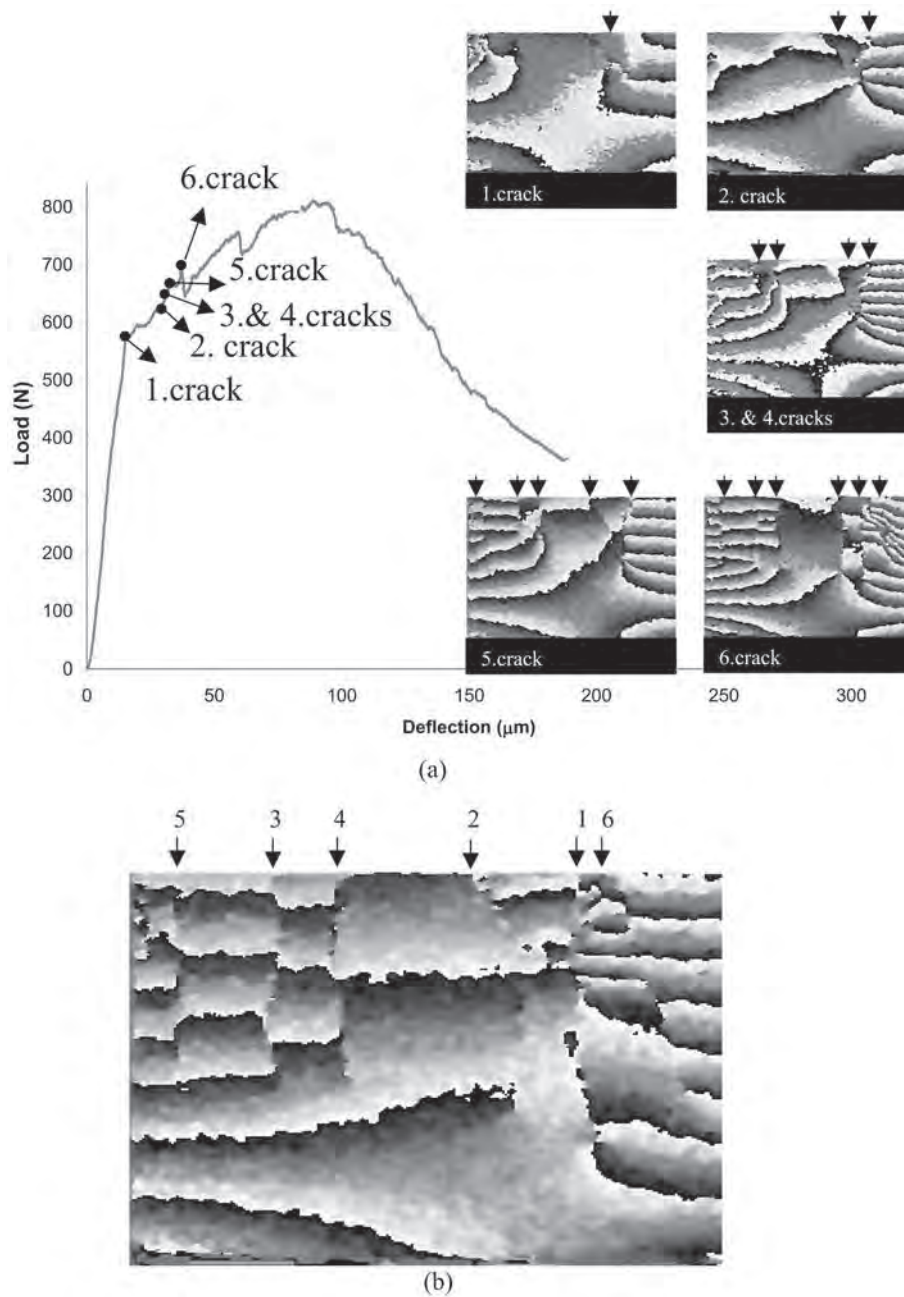


Figure 6—(a) ESPI pictures of the crack locations of a sample (b) location and sequence of the cracks.

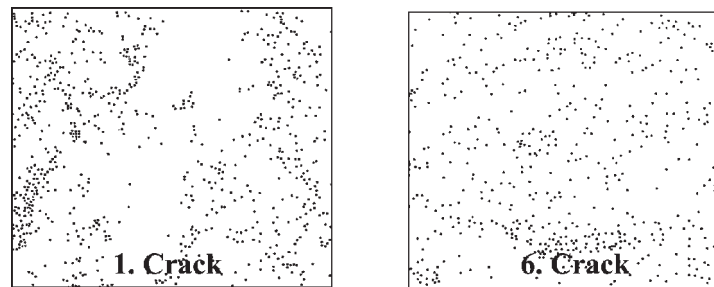


Figure 7—Pictures used in the image analysis program for specimen S1. (x50)

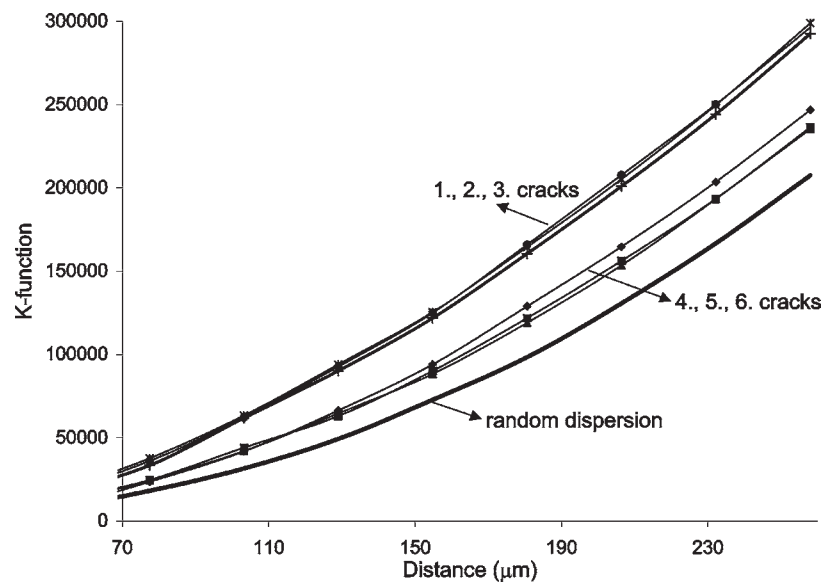


Figure 8—K-function of the fiber dispersions at the crack cross-sections and random dispersion (Specimen S1).

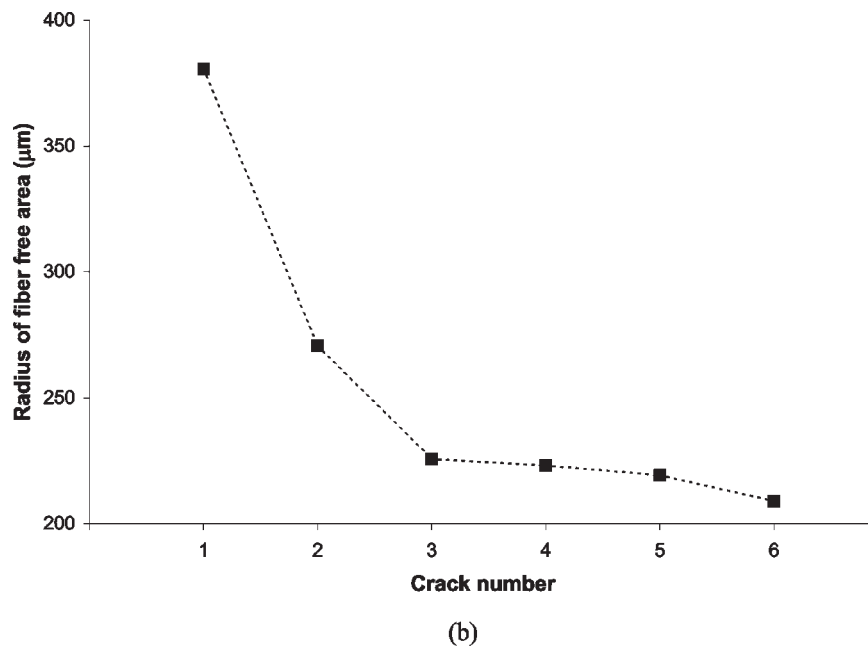
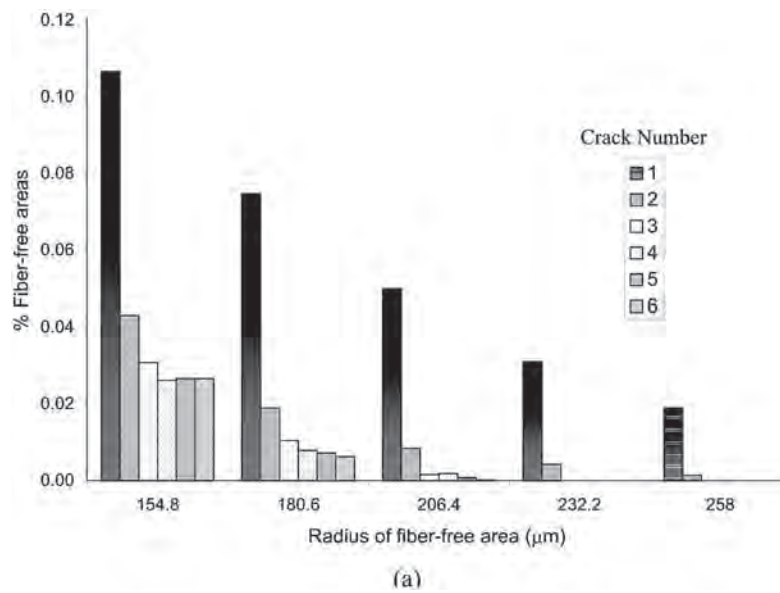


Figure 9—(a) (1-F)-function values of the sequential cracks (b) comparison of the largest fiber free area at the crack locations (Specimen S1).

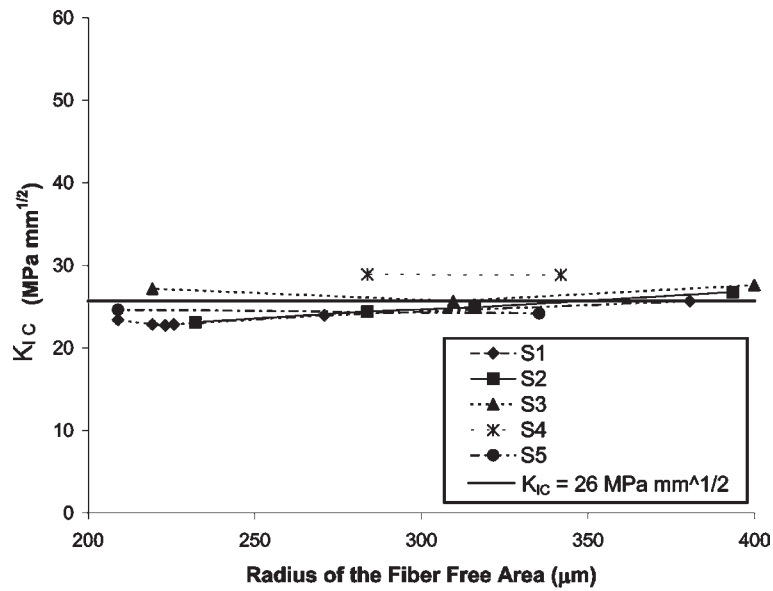


Figure 10— K_{IC} versus defect size of the specimens.

