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Effectiveness of Steel Fiber as Minimum Shear Reinforcement

by Jimmy Susetyo, Paul Gauvreau, and Frank J. Vecchio

Ten 35 x 35 x 2.75 in. (890 x 890 x 70 mm) concrete panels were tested under in-plane pure-shear monotonic loading conditions to evaluate the effectiveness of steel fibers in meeting minimum shear reinforcement requirements for concrete elements. The test results indicate that concrete elements exhibiting ductile behavior, sufficient shear strength, and good crack control characteristics can be obtained with an adequate addition of steel fibers, meeting or exceeding the level of performance achievable using code-prescribed minimum amounts of conventional shear reinforcement. Fiber aspect ratio, fiber length, fiber tensile strength, fiber volume content, and concrete compressive strength are found to influence the shear performance of fiber-reinforced concrete (FRC) to varying extents. Details and results are provided.

Keywords: crack control; ductility; panel tests; reinforcement; shear; steel fiber; strength.

INTRODUCTION

The addition of steel fibers to concrete has been found to significantly increase the post-cracking toughness and ductility of the concrete, even when the amount of fiber added is low.[1-3] Increases in tensile strength have also been observed,[4] although a high volume percentage of fibers are required to obtain a substantial increase.[5] Moreover, reductions in crack width and spacing have been reported[6-9]; fibers were found to control crack development and prevent the occurrence of large crack widths. This enhanced post-cracking tensile behavior and improved crack control translates to potentially significant increases in the shear strength and ductility of the concrete. The use of steel fibers therefore holds potential for reducing or eliminating conventional stirrups, particularly in high-strength, high-performance concrete elements, which in turn can lessen the congestion of reinforcement and produce more efficient designs.

Numerous studies have been conducted on the shear strength of steel fiber-reinforced concrete (SFRC) beams. Research conducted on rectangular fiber-reinforced concrete (FRC) beams without stirrups performed by Adebar et al.[10] shows that the addition of fibers significantly increases the shear strength and ductility of the beams. Increases in shear strength of 67% and 90% were achieved with the addition of 0.4% and 0.6%, respectively, of hooked-end steel fibers with lengths of 30 mm (1.2 in.) and diameters of 0.5 mm (0.02 in.). The results also illustrate the importance of the fiber aspect ratio in improving the shear resistance of the beams, with high aspect ratio fibers performing better than low aspect ratio fibers for the same fiber content. Tan et al.[11] tested six simply supported I-beams containing no stirrups under two-point loading. Their test results indicate that an increase in shear strength of up to 73% can be achieved with the addition of hooked-end steel fibers, with the degree of strength improvement dependent on fiber content. The addition of steel fibers also resulted in a significant increase in the shear strains sustainable in the web. Ashour et al.[12] studied the effect of fiber addition on the shear resistance of high-strength concrete beams containing no stirrups. Their results show that the addition of fibers can transform the typical brittle shear failure mode into one of a more ductile nature, particularly for beams with large shear span-depth ratios. The addition of steel fibers was also found to increase the stiffness of the beams, thus reducing beam deflection. Finally, their results indicate that the shear strength of the beams increases with an increase in fiber content, and with a decrease in shear span-depth ratio. Minelli and Vecchio[13] reported the results of tests on large-scale shear-critical concrete beams containing 0.64% of hooked-end steel fibers. Their results reaffirm the understanding that the addition of steel fibers substantially improves the shear behavior of the beams. Due to the bridging effects provided by the fibers, less brittle shear failures were achieved. Moreover, the use of short fibers with a high aspect ratio (fiber length = 30 mm [1.2 in.], fiber aspect ratio = 80) transformed the mode of failure from shear to a ductile flexural mode. Tests on 12 reinforced concrete beams performed by Kwak et al.[14] showed that the addition of steel fibers can reduce crack width and spacing, increase the deformation capacity of beams, and change the mode of failure from brittle to more ductile. Increases in ultimate shear strengths were also observed, particularly in beams with low shear span-depth ratios. Finally, a database of 147 SFRC beams compiled by Parra-Montesinos[15] suggests that the use of steel fiber with a volume content higher than 0.5% is required to achieve failure shear stresses greater than $2\sqrt{f_c'}$ psi ($0.17\sqrt{f_c'}$ MPa).

Although these studies are significant in that they demonstrate the effectiveness of steel fibers as a means of improving shear behavior, they stop short of defining general constitutive models that characterize the contribution of the steel fibers to shear resistance. To provide an experimental basis for developing such models, a series of panel tests was conducted at the structural laboratories of the University of Toronto using the panel element tester. The use of panel tests enables a more thorough investigation of FRC behavior under shear-dominant conditions because they enable constant and uniform shear stress conditions to be imposed over a large specimen area without the presence of the obscuring effects of flexure. Interactions between conventional steel reinforcement and FRC can thus be assessed, and various aspects of concrete behavior such as

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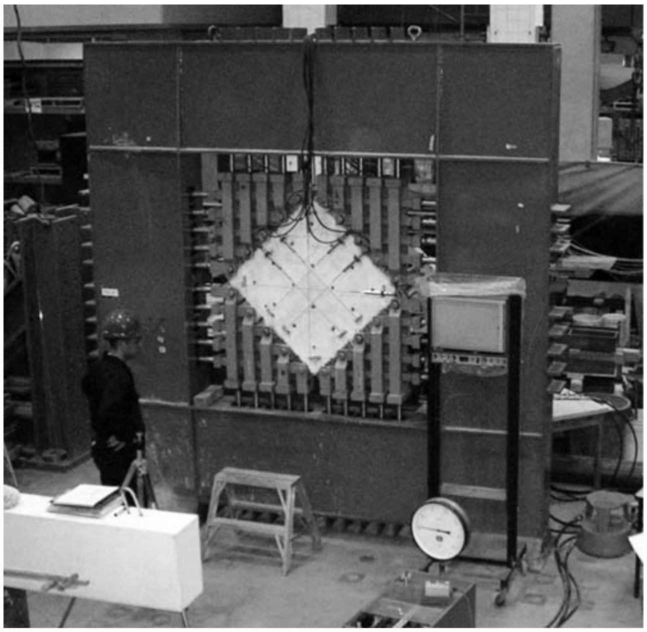


Fig. 1—Panel Element Tester.

tension stiffening, tension softening, and compression softening can be evaluated. In addition, factors that influence the effect of steel fibers on concrete shear resistance can be assessed. These factors include fiber volume content, fiber aspect ratio, fiber length, fiber tensile strength, and concrete compressive strength.

RESEARCH SIGNIFICANCE

The addition of steel fibers results in improved shear strength and ductility, owing to enhanced post-cracking tensile behavior and improved crack control characteristics. To be able to design FRC structures having comparable shear resistance to conventionally reinforced concrete structures, however, additional research is required to develop constitutive models that better characterize the behavior of FRC. The panel tests performed in this experimental program permit a more comprehensive understanding of the behavior of FRC elements subjected to pure shear loading, and their results facilitate further development of constitutive models for FRC.

EXPERIMENTAL PROGRAM

Ten 35 x 35 x 2.75 in. (890 x 890 x 70 mm) panels were tested under in-plane pure-shear monotonic loading condition using the Panel Element Tester facility shown in Fig. 1. Two panels served as control specimens, and as such were orthogonally reinforced with 40-D8 deformed wires in the longitudinal direction (ρ_x = 3.31%) and 10-D4 deformed wires in the transverse direction (ρ_y = 0.42%) (refer to Fig. 2(a)). The cross-sectional area of one D8 wire is 51.61 mm^2 (0.08 $in.^2$), and the area of one D4 wire is 25.81 mm^2 (0.04 $in.^2$). The 0.42% reinforcement ratio in the transverse direction is larger than the prescribed minimum shear reinforcement of many design standards. For example, CSA A23.3-04[16] prescribes a minimum ratio of only 0.13% for a 35 x 35 x 2.75 in. (890 x 890 x 70 mm) panel with a concrete compressive strength of 13.0 ksi (90 MPa). However, a symmetric reinforcement arrangement about the axis of the shear keys, which serve to transfer the load from the machine hydraulic jacks to the concrete panel, was required to prevent the introduction of moments at the shear keys, necessitating the higher transverse reinforcement ratios.