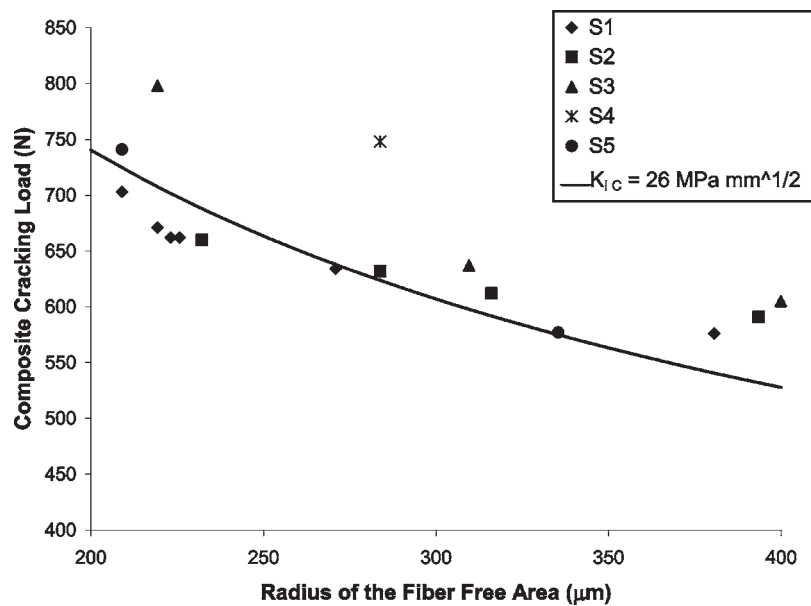


Figure 11—Comparison of theoretical and experimental cracking stress vs. defect size.

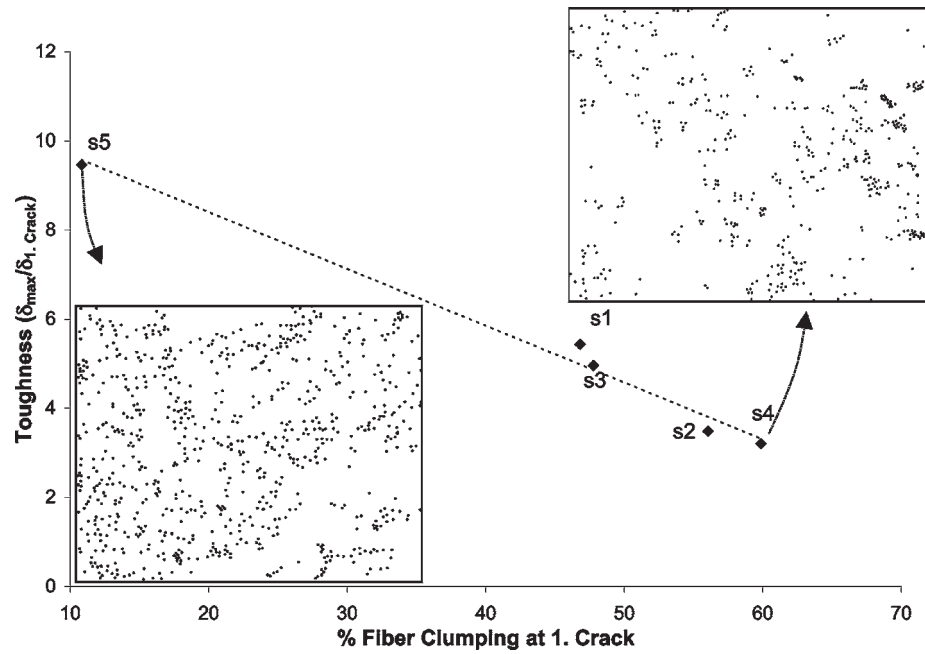


Figure 12—Effect of fiber clumping at the first crack location on the specimen toughness.

Effect of Specimen Size on the Compressive Response of High Strength Concrete Containing Fibers and Spiral Steel Reinforcement

by G. Campione and S. Mindess

Synopsis: The purpose of the present investigation was the study of the effect of specimen size on the compressive strength of concrete reinforced with steel spirals and with fibers. Concrete cylinders of different sizes were loaded under uniaxial compression. The cylinders, ranging in size from 60×120 mm to 100×400 mm, were reinforced with short fibers (steel and polyolefin) at volume percentages of 1% and 2%. Some specimens also contained transverse steel reinforcement (ϕ 5 mm) at pitches of 25 and 50 mm. There was a reduction in compressive strength as the specimen size increased in all cases (plain concrete, fiber reinforced concrete or spirally reinforced concrete). The effect of the fibers and of the steel spirals was a reduction in the brittleness of composite with increasing size of the specimens. The experimental results confirm that the size effect laws proposed in the literature for compressive strength provide a satisfactory prediction of the reduction in compressive strength with increasing specimen size.

Keywords: fiber-reinforced concrete; high strength concrete; polyolefin fibers; size effect law; steel fibers; steel spirals

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INTRODUCTION

In quasi-brittle materials such as concrete, especially in the case of high strength concrete, if there is inadequate lateral confinement, brittle splitting tension failure in compression and a very significant size effect are observed¹⁻²⁻³. These aspects are very important, especially in seismic areas and for high strength concrete, where very high percentages of lateral and longitudinal reinforcement are necessary⁴⁻⁶ to ensure ductile behavior.

Recent studies have shown that by adding randomly distributed short fibers of appropriate geometry at adequate volume percentages to the matrices, it is possible to improve the behavior of plain and conventional reinforced concretes, in terms of both ductility and residual strength⁷⁻⁹. The confinement effect exercised by the fibers due to their pull-out resistance ensures ductile behavior, reducing the quantity of lateral steel reinforcement required.

The purpose of the present study is to investigate the effects of different types of fibers, and of transverse steel reinforcement, on the apparent reduction in compressive strength as the specimen size increased (i.e. the size effect), and also to investigate the post-peak response of these materials in compression.

PARAMETERS AFFECTING COMPRESSIVE RESPONSE

The main parameters affecting the compressive strength and the softening of a concrete specimen are¹⁰: frictional restraint between the loading platen and the specimen; the allowable rotations of the loading platens before and during the test; the stiffness of the testing machine; the type of feed back signal; the loading rate and the shape and the size of the tested specimen.

The standard tests generally involve loading the specimen between rigid steel loading platens, but it would be preferable to reduce to a minimum the friction between the loading platens and the specimen. When steel platens are used, as is prescribed in most test methods, confined zones develop in the parts of the specimens in contact with the loading platens. This is due to frictional restraint that builds up during the test.

The shear stresses between the loading platens and the concrete specimen are caused by a mismatch in the lateral expansions (due to different Poisson's ratios, ≈ 0.33 for steel and ≈ 0.20 for concrete) and stiffness of the loading platens and specimens. The confined zones lead to a higher measured strength at low h/d (height to diameter) ratios of the

cylindrical specimens, because the restrained zones then include most of the specimen. The apparent compressive strength of the concrete thus increases compared to the case of much reduced friction and also increases when the slenderness of the specimens decreases.

When lubrication between the loading platens and concrete using teflon and grease is provided, this increase of specimen strength with reduced slenderness ratio does not occur; rather, a constant strength is measured¹¹ However, if too much grease or a thick teflon layer are applied, the apparent strength may decrease due to tensile splitting forces at the top and bottom of the specimen¹⁰ (Figure 1).

If the testing machine at the end platen is fixed against rotation once significant load is applied to the specimen some rotation of the upper platen is permitted prior to this, as the specimen ends are rarely perfectly parallel. Some cross-head deflection is also needed to ensure full contact between the loading platens and the specimen. Often, capping of the specimens ends is used to ensure plane and parallel ends.

In many cases axial deformations are recorded using LVDTs. The gauge length of the control LVDTs in a conventional displacement controlled test is chosen to be equal to the specimen length (i.e. from platen to platen).

Regarding the testing procedure, a more accurate control test control for concrete in uniaxial compression would be a closed-loop, servo-controlled loading system. In this case the axial deformation cannot always be used as feed-back signal¹⁰, particularly in the case of high strength matrices that release a large amount of energy during failure, and where snap-back behavior can occur, leading to unstable regimes in the softening branch.

In these cases, some investigators¹¹⁻¹² have proposed lateral deformation instead of axial deformation as the feed-back signal. Unfortunately, a closed-loop servo-controlled loading system is not always available, and very often open loop testing machines, with displacement control, are used to test concrete in compression.

Depending on the rate of displacement chosen for the test, on the stiffness of the testing machine, on the size of concrete members and on the strength of the material, it may be possible to perform a stable test, but if snap-back occurs no strain-softening branches can be recorded. However, if the material also exhibits ductile behavior in the softening branch of the response (concrete confined by transverse steel reinforcement or fiber reinforced concrete at a high volume fraction of fibers) it is possible to obtain a stable post-peak response by utilizing an open loop testing machine with displacement control and with a very slow rate of displacement.

EXPERIMENTAL PROGRAM

Material properties

High strength matrices were used to investigate the influence of different specimen sizes and variations in the volume percentages of fibers, with and without traditional spiral steel reinforcement on the compressive strength. The high strength concrete (HSC) mix designs previously investigated by the authors⁸ yielded 28-day compressive strengths of 70 MPa, measured on 100 x 200 mm cylinders, and had 3.61 MPa direct tensile strength. To produce the HSC matrix, the following mix proportions were used:

- 400 kg / m³ of CSA Type 10 (ASTM Type I) Portland cement

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- 1050 kg / m³ of 10 mm maximum size coarse aggregate (grain size range 5÷10 mm)
- 55 kg / m³ of silica fume
- 720 kg / m³ of fine aggregate (grain size range up to 5 mm)
- 150 l / m³ of water
- 8 kg / m³ of superplasticizer
- $w / (\text{cementitious material}) = 0.33$

Three different types of fibers, straight polyolefin, crimped steel and hooked-end steel, were used at volume percentages of 1% and 2%. The properties of the fibers are given in Table 1. Sets of three cylindrical specimens (for a total of 66 specimens) were cast for the different mixes (plain concrete and fiber reinforced concrete) and sizes investigated.

The dimensions of the cylindrical specimens were 60 × 120 mm, 100 × 200 mm, 150 × 300 mm and 100 × 400 mm.

Some specimens (of 100 × 200 mm and 100 × 400 mm) were also reinforced with steel spirals of 5 mm diameter, and with a yield stress of 550 MPa. The spirals were placed with a cover of 3 mm, at pitches of either 25 or 50 mm and they were left loose at the ends.