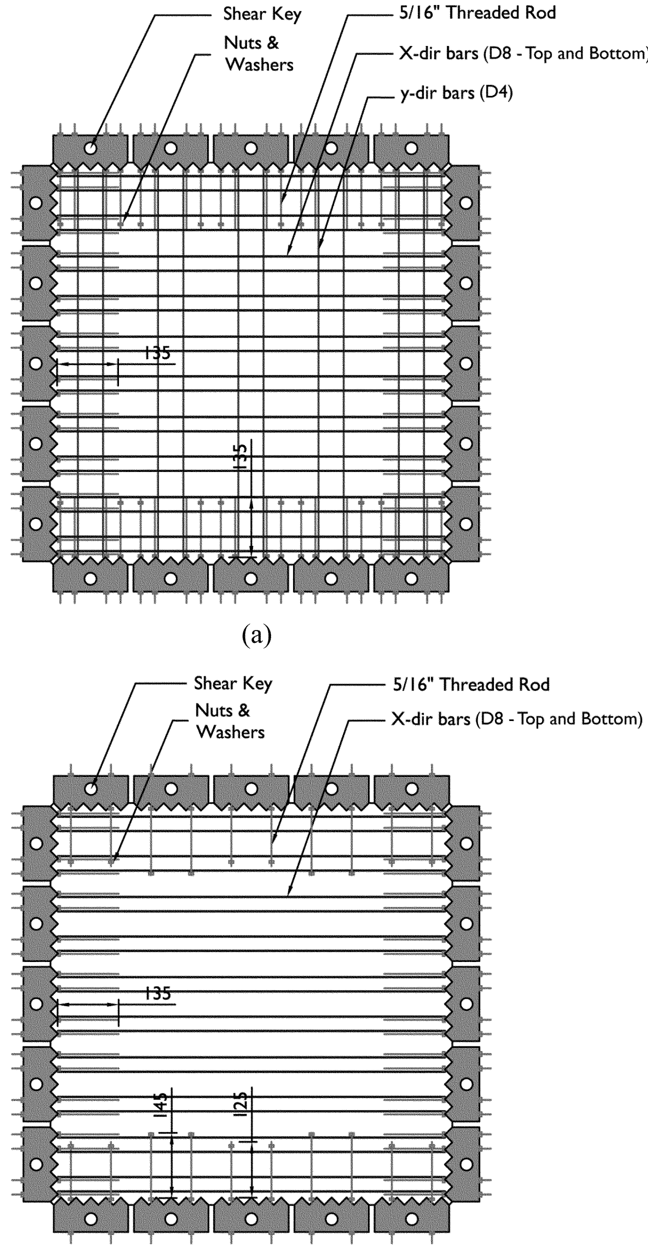


Fig. 2—Details of test specimens: (a) control panels; and (b) FRC panels. (Note: 1 in. = 25.4 mm.)

The remaining eight panels, containing steel fibers, were reinforced in the longitudinal direction only with 40-D8

Table 1—Test matrix

ID	Specified f'_c, ksi (MPa)	Fiber content, %	Fiber type
C1C	7.25 (50)	—	—
C1F1V1	7.25 (50)	0.5	RC80/50-BN
C1F1V2	7.25 (50)	1.0	RC80/50-BN
C1F1V3	7.25 (50)	1.5	RC80/50-BN
C1F2V3	7.25 (50)	1.5	RC80/30-BP
C1F3V3	7.25 (50)	1.5	RC65/35-BN
C2C	11.60 (80)	—	—
C2F1V3	11.60 (80)	1.5	RC80/50-BN
C2F2V3	11.60 (80)	1.5	RC80/30-BP
C2F3V3	11.60 (80)	1.5	RC65/35-BN

Table 2—Properties of conventional reinforcement

Reinforcing bar type	d_b, in. (mm)	A_s, in.^{2} (mm^2)	E_s, ksi (GPa)	f_{ys}, ksi (MPa)	ε_{ys}, mε	f_{us}, ksi (MPa)	ε_{us}, mε
D4	0.225 (5.72)	0.04 (25.81)	27,150 (187)	64.8 (447)	2.41	79.6 (549)	57.6
D8	0.319 (8.10)	0.08 (51.61)	32,590 (225)	80.1 (552)	2.58	93.9 (647)	45.4

Table 3—Properties of fibers[25]

Fiber type	l_f, in. (mm)	d_f, in. (mm)	l_f/d_f	f_{uf}, ksi (MPa)
RC80/50-BN	1.97 (50)	0.024 (0.62)	81	152 (1050)
RC80/30-BP	1.18 (30)	0.015 (0.38)	79	334 (2300)
RC65/35-BN	1.38 (35)	0.022 (0.55)	64	160 (1100)

deformed bars (ρ_x = 3.31%); refer to Fig. 2(b). Three types of hooked-end steel fibers (RC80/50-BN, RC80/30-BP, and RC65/35-BN) were used to investigate the influence of fiber aspect ratio, fiber length, and fiber tensile strength on the shear performance of FRC. In addition, three different fiber volume contents (0.5, 1.0, and 1.5%) and two different concrete nominal compressive strengths (7.3 and 11.6 ksi [50 and 80 MPa]) were investigated. The test matrix is summarized in Table 1.

Materials

The properties of the reinforcing steel and steel fibers used in the test panels are listed in Tables 2 and 3, respectively. The deformed wires, due to cold-forming, did not exhibit a yield plateau. Their yield strength and yield strain were thus determined from the proportionality limit.

The addition of fibers is known to reduce the workability of concrete. Adjustments were therefore made to the composition of the concrete mixture such that the requirements for compressive strength and workability could be satisfied without segregation. Naaman et al.[17] discovered that the addition of fibers required a reduction in the ratio of the coarse to fine aggregate. This results in a higher paste volume required to fill the void between the aggregate particles and the fibers, and to coat the fibers fully. These requirements were reflected in the higher fine aggregate and binder contents in the fiber-reinforced mixtures in comparison to the plain concrete mixtures. In addition, greater amounts of water reducer and high-range water-reducing admixture were added to the fiber-reinforced mixtures to ensure sufficient concrete workability. With the high-strength fiber-reinforced mixture, the combination of a low water-binder ratio (w/b) and the addition of fibers resulted in a nonworkable concrete, particularly for high volumes of fiber addition. This necessitated the use of self-consolidating concrete to ensure proper compaction. The dry compositions of the concrete mixtures can be found in Susetyo.[18]

Casting procedure

During the casting of the panels, the concrete was consolidated using external vibrators attached to the bottom of the formwork. Once the concrete placement was completed, the surface was finished. After the concrete had set, the specimens were moist-cured for 7 days using plastic-covered wet burlap.

Instrumentation

The instrumentation of the panels consisted of linear variable displacement transducers (LVDTs) mounted on the surface of the concrete, displacement transducers (Zurich gauges) used in conjunction with fixed targets mounted on the surface of the concrete, and electrical resistance strain gauges mounted on the surface of the reinforcing steel. The LVDTs were used to monitor the overall deformation of the panels. Six LVDTs were mounted on each side of a test panel: two in the longitudinal direction, two in the transverse direction, and one in each diagonal direction. The Zurich gauges were used to obtain more detailed definitions of local deformations. Sixteen Zurich targets were affixed to each side of a test panel, forming nine 7.87 x 7.87 in. (200 x 200 mm) subgrids; within each subgrid, longitudinal, transverse, and diagonal displacements were measured at each load stage. Electrical resistance strain gauges with a gauge length of 0.2 in. (5.0 mm) were attached to the reinforcing steel to provide continuous measurement of local strains in the reinforcement. Pressure transducers on the tester's hydraulic control system and load cells placed on several load actuators provided a continuous monitoring of the loads applied to the specimens.

Test procedure

The tests were conducted by loading the panels under monotonically increasing in-plane pure shear load. Prior to loading, initial Zurich gauge readings were taken to determine the undeformed state of a test panel. Load was then applied until the first crack was detected, at which point readings corresponding to the first load stage were taken. Loading then continued in stages until the panel reached its maximum load-carrying capacity, entered post-peak response, and eventually failed. Between the first cracking and failure, typically 10 to 15 load stages were employed. At each load stage, Zurich gauge readings were taken, crack patterns were marked and photographed, crack widths were measured, and surface conditions were carefully surveyed.