Test set-up

For the tests described here, a single size of rigid steel platens with a diameter considerably larger than the specimens diameters was used. This would lead to greater confinement for the specimens with smaller diameters, due to the additional frictional forces compared to the use of steel platens having the same diameter as the specimens (Fig. 2). The test arrangement used would also lead to a reduction in apparent strength for specimens with greater length/diameter ratios (less of the specimen volume experiencing confining stresses).

Specimens were tested using an open loop testing machine with a maximum load capacity of 1600 kN at a slow rate of displacement. The testing machine was equipped with a spherical joint for the upper platen, ensuring uniform contact between the loading platens and the specimens. The specimen ends were prepared using an end grinder. To obtain accurate load vs. deformation curves (avoiding any extraneous deformations at the specimen ends), three LVDTs were mounted on special metal rings fixed to the specimens in their mid-portions, with a gauge length of 100 mm. Similar procedures were also utilized in JSCE-SF5¹³.

Three additional LVDTs were mounted using a gauge length equal to the full length of the specimens to measure the “total” deformation.

Figure 3 shows the σ - ϵ curves was obtained from these two different gauge lengths. It may be seen that, with the longer gauge length, the strain corresponding to the peak stress is shifted to the right due to initial settlement of the setup. It should be noted that the deformations obtained using the LVDT's measuring the platen-to-platen deformation are affected in the ascending branch of the curve by extraneous deformations at the contact between the platens and the specimen ends and the minor deformation of the platens themselves. However, beyond the peak load, the platen-to-platen measurement becomes the same as those measured on the central gauge length (1/2 of the specimen

length) measurements. The disadvantage of the measurements based on only the central portion of the specimen is that they do not take into account strain localization outside of this region. In the data that are presented below, the full platen-to-platen deformations are used after linearization of the initial part of the curve.

EXPERIMENTAL RESULTS

The curves shown below represent the average values of three tests for each series investigated. The stresses are calculated on the basis of the initial transverse cross-section, while the strains are evaluated on a gauge length equal to the entire length of the specimens for the different sizes. For the plain concrete, the testing machine could not provide a stable post-peak response and thus only the ascending portion of the load vs. deflection curve could be obtained. However, for the fiber reinforced or spiral reinforced concrete specimens, stable curves were obtained in the softening branch of the response also, because of the combination of the high stiffness of testing machine, the slow rate of displacement, the dimensions of the specimens and the pseudo-ductility of the materials.

Plain concrete

The nominal stress-strain curves for the different size specimens of plain concrete are shown in Fig 4. With an increase in the size or the slenderness of the specimens, HSC appears to exhibit a more brittle behavior, and the strength decreases.

These results are consistent with those reported in a RILEM Round Robin test series¹⁰, carried out on both prisms and cylinders, with considerable friction between the specimen ends and the loading platens. They too found a reduction in apparent strength for rigid platens as the slenderness (height/diameter) increased. There were no significant differences in behavior when the platens were lubricated for h/d ratios up to 3.0. Similar results for specimen tested with reduced end friction were found by Jansen and Shah¹¹ and Jansen et.al¹².

For the smallest specimens (60 x 120 mm), because of a combination of the wall effect and the fact that the diameter of the loading platen was substantially greater than that of the specimen, the measured strengths are much higher than would be expected from size considerations alone, and so these results must be treated with great caution.

Plain concrete reinforced with steel spirals

Stress-strain curves for HSC specimens of different sizes (i.e. 100 × 200, 100 × 400 mm) reinforced with steel spirals at pitches of 25 and 50 mm (corresponding to volumetric ratios of $\rho_s = 0.01765$ and 0.03530, respectively) are shown in Fig. 5. For these specimens the stress values are obtained by dividing the actual load by the cross-sectional area of the concrete core (inside the spirals). Curves for unreinforced specimens are shown for comparison.

The higher volume fraction of transverse steel reinforcement clearly provided more effective confinement, leading to both a higher apparent compressive strength, and a somewhat reduced size effect in terms of peak strength. Also, a more brittle failure (steeper slope in the descending branch of the σ - ϵ curves) was observed for the more slender specimens.

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The mode of failure of the spirally reinforced specimens is shown in Fig.6. In all cases failure was characterized by the formation of an inclined shear failure plane, separating the concrete core into two wedges laterally confined by the steel spiral. The confining pressure was not constant, as it depended on the lateral dilation of the concrete and the stress-strain relationship of the confining steel. During the tests, it was observed that the inclination angle of the failure plane varied with the effective percentages of spiral steel, but it seemed to be independent of the size of specimens. After the complete formation of the failure plane, the steel spirals became heavily stressed and when the ultimate strain in the spirals was reached, the specimens failed in a brittle manner.

Fiber reinforced concrete

Typical results of tests on HSC containing 2.0% fibers by volume are shown in Fig. 7 for steel fibers and Fig. 8 for polyolefin fibers. For FRC also, a size effect was observed in terms of both maximum strength and energy absorbed to failure.

For the FRC with both types of steel fibers (fibers with high modulus of elasticity and tensile strength), more residual strength and energy absorption were observed than with the polyolefin fibers (fibers with reduced mechanical properties).

For the smallest (60 mm diameter) cylinders, the ratio of cylinder diameter/fiber length was 2, leading to much more bridging capacity of internal cracks. This along with the wall effect and the effect of end confinement mentioned previously, led to much higher apparent strengths than would be expected from size considerations alone; thus, as in the case of plain concrete, these results too should be interpreted with great caution.

Table 2 gives characteristic values of strength and strain for the fiber reinforced specimens of different sizes. For concrete specimens the standard deviation of the actual value of the test was about 18% for plain concrete and 23% for FRC.

Fig. 9 shows that the fiber reinforced specimens of all sizes failed in shear. However, the precise mode of failure appeared to depend upon the frictional (shear) forces between the specimen ends and the loading platens. As the slenderness of the specimens increased, the portion of the specimen experiencing both confining stresses and axial stresses decreased, as shown schematically in Fig. 10.

For the 2% by volume fiber reinforced concrete, size effect in terms of residual strength (or softening response) was observed, but the shape of the softening branch appeared to be independent of the size of the specimens tested.

ANALYTICAL EVALUATION OF COMPRESSIVE STRENGTH

Many experimental and theoretical investigations have been carried out to determine the reduction in measured strength of cylindrical and prismatic concrete specimens as their size increases. The reduction in strength, due to the increasing size of concrete specimens in tension can be predicted very accurately using the size law proposed for plain concrete in tension by Bazant¹ :

$$\sigma_N = B \cdot \sigma_0 \cdot \left(1 + \frac{D}{D_0}\right)^{-\frac{1}{2}} \quad (1)$$

where B and D_0 are two empirical constants, σ_0 is the tensile strength, and D is the transverse dimension of the specimens (specimens similar geometrically).

Based on a fracture mechanisms type of approach, Kim et al.¹⁴ have recently presented both theoretical and statistical analyses to predict the compressive strength of cylindrical specimens with various diameter and height / diameter ratios. This model was then extended to prediction of the compressive strength of spirally reinforced concrete cylinders. The modification is based on the concept that when a confining stress is applied to concrete, the effect of size on the compressive strength would be less than for plain concrete. The trend of the size effect could also be influenced by the degree of confining stress (or volumetric spiral steel). For a concrete cylinder confined with heavy spiral reinforcement (high confining stress), the size effect should then be largely eliminated because the concrete will behave in a pseudo-plastic manner. According to the Kim et al.¹⁴ model, for plain concrete cylinders reinforced with steel spirals the reduction of compressive strength f_c' with increasing specimen size can be predicted by:

$$f_0 = 0.8 \cdot f_c' + \frac{0.4 \cdot f_c'}{\sqrt{1 + \frac{(h-d)}{50} \cdot \left[1 - \frac{8000 \cdot A_{sp} \left(1 - \frac{s}{d_c} \right)}{d_c \cdot s \cdot f_c'} \right]}} + \frac{5.4 \cdot A_{sp} \cdot f_y}{d_c \cdot s} \cdot \left(1 - \frac{s}{d_c} \right) \quad (2)$$

where A_{sp} is the cross-sectional area of spiral reinforcement, f_y is the yield strength of spiral reinforcement, d_c is the outer diameter of spiral reinforcement, s is the spacing of the spiral reinforcement, and d and h are respectively the external diameter and the length of the specimen.

The size effect law (Eq.2) and the experimental results for plain concrete and FRC are plotted in Fig. 11, while the data for concrete reinforced with steel spirals are plotted in Fig. 12. Because of the uncertainties mentioned earlier, the data for the 60 mm diameter specimens are not plotted here. For HSC plain concrete and the FRC, there is an acceptable scatter between the experimental data and the predicted values; the data are also in agreement with the upper limit defined by ASTM¹⁶.

It should be noted that Eq. 2, based on non-linear fracture analysis, combined with a statistical analysis of the plentiful data available in the literature, does not take into account the effects of concrete strength or of maximum size of aggregate. These are considered to be negligible effects.

Eq. 2 also implies that as the volume of spiral steel is increased, the magnitude of the size effect is reduced; the minimum spiral steel volume required to eliminate the size effect seems to increase with increasing concrete strength (Fig. 12). It is of interest to note that Eq. 2 assumes that, at failure, the lateral pressure on the concrete core is that corresponding to the yield stress in the steel spirals. This is, indeed, generally true for normal strength concrete reinforced with a high volume percentage of spiral steel.

However, for HSC matrices, it has been observed¹⁶ that at failure the stress in the steel spirals is lower than the yield stress, and therefore assuming yielding in the spirals

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at failure overestimates the compressive strength (compared to that determined experimentally). Thus Madas and Elnashai¹⁷ have developed an analytical procedure to estimate the effective stress in the steel spirals at failure; Eq. 2 should probably be corrected to take this effect into account. As may be seen from Fig. 12, the theoretical curve for a high volume fraction of spiral steel ($\rho_s = 0.03530$) considerably overestimates the experimental results obtained in the present study. This shows the necessity of evaluating more accurately the stress in the steel spirals at failure.

CONCLUSIONS

The results show that:

1. The apparent compressive strength of high strength, fiber reinforced concrete cylinders subjected to concentric loads and with steel end platens varies with the size of the specimen;
2. For both plain concrete and for FRC a reduction in the apparent strength was observed with an increase in the size of the specimens; there was also a moderate reduction in residual post-peak strengths with an increase in the size of the FRC specimens;
3. With steel spirals, a much smaller size effect on strength was observed;
4. Significant differences on the localization and on the failure mode for cylinders of plain concrete, FRC or confined concrete with spiral steel were observed: wider cracks and splitting tension failure in compression for plain concrete; more fine cracks and mixed mode between twin cone and shear modes of failure for FRC; and finally the formation of an inclined shear failure plane, separating the concrete core into two wedges laterally confined by the steel spiral for spiral steel confined cylinders;
5. The analytical model recently proposed in the literature¹⁴ for both plain concrete cylinders and those reinforced with steel spirals was able to predict the reduction in strength due to the increase in the size of the specimens for plain concrete, FRC, and plain concrete containing a relatively low volume of spiral steel. It greatly overestimated the experimental results when a high volume of spiral steel was used.