EXPERIMENTAL RESULTS

Key results, obtained from the panel tests and calculated using Disturbed Stress Field Model,[19] are summarized in Table 4. As indicated by these results, good shear behavior was obtained with sufficient addition of steel fibers, even without the presence of transverse reinforcement. In some cases, the shear behavior of the FRC panels surpassed that of the conventionally reinforced concrete panels. Discussion of test observations regarding maximum shear resistance, ductility, failure mode, crack widths, and crack spacing follows.

Maximum shear resistance

All FRC panels, except for Panel C1F1V1, managed to withstand at least 87% of the maximum shear stresses

Table 4—Summary of panel tests

ID	f'_c, ksi (MPa)	v_{cr}, ksi (MPa)	γ_{cr}, mε	v_u, ksi (MPa)	γ_u, mε	w_m, in. (mm)	s_m, in. (mm)	$f_{c1,cr}$, ksi (MPa)	$f_{c1,max}$, ksi (MPa)	$f_{c1,u}$, ksi (MPa)	$f_{c2,u}$, ksi (MPa)	f_{sx}, ksi (MPa)	f_{sy}, ksi (MPa)	Failure mode
C1C	9.53 (65.7)	0.29 (2.01)	0.086	0.837 (5.77)	6.01	0.55	57.2	0.297 (2.05)	0.416 (2.87)	0.207 (1.43)	–1.697 (–11.70)	36.3 (250)	72.7 (501)	y-bar yield
C1F1V1	7.45 (51.4)	0.303 (2.09)	0.197	0.512 (3.53)	2.77	0.55	114.4	0.321 (2.21)	0.410 (2.83)	0.268 (1.85)	–0.976 (–6.73)	21.5 (148)	—	Shear slip
C1F1V2	7.75 (53.4)	0.384 (2.65)	0.139	0.750 (5.17)	5.27	0.45	54.7	0.376 (2.59)	0.441 (3.04)	0.409 (2.82)	–1.372 (–9.46)	29.2 (201)	—	Shear Slip
C1F1V3	7.21 (49.7)	0.265 (1.83)	0.055	0.779 (5.37)	5.10	0.45	57.2	0.268 (1.85)	0.454 (3.13)	0.431 (2.97)	–1.407 (–9.70)	29.6 (204)	—	Shear slip
C1F2V3	8.66 (59.7)	0.268 (1.85)	0.070	0.969 (6.68)	6.20	0.45	38.1	0.268 (1.85)	0.564 (3.89)	0.535 (3.69)	–1.758 (–12.12)	37.1 (256)	—	Shear slip
C1F3V3	6.60 (45.5)	0.325 (2.24)	0.118	0.811 (5.59)	4.27	0.50	57.2	0.323 (2.23)	0.558 (3.85)	0.447 (3.08)	–1.468 (–10.12)	30.9 (213)	—	Shear slip
C2C	13.13 (90.5)	0.373 (2.57)	0.099	0.928 (6.40)	7.00	0.50	66.2	0.367 (2.53)	0.370 (2.55)	0.178 (1.23)	–2.107 (–14.53)	49.5 (341)	74.3 (512)	y-bar yield
C2F1V3	11.46 (79.0)	0.315 (2.17)	0.131	1.001 (6.90)	5.25	0.70	36.0	0.305 (2.10)	0.531 (3.66)	0.499 (3.44)	–2.006 (–13.83)	45.7 (315)	—	Shear slip
C2F2V3	11.10 (76.5)	0.231 (1.59)	0.110	0.915 (6.31)	4.35	0.65	46.6	0.228 (1.57)	0.544 (3.75)	0.525 (3.62)	–1.594 (–10.99)	32.5 (224)	—	Shear slip
C2F3V3	8.99 (62.0)	0.305 (2.10)	0.222	0.808 (5.57)	4.97	0.60	40.6	0.302 (2.08)	0.425 (2.93)	0.419 (2.89)	–1.559 (–10.75)	34.5 (238)	—	Shear slip

Table 5—Shear resistance of concrete panels containing minimum shear reinforcement as predicted by ACI 318M-08[20] and CSA A23.3-04[16]

ID	f'_c, ksi (MPa)	$v_{u\text{-}exp}$, ksi (MPa)	ACI318M-08			CSA A23.3-04		
			$A_v/(b \times s)^*$, %	v_r, ksi (MPa)	$\frac{v_{u-exp}}{v_r}$	$A_v/(b \times s)^*$, %	v_r, ksi (MPa)	$\frac{v_{u-exp}}{v_r}$
C1C	9.53 (65.7)	0.837 (5.77)	0.120†	0.406 (2.80)	2.06	0.109†	0.416 (2.87)	2.01
C1F1V1	7.45 (51.4)	0.512 (3.53)	0.106	0.360 (2.48)	1.42	0.096	0.379 (2.61)	1.35
C1F1V2	7.75 (53.4)	0.750 (5.17)	0.108	0.367 (2.53)	2.05	0.098	0.384 (2.65)	1.95
C1F1V3	7.21 (49.7)	0.779 (5.37)	0.104	0.354 (2.44)	2.20	0.095	0.373 (2.57)	2.09
C1F2V3	8.66 (59.7)	0.969 (6.68)	0.114	0.387 (2.67)	2.50	0.104	0.403 (2.78)	2.41
C1F3V3	6.60 (45.5)	0.811 (5.59)	0.100	0.338 (2.33)	2.40	0.091	0.358 (2.47)	2.26
C2C	13.13 (90.5)	0.928 (6.40)	0.123†	0.416 (2.87)	2.23	0.128†	0.431 (2.97)	2.15
C2F1V3	11.46 (79.0)	1.001 (6.90)	0.123	0.416 (2.87)	2.40	0.119	0.424 (2.92)	2.36
C2F2V3	11.10 (76.5)	0.915 (6.31)	0.123	0.416 (2.87)	2.20	0.117	0.422 (2.91)	2.17
C2F3V3	8.99 (62.0)	0.808 (5.57)	0.116	0.395 (2.72)	2.05	0.106	0.409 (2.82)	1.98

*Code-prescribed minimum shear reinforcement ratio.
†Actual reinforcement ratio = 0.42%.

sustained by the control panels (refer to Table 4). Panel C1F2V3, with 1.5% RC80/30-BP fibers, surpassed the strength of the control panels by a factor of 1.16. Panel C1F1V1, having a low 0.5% fiber volume content, was able to sustain only 61% of the ultimate shear resistance of Panel C1C, its respective control panel. It is evident that this low fiber content was not sufficient to guarantee equivalent shear resistance. Nevertheless, good shear behavior of the concrete panels without transverse reinforcement was achievable with a moderate-to-high fiber addition, especially given that the control panels contained approximately three to four times the code-prescribed minimum shear reinforcement amounts.

Table 5 compares the nominal shear resistances calculated using ACI 318M-08[20] and CSA A23.3-04[16] for concrete panels containing minimum shear reinforcement to the resistances observed experimentally. For these calculations, the capacity reduction factor[20] and the material resistance factors[16] were set to 1.0. Longitudinal reinforcement and concrete compressive strength were taken as equal to those of the test panels. The results summarized in Table 5 again suggest that steel fibers are a viable means of replacing minimum-to-low amounts of shear reinforcement. Even with a low fiber volume content of 0.5%, the shear resistance of the fiber-reinforced Panel C1F1V1was 42% greater than the calculated shear resistance for minimum shear reinforcement in accordance with ACI 318M-08[20] and 35% greater than the calculated shear resistance calculated for minimum shear reinforcement in accordance with CSA A23.3-04.[16] Increasing the fiber volume content to 1.0% and 1.5% resulted in shear resistances that were at least double the values calculated using ACI 318M-08[20] and CSA A23.3-04.[16]

Ductility

The shear stress-shear strain responses of the test panels are compared in Fig. 3. All fiber-reinforced panels, except Panel C1F1V1, performed reasonably well in terms of the maximum shear strain attained at failure. With the exception of Panel C1F1V1, all fiber-reinforced panels were able to undergo a maximum shear deformation of at least 62% of that of the control panels. Although not exceptional, this was