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Table 1 Mix Proportions

Specimen	Mix Proportion by Weight Cement [‡] : Sand : Stone* : Water	Static Compressive Strength (MPa)	Steel Fiber Content —————→	Polypropylene Fiber Content ~~~~~
Control	1 : 2 : 2 : 0.45	38		
SFRC-1	1 : 2 : 2 : 0.45	43	1 % V _f	
SFRC-2	1 : 2 : 2 : 0.45	46	2 % V _f	
PFRC-1	1 : 2 : 2 : 0.45	40		1 % V _f
PFRC-2	1 : 2 : 2 : 0.45	44		2 % V _f

[‡] CSA Type 10

* 6-12 mm crushed aggregate

Table 2 Dynamic Fracture Properties

Mix Description	Peak Load (kN)				Fracture Toughness (MPa√m)			
	QS [§]	Impact Drop Height			QS	0.5 m	0.75 m	1.0 m
		0.5 m	0.75 m	1.0 m				
Plain Concrete	0.61	9.48	12.45	13.71	0.50	7.06	18.67	7.02
SFRC 1	0.98	13	22.23	20.08	0.95	12.78	20.98	13.48
SFRC 2	1.47	14.24	36.35	27	1.37	13.41	35.28	19.68
PFRC 1	0.78	11.78	10.62	14.63	0.52	6.70	10.31	13.03
PFRC 2	1.01	15.82	16.2	22.98	0.57	9.53	14.81	18.60

[§] Quasi-Static Loading

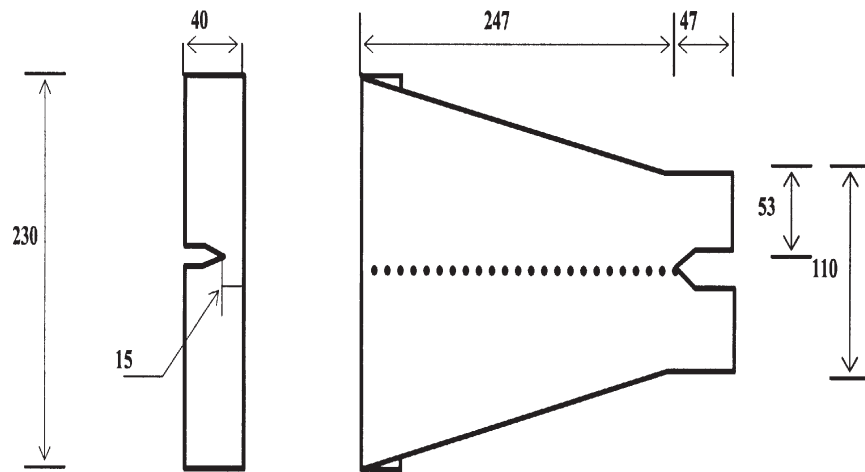


Fig.1—Effect of different end condition.

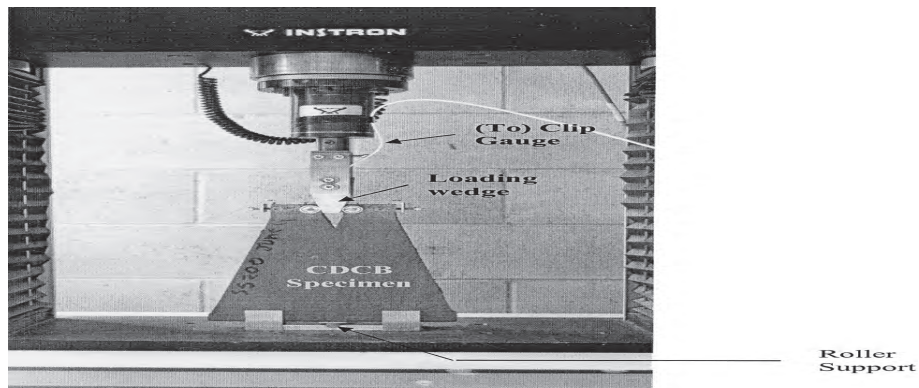


Fig. 2—Effect of different steel plate dimensions.

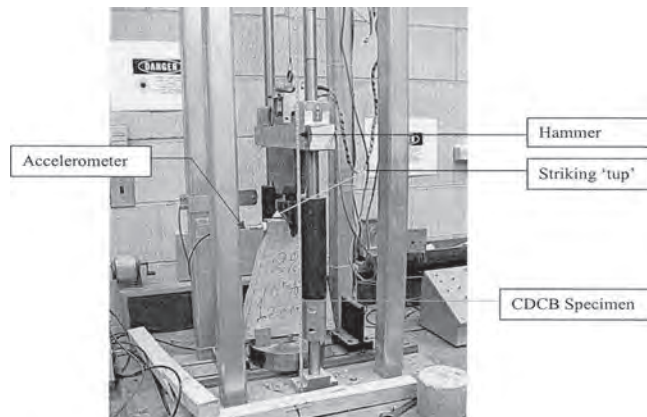


Fig. 3—Effect of gauge length on the compressive response of specimens.

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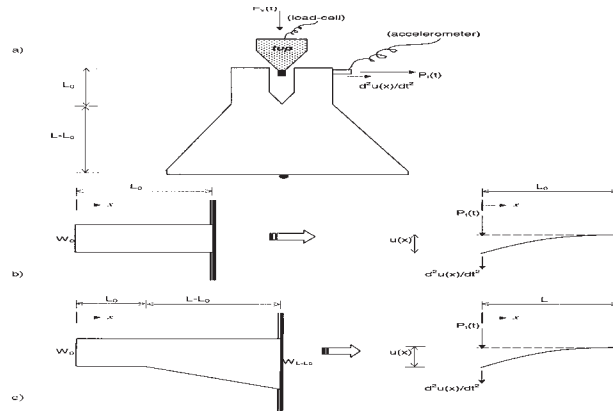


Fig. 4—Stress-strain curves for HSC specimens in compression.

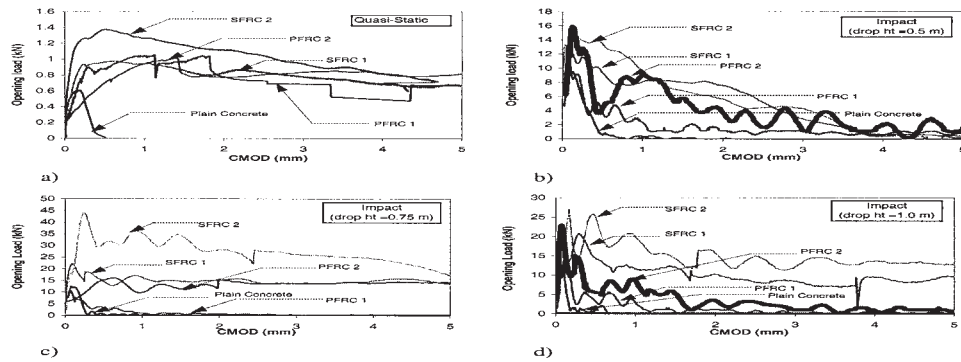


Fig. 5—Stress strain curves for HSC specimens with steel spirals:
(a) 50 mm pitch; (b) 25 mm pitch.

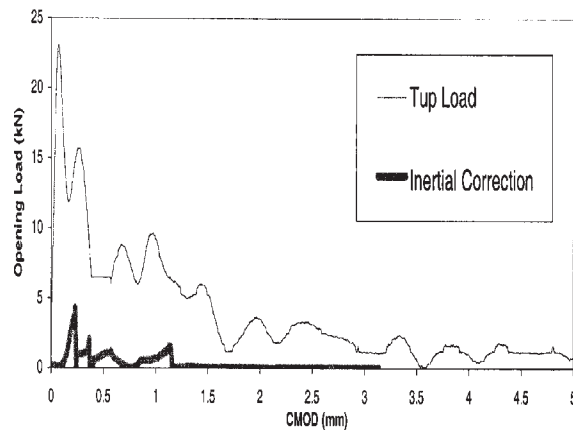


Fig. 6—Mode of failure of HSC with spirals.

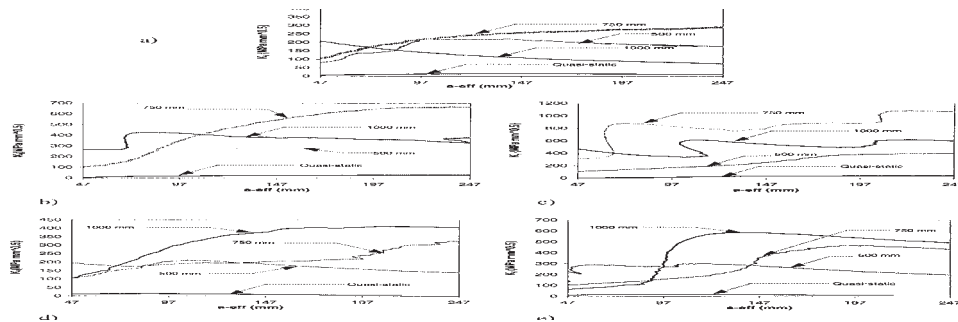


Fig. 7—Stress-strain curves for HSC with steel fibres at 2% by volume percentage.

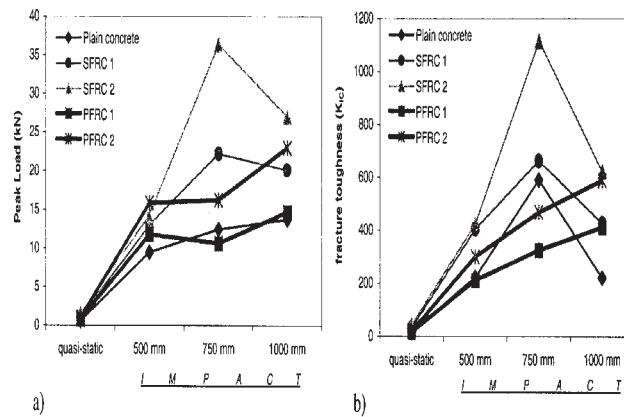


Fig. 8—Stress-strain curves for HSC with polyolefin fibres at 2% by volume percentage.

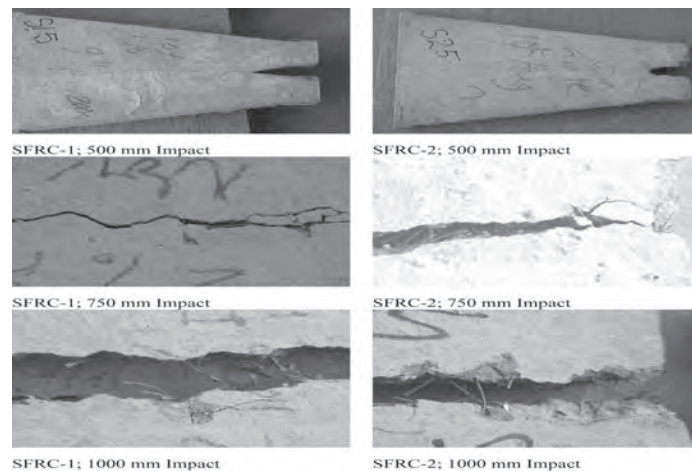


Fig. 9—Mode of failure of HSF fiber reinforced cylinders with 2% hooked steel fibres.