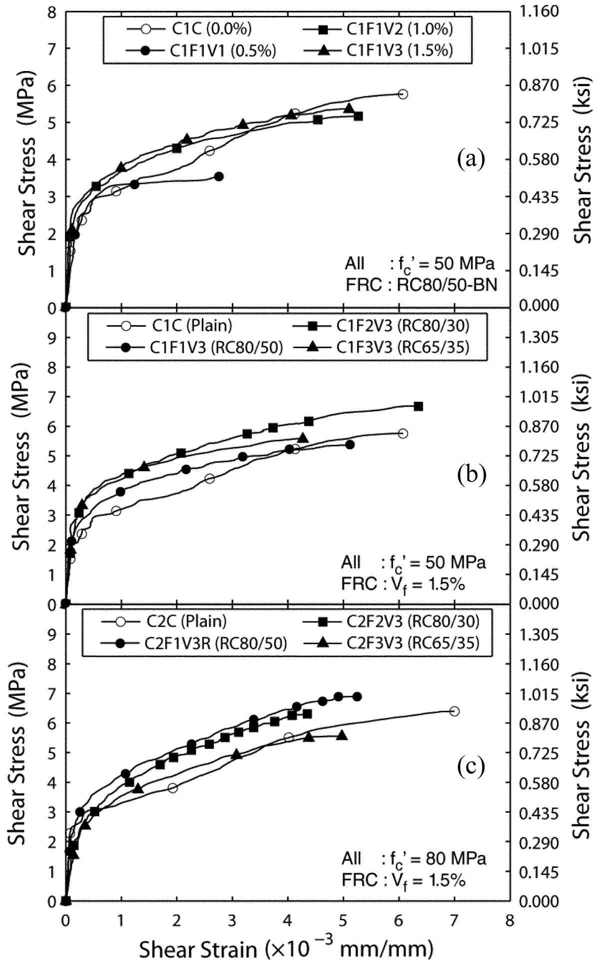


Fig. 3—Shear stress-shear strain response of test panels.

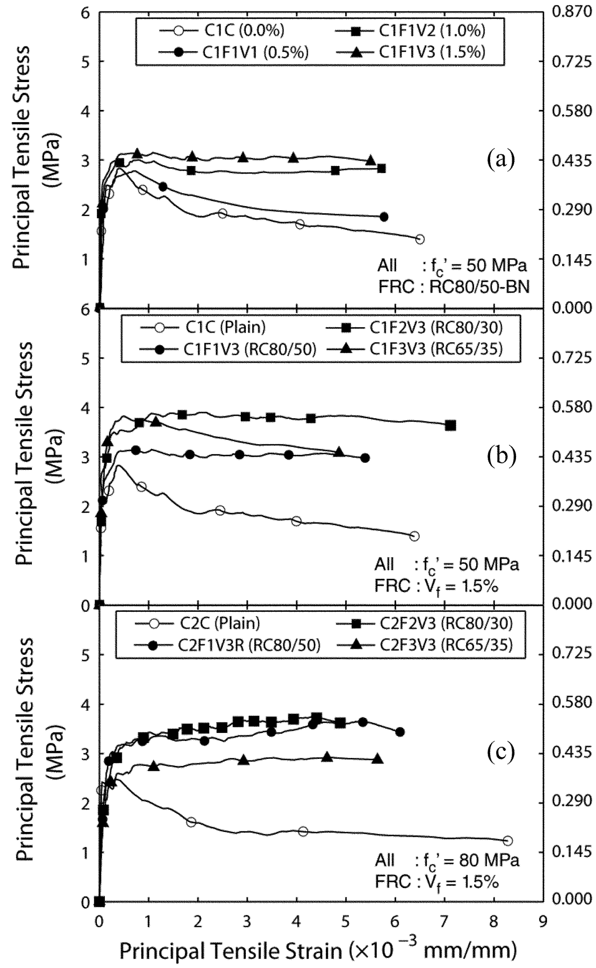


Fig. 4—Principal tensile stress-strain response of concrete in test panels.

remarkably good performance considering the absence of transverse reinforcement in the panels. Once again, Panel C1F2V3 was able to match Panel C1C in terms of maximum shear deformation. In contrast, Panel C1F1V1, with its low fiber volume content of 0.5%, was only able to deform to 46% of the maximum shear deformation exhibited by the control panel. This further suggests that to ensure good shear behavior in the absence of transverse reinforcement, a fiber volume addition of greater than 0.5% is required.