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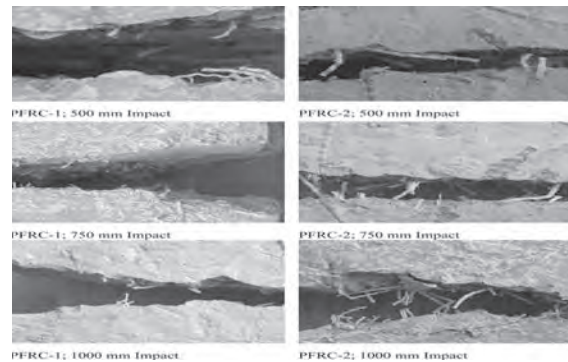


Fig. 10—Effect of the size of the specimens on the confinement effect (larger specimens have a smaller relative volume of concrete affected by confining stresses).

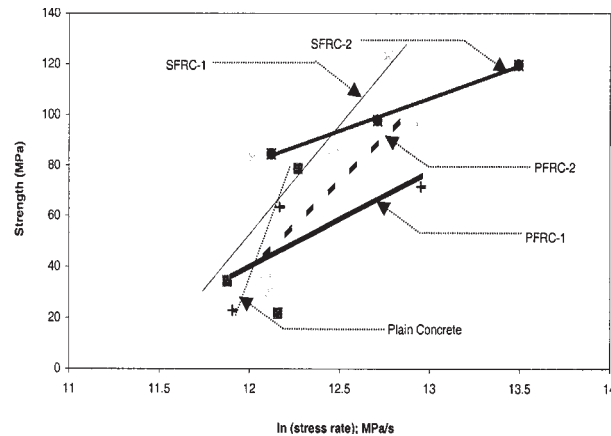


Fig. 11—Size effect law for plain concrete and FRC with 2% fibers.

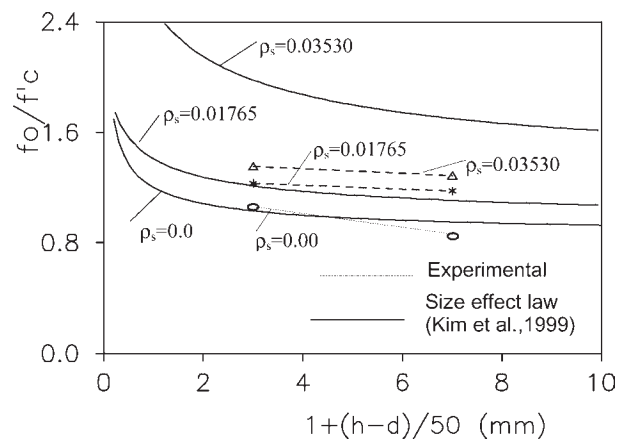


Fig. 12—Size effect law for plain concrete with steel spirals.

High Strength Lightweight Aggregate Fiber-Reinforced Concrete (HS-LWA FRC) Filled Steel Tube Columns for Increased Seismic Resistance

by J. Punchín, N. Krstulovic-Opara, and B. Brezac

Synopsis: Damage due to recent earthquakes and the high costs of repairing old and building new structures emphasizes the need for novel ways of designing cost-effective and more seismically resistant buildings. To address this problem, the structural system considered in this paper is constructed with composite materials that display excellent earthquake-resistant properties such as high strength, high ductility and increased energy dissipation capacity. The particular composite used in this project is High Strength-Lightweight Aggregate Fiber Reinforced Concrete (HS-LWA FRC). Its use in concrete-filled steel tubular (CFT) columns allows for both: (a) improved seismic resistance and (b) possibly a faster and more cost-effective method of construction, than is the case with conventional buildings. The main goal of this research was to manufacture and study the seismic performance of such a composite CFT column. The ductility, stiffness degradation, energy dissipation and hysteretic response of the CFT member are reported. Testing of the CFT revealed that adequate strength was developed in the critical section, with respect to the predetermined yielding of the system. The use of fibers prevented an otherwise extremely brittle failure of HS-LWA concrete and increased its capacity for lateral expansion, leading to a ductile response of the critical column zones. As a result, large levels of ductility and energy dissipation capacity were measured during testing.

Keywords: concrete filled tubes; ductility; energy methods; fiber-reinforced concrete; high-performance construction materials; metal fibers; stiffness

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INTRODUCTION

A promising way of cost-effectively increasing load-capacity, durability, seismic resistance and speed of construction of new buildings is to selectively use advanced composites, such as High-Performance Fiber Reinforced Cementitious Composites (HPFRCCs). This is the main objective of the ongoing National Science Foundation (NSF) project conducted at North Carolina State University in which parts of a large scale High Performance Fiber Reinforced Concrete (HPFRC)-based beam-column sub-assembly, shown in Figure 1, are being developed and tested. Such a composite frame system, termed herein High-Performance Composite Frame (HPCF), is made by selectively using Slurry Infiltrated fiber-Mat Concrete (SIMCON), Slurry Infiltrated Fiber Concrete (SIFCON), and High Strength - Lightweight Aggregate Fiber Reinforced Concrete (HS-LWA FRC). No conventional concrete materials are used. HPCF consists of three main elements: (1) a composite Concrete Filled Tube (CFT) column member filled with HS-LWA FRC, (2) a composite beam member made with a SIMCON "jacket" and (3) a SIFCON "fuse" connecting the two, as shown in Figure 1. This paper presents the behavior of the high-performance composite CFT column members.

If adequately confined, HS-LWA concrete is ideally suited for column members.^{1, 2, 3} To maximize confinement and thus maximize column strength and ductility while at the same time eliminating reinforcement congestion, fibers were added to the HS-LWA concrete and the resulting HS-LWA FRC was encased in a steel tube, as shown in Figure 2. To further improve seismic behavior of the column member, as well as increase its fire resistance and durability, SIMCON jacketed CFT columns will also be considered in the near future. This paper presents the behavior of the reference column made without the SIMCON jacket.

Background on Concrete Filled Tubular Members

Concrete filled steel tubes have been utilized as columns in numerous applications, such as for supporting platforms in offshore structures or the roofs of storage tanks, bridge piers, piles, and earthquake-resistant structures.⁴ While concrete was primarily used in most of these applications, steel tubes filled with FRC were used in a 60 m (197 ft) span spaced truss bridge recently built in Quebec, Canada. In this case, high-performance, steel fiber reinforced reactive powder concrete (FR-RPC) was pressed into steel tubes which were used as diagonal members of the truss bridge. The compressive strength of the confined and pressed FR-RPC reached 54,400 psi (375 MPa). In 1987, Narayan⁵ also proposed an innovative double skin composite construction for submerged tube tunnels, where concrete is confined between two tubular steel sections.

Construction advantage of CFTs is that the steel tube acts both as a permanent formwork, and provides well-distributed lateral reinforcement.^{4, 6-8} Structural advantage of CFT members is that their composite strength is larger than the sum of the individual load capacities of the steel tube and the concrete core.⁴ As a result, filling the steel tube with concrete increases its (a) bending strength by about 10 to 30%, (b) initial flexural stiffness and (c) ductility, without significantly increasing member cost or size.⁹ Furthermore, due to the large shear capacity of concrete filled steel tubular members, they usually fail in ductile flexure.

The most effective CFTs are those made with circular tube sections. Circular steel sections provide both the best confinement as well as more uniform flexural strength and stiffness in all directions. Confinement effectiveness is decreased if (a) rectangular or square tubes are used, (b) high strength concrete is selected, (c) column slenderness is increased, or (d) the column is loaded in pure flexure.^{4, 6, 8, 10-14}

Background on HS-LWA Fiber Reinforced Concrete

The high strength and low weight of HS-LWA concrete leads to lower seismically-induced forces, reduced column sizes and increased usable floor space, longer spans and increased live load capacities. Incurred savings are significantly greater than the added cost of the higher quality concrete.¹⁵⁻¹⁸ However, HS-LWA concretes are very brittle, which is their key drawback when used in seismic resistant design. Therefore, special precautions must be taken when HS-LWA concretes are used in applications such as seismic design, where brittle failure must be avoided.^{19,20}

The lack of ductility can be offset using high strength lateral confinement and/or adding fibers to the mix.¹⁻³ Similarly, the addition of only 1% volume fraction of steel fibers to LWA concrete I-beams made without conventional shear reinforcement, increased the ultimate shear strength 1.6 to 2.1 times, substantially improved ductility, and first crack strength, as compared to reference lightweight beams made without steel fibers²¹. Fibers also: (1) reduced deflections, (2) prevented tensile splitting at the level of tension reinforcement, and (3) changed crack patterns to a larger number of narrower cracks.

The goals of the research presented in this paper was to (1) achieve the same increase

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in ductility as when HS-LWA concrete is confined by closely spaced interlocking spirals, while at the same time (2) eliminate construction problems caused by reinforcement congestion. Therefore, to eliminate the need for interlocking spirals, fibers were added to HS-LWA concrete resulting in the response presented in Figures 3 and 4. An additional benefit of adding fibers to the concrete mix is that it also improves the fire resistance of high-strength concrete.²²

EXPERIMENTAL INVESTIGATION

Material Manufacturing and Behavior

HS-LWA FRC mix proportions were 1/0.35/1/1.15/0.16/0.022 by weight of Type III Portland cement, total water⁺, 3/8 in (0.95 cm) Stalite light weight aggregate, sand, solid portion of microsilica slurry⁺⁺, and superplasticizer, respectively. Fiber volume fraction of 30/50 hooked Dramix steel fibers was 1.5%, the total water to cementitious material⁺⁺⁺ ratio was 0.30, and the unit HS-LWA FRC weight was 123 lb/ft³ (1.962 kg/m³). Average 4 in. x 8 in. (10.2 cm x 20.3 cm) cylinder compressive and split-tensile strengths were 15,636 psi (107.8 MPa) and 2,251 psi (15.5 MPa), respectively. Representative compressive response is shown in Figure 4.

Specimen Manufacturing, Instrumentation and Testing Procedure

The CFT specimen was made by first inserting SAE J429 shear studs into the A53 Grade B steel tube, as shown in Figure 2, followed by casting of the HS-LWA FRC core. Shear studs were used in this study to guarantee composite action between the steel tube and HS-LWA FRC core, and thus permit a clearer understanding and interpretation of the obtained data. After casting the HS-LWA FRC the top of the column was moist cured for 21 days, followed by encasing the specimen bottom into the prestressed concrete footing, as shown in Figure 5.

Instrumentation: Load, load-point displacement, as well as curvatures, longitudinal and hoop strains of eight zones shown in Figures 6 and 7, were measured during testing. Curvatures of the steel tube and curvatures of the HS-LWA FRC core were measured using potentiometers. Additional longitudinal strain gages as well as clinometers were used to monitor curvatures of the steel tube. Horizontal strain gages were used to measure hoop strains in the steel tube.

Test Setup and Testing Procedure: The specimen was tested under static reversed-cyclic loading, following the displacement history shown in Figure 8, and using the test setup shown in Figure 5. The story drifts are defined herein as the horizontal load-point

⁺ Total water is defined as water physically added to the mix as well as moisture from all components (i.e. aggregate, sand, Microsilica slurry, and superplasticizer). The weight of total water was calculated as the sum of 50% of the Microsilica slurry weight and the weight of the water added to the mix.

⁺⁺ Microsilica slurry produced by Elkem Materials, Inc. consisted of 50% solid silica fume and 50% water.

⁺⁺⁺ The cementitious material included cement and dry portion of microsilica slurry.

displacement divided by the distance between the footer and the load point displacement of 79.5 in (201.9 cm).

Specimen Response

Measured load-displacement response is shown in Figure 9. Representative moment-curvature behavior of different column zones is presented in Figure 10. Load, displacement, moment and curvature values are presented as positive when the actuator was “pushing” the column southwards, away from the reaction wall. Energy levels dissipated within each load-displacement cycle are shown in Figure 11.

The maximum average load of 53,790 lb (239,270 N) was reached in cycle 26. Beyond this cycle and up to specimen failure in cycle 30 at 52,910 lb (235,355 N), very limited decrease in load capacity took place, as shown in Figure 9. Specimen response was controlled by zone G, as anticipated. The “elephant foot”- like buckling that occurred in this zone during the 28th cycle (4.0 % drift), was followed by fracture of the steel tube in the 31st cycle (5.0% drift).

Ductility of the tested member is presented herein in terms of both the load-point displacement ductility, μ_{Δ} , of the entire member and the curvature ductility of the critical regions, μ_{ϕ} . Both ductilities are defined as ratios of displacements, i.e., curvatures, respectively, at maximum load and at yielding. Load-point displacements at yielding, maximum (i.e., ultimate) load and at specimen failure were 0.41469 in (1.05 cm), 3.2028 in (8.14 cm), and 4.008 in (10.18 cm), respectively. The resulting displacement ductility at ultimate load and specimen failure were 7.73 and 9.67, respectively. Since specimen response was governed by the failure of zone G, the highest curvature ductility of 14.73 was reached in this zone. Zones F and E reached curvature ductility of 5.55 and 3.38, respectively. (Note: length of each zone was equal to half the tube diameter.)

Dissipated Energy: The energy dissipated by the entire specimen in each load-displacement cycle, shown in Figure 11, was measured as the area enclosed within the given load-displacement hysteretic loop. The energy dissipated in selected column regions, shown in Figure 12, was measured as the area enclosed within the given moment-curvature hysteretic loop multiplied by the length of the corresponding zone. The entire specimen as well as selected regions exhibited continuously increasing level of energy dissipation throughout testing. Approximately similar energy levels were dissipated in both the critical zone G and the adjacent zone F, which indicates that the plastic hinge extended for a length equal the tube diameter away from the critical section. This is consistent with the behavior of well-designed, conventional reinforced concrete, in which case plastic hinge lengths vary between 1 to 1.5 times the member diameter. After cycles 26 and 31 local buckling affected the potentiometer readings in zone F and G, respectively, and therefore energy values could not be obtained for these cycles.

Longitudinal Strains, ϵ : For the A53 Grade B steel tube, ASTM defines strain at nominal yield strength of $\epsilon_y = \pm 0.0012$, and the nominal ultimate steel strain, ϵ_u , that ranges between 0.225 to 0.295 in/in. While the nominal yield strain was exceeded during testing, the nominal ultimate steel strain level was never reached in the tested specimen,

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as shown in Figure 13. This indicates that the steel tube fractured due to the stress concentration caused by the presence of the shear-stud holes.

SUMMARY AND CONCLUSIONS

The purpose of this investigation was to eliminate brittle behavior of HS-LWA concrete columns by both (a) adding a relatively low amount of steel fibers, and (b) confining the column with a low yield strength steel tube. The goal was to determine if this otherwise very brittle seismic response of HS-LWA concrete columns could be changed to a more desirable behavior, comparable to that of seismically designed normal-weight concrete columns.

Obtained data show a seismic response comparable to that of seismically designed columns made using conventional CFT or reinforced concrete members. Specimen response was stable until failure of the steel tube. Behavior was characterized by wide hysteretic loops and continuously increasing energy dissipation capacity. A very stable plastic hinge developed in zones F and G. Hinge length was comparable to that expected for well designed conventional CFT or reinforced concrete columns. The specimen exhibited good ductility: the column reached displacements of approximately ± 4 in (± 10.16 cm) and a story drift of 5% before a significant loss in moment capacity occurred as a result of steel fracture within the buckling zone that started during the 5% drift cycle. Measured steel strains in this zone were much lower than the minimum nominal ultimate strains of ASTM A53 steel tubes. This suggests that the tube failed due to a combination of local buckling and stress concentration caused by the shear-stud holes. However, despite the premature tube failure, the specimen still achieved high ductility, as reflected in the 5% story drift ratio and displacement ductility of 7.73 at maximum load and displacement ductility of 9.67 at specimen failure. These values very favorably compare to a drift ratio of about 4% that ACI-ASCE Committee 441 cites as a very good level of ductility.²³ Similarly, in terms of the displacement ductility, a column is generally considered ductile if displacement ductility ranges from 4 to 6.^{24,25}

In summary, obtained data indicate that HS-LWA FRC filled steel tube columns can be successfully used in seismic resistant design. Use of low fiber volume fraction of steel fibers in combination with a low-yield strength steel tube eliminates the need for closely spaced, high strength interlocking spirals. Low concrete weight also leads to a significant reduction in member mass and, thus, in the seismically induced forces. The use of steel fibers along with the steel tube increases the flexural and shear strength of the high strength-lightweight aggregate concrete core while also increasing its ductility and energy absorption capacity.

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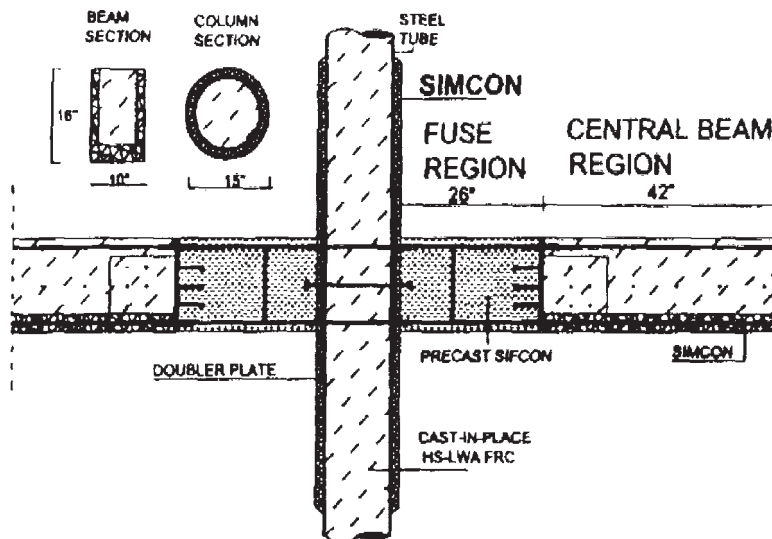


FIGURE 1: Layout of the beam-column sub-assembly of the proposed HPCF.

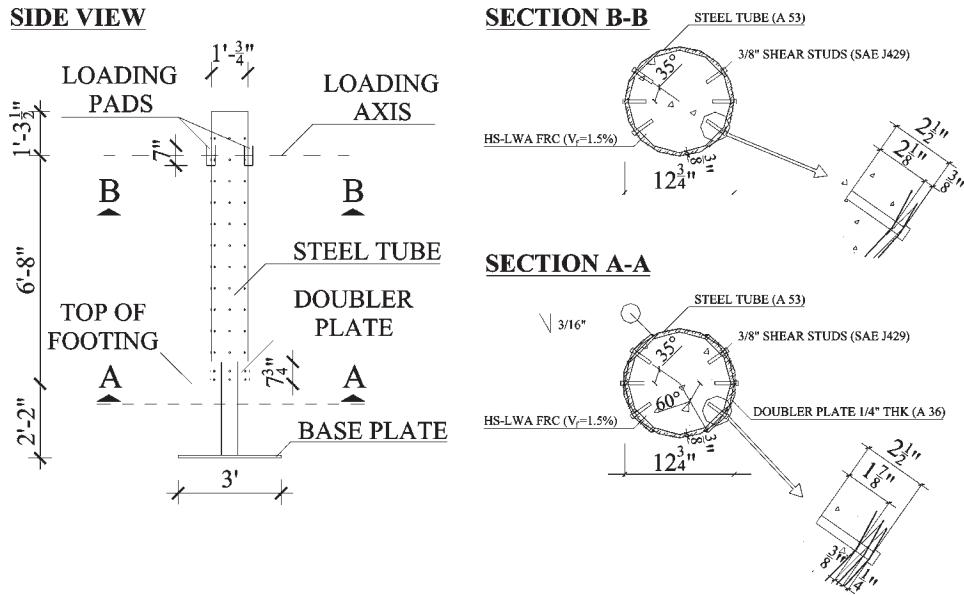


FIGURE 2: Column details.

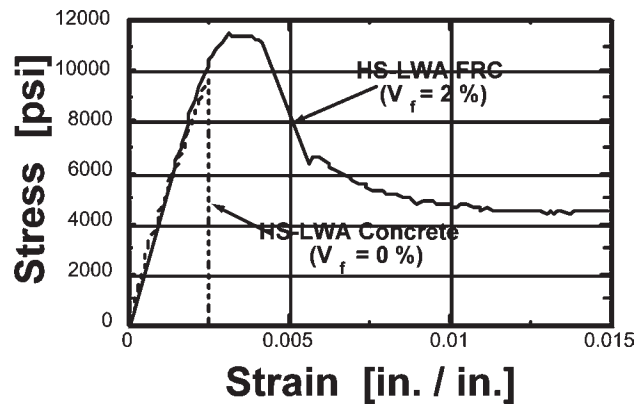


FIGURE 3: Compressive behavior of HS-LWA Concrete and LWA FRC developed in previous research.

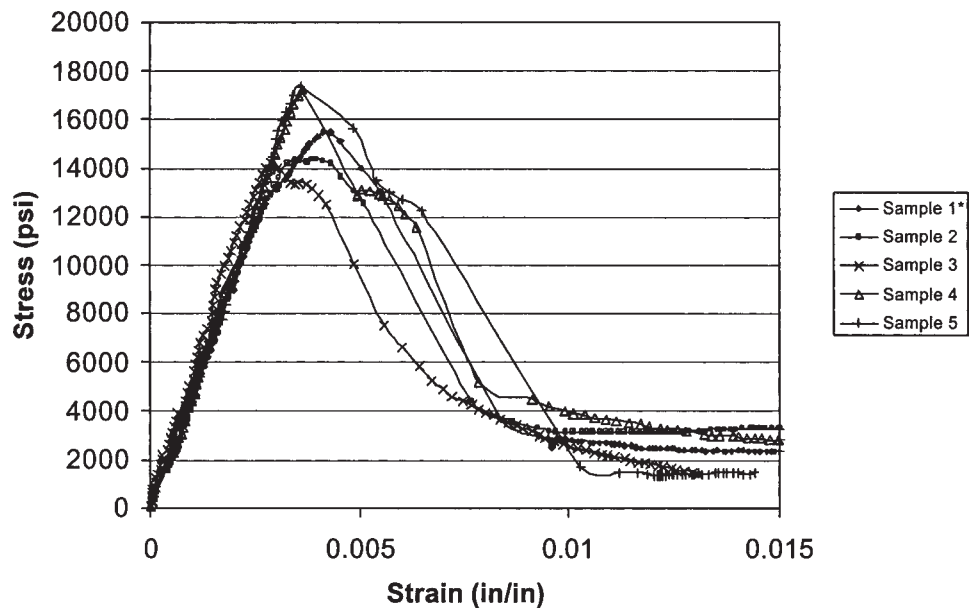


FIGURE 4: Representative stress-strain response of 1.5% fiber volume fraction HS-LWA FRC used in this investigation.

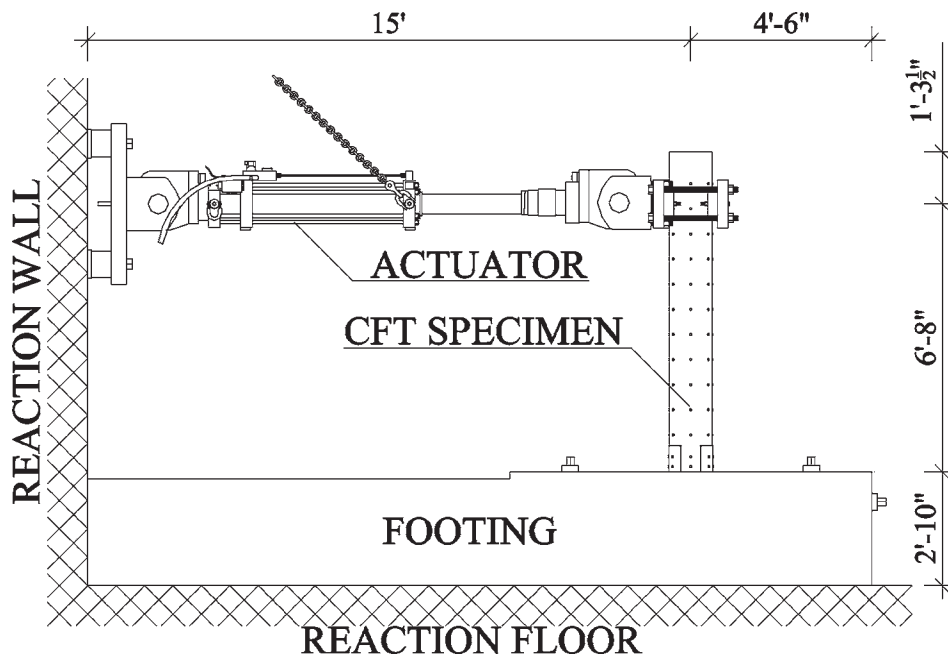


FIGURE 5: Test layout.

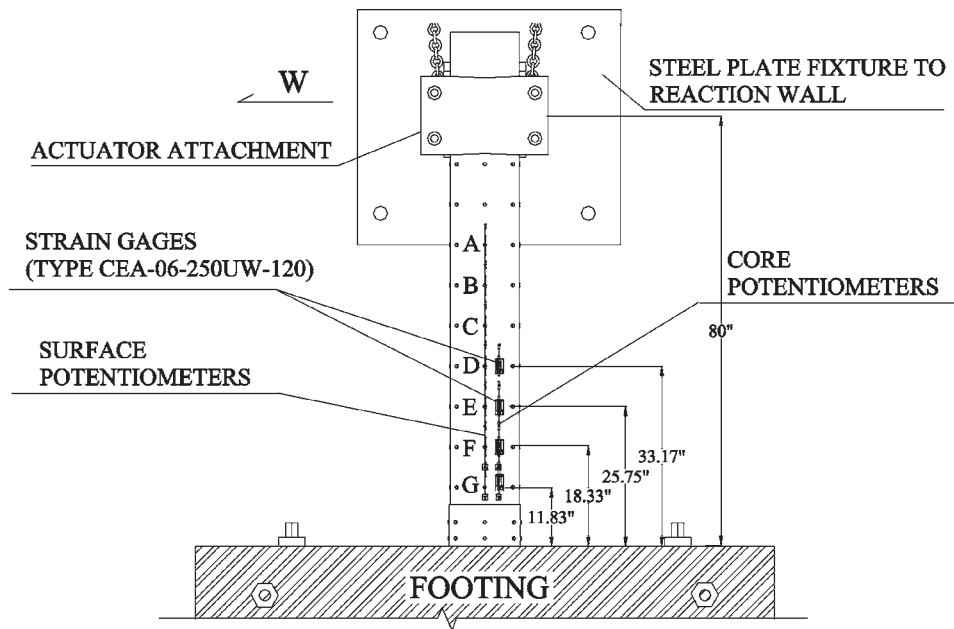


FIGURE 6: Layout of vertical strain gages and potentiometers.

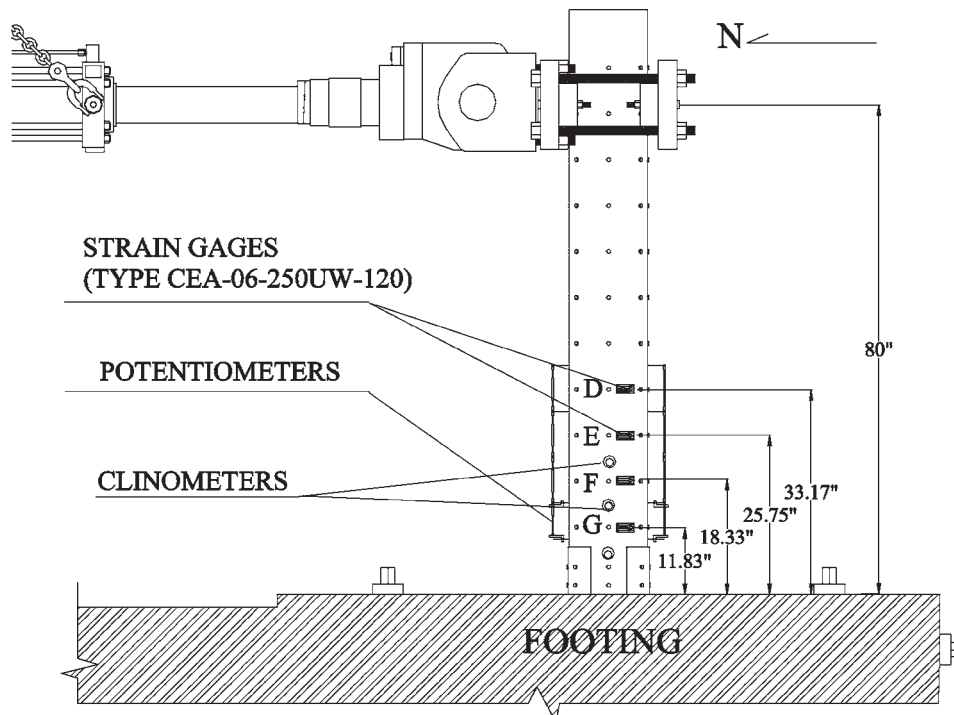


FIGURE 7: Layout of horizontal strain gages, clinometers and core potentiometers.

Testing Displacement History

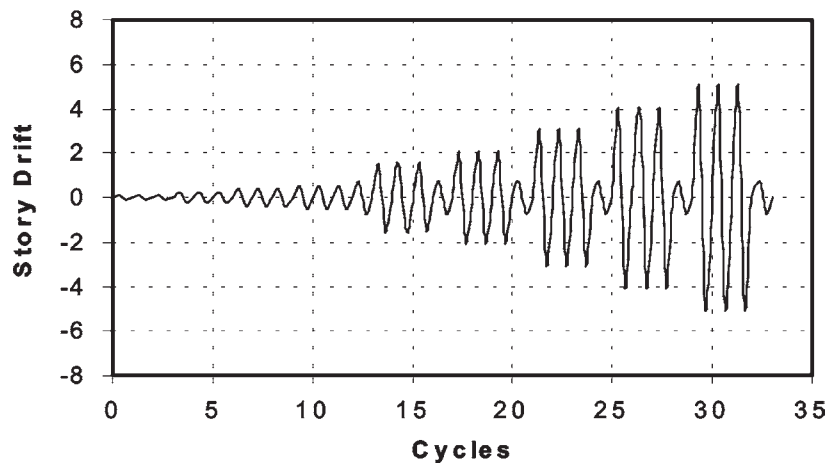


FIGURE 8: Testing displacement history.

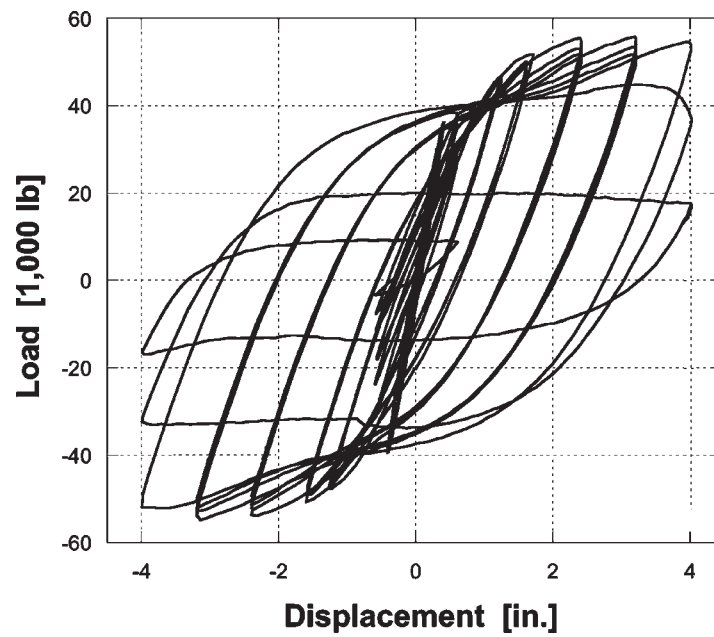


FIGURE 9: Load-displacement response.

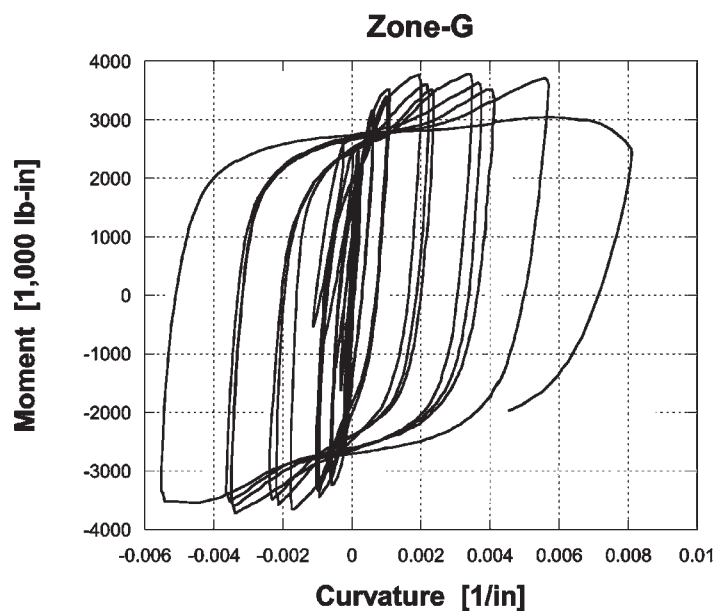
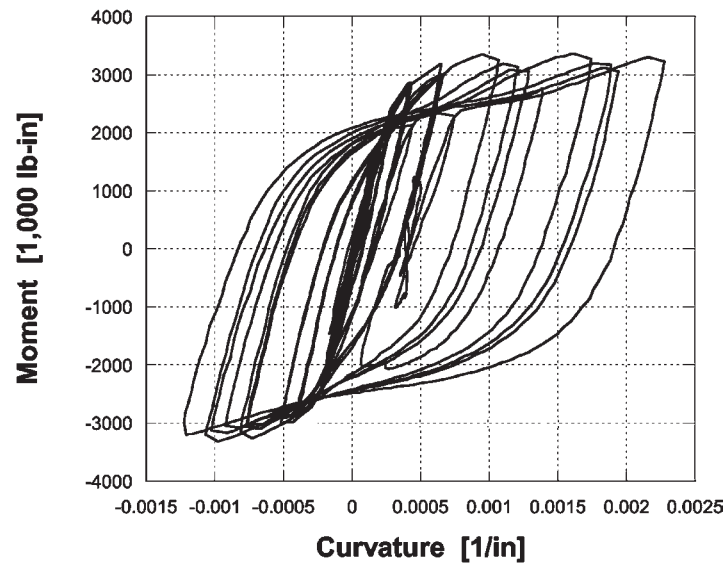


FIGURE 10: Moment-curvature response of zones F and G.

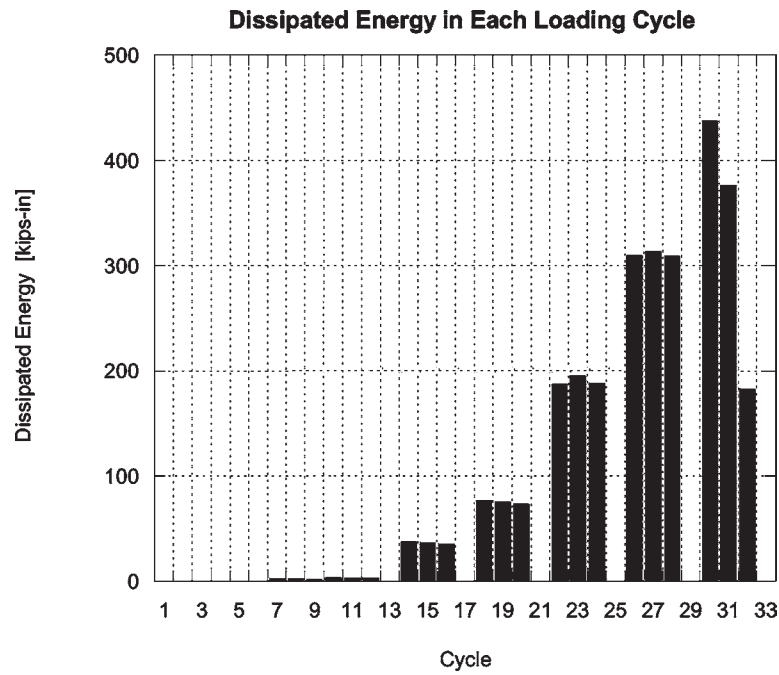


FIGURE 11: Energy dissipated in each load-displacement cycle.

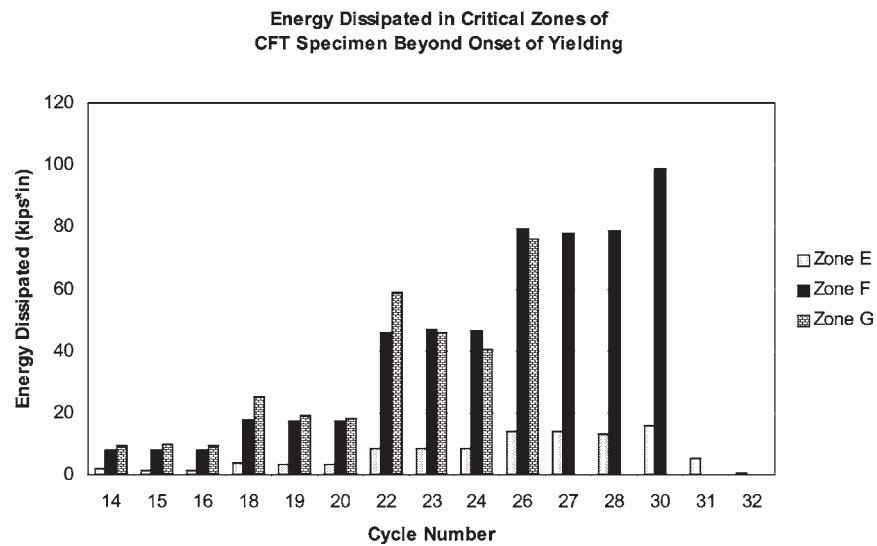


FIGURE 12: Energy dissipated in zones E, F and G.

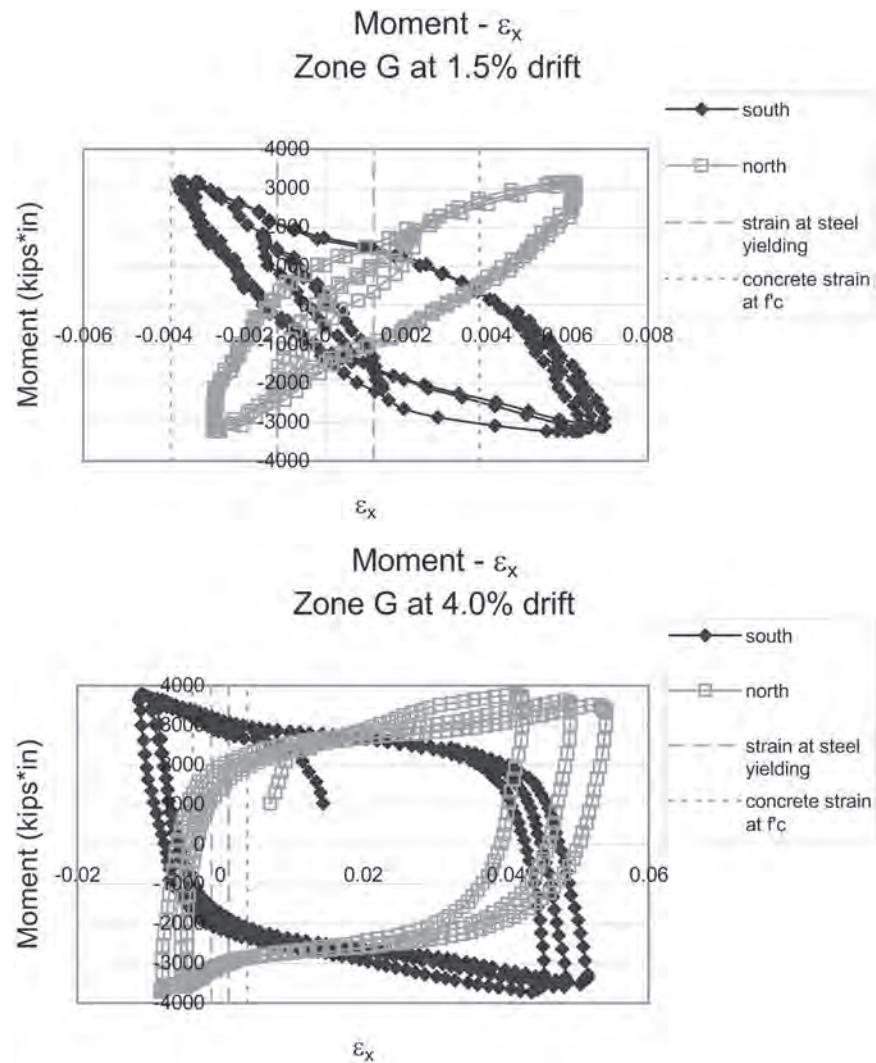


FIGURE 13: Moment vs. longitudinal strain response of zone G. Strain levels at both steel yielding and compressive cylinder strength of HS-LWA FRC (f_c') are marked on graphs.

Fiber Reinforced High Strength Concrete: Residual Strength and Pore Structure Subjected to High Temperatures

by X. Luo, W. Sun, Y. M. Zhang, and S. Y. N. Chan

Synopsis: The mechanical properties and pore structure of fiber reinforced high strength concrete subjected to different high temperatures and cooling regimes were investigated. The results showed that the residual strength of high strength concrete with or without fibers worsened after exposure to high temperatures. Evident drop in compressive and splitting tensile strengths took place between 400°C and 600°C. However, the decline in flexural strength mainly took place before 400°C. Among the specimens, the concrete with 10% of silica fume, 25% of fly ash and 1% of steel fiber provided the highest residual strength and relative residual strength. Steel fibers played a significant role in preventing the worsening of the mechanical properties, particularly at the temperature from 400°C to 600°C. Thermal stress was not the key factor that caused the spalling or explosive spalling in high strength concrete under high temperatures. The re-curing brought recovery of the residual strength in the concretes damaged at high temperatures due to the remarkable improvement of the pore structure.

Keywords: fiber; high strength concrete; high temperature; pore structure; residual strength

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INTRODUCTION

Concrete is one of the most common structural material used globally. The stress distribution, deformation and the development of micro-cracks in concrete are very complex and diverse. This is due to the differences in the composition and mechanical characteristics of raw materials, variation in water content and age of concrete, and the complex physical and chemical actions of fire. In order to evaluate the damage and the residual loading capacity of reinforced concrete structure damaged by fire, it is necessary to determine the mechanical properties of concrete after exposure to fire and to understand the fundamental cause-effect relationships. Furthermore, developing on understanding of the properties of concrete at high temperature is critical and necessary for designing fire resistant concrete structure. Therefore, it is essential to study fire resistance of high strength concrete.

The mechanical behavior of high strength and ordinary concrete after fire differs markedly, and many inconsistent research results have been reported in the past [1-5]. The surface of high strength concrete is prone to spall explosively under fire, and the integrity of the concrete structure tends to deteriorate more easily than ordinary concrete. In some cases, if the temperature was elevated rapidly, an explosive destruction of the structure could take place [6]. Although it is still not fully understood why explosive spalling happens, two mechanisms have been proposed: one is the thermal stress mechanism [7], and the other is pore pressure mechanism [8, 9]. The spalling behavior of high strength concrete depends on many factors, such as the rate of temperature, the mineral composition of aggregates, thermal stress distribution, density and moisture content of