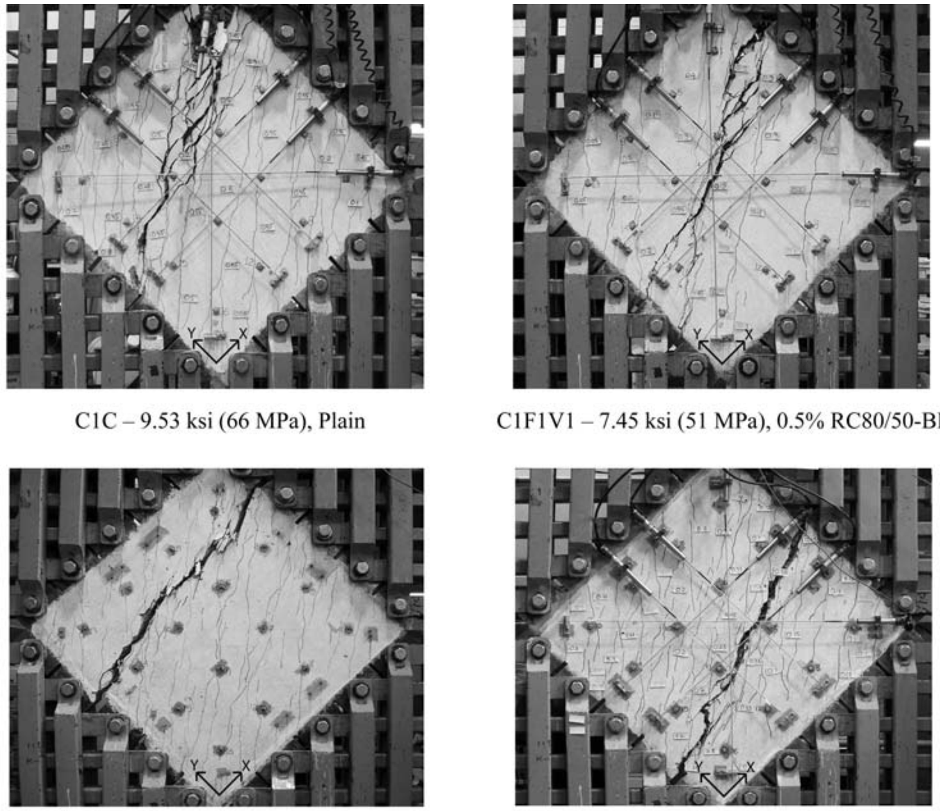
Post-cracking tension response

The post-cracking principal tensile stress-strain responses of the panels are plotted in Fig. 4. It is evident that the addition of fibers significantly improved the post-cracking tensile behavior of the panels. The control panels exhibited strain-softening behavior due to gradual deterioration of bond between the concrete and the reinforcement, reducing the concrete's ability to transmit tensile stresses across the cracks. Conversely, all the fiber-reinforced panels, except Panel C1F1V1, exhibited strain-hardening behavior as the fibers enabled significant transmission of tensile stresses across the cracks, even under high strain conditions. Due to its low fiber volume content, Panel C1F1V1 exhibited strain-softening behavior soon after cracking, similar to the behavior exhibited by the control panel. Despite the lack of the transverse reinforcement, Panel C1F1V1 demonstrated an improved principal tensile stress-strain response relative to that of the conventionally reinforced panel.

Failure modes

All fiber-reinforced panels failed in a mode governed by crack interface shear due to the eventual inability of the concrete to transmit load across the cracks, regardless of the type of fibers used, the amount of fiber addition, or the strength of the concrete. Representative failure conditions are shown in Fig. 5 and 6. Concrete stress remained less than 25% of cylinder compressive strength. Stress in longitudinal reinforcing steel remained less than 60% of yield strength. This suggests that neither crushing of concrete nor yielding of reinforcement was the governing failure mechanism for the fiber-reinforced panels.

In contrast, the failures of the control panels were initiated by the yielding of the transverse reinforcement, followed by a loss of aggregate interlock or rupture of the transverse reinforcement. As the reinforcement yielded, the ability to transmit load through cracks quickly diminished due to crack slip. Because of its higher compressive strength, Panel C2C had a higher resistance to cracking than Panel C1C. This enabled the panel to sustain a higher load and led to rupture of transverse reinforcement before loss of aggregate interlock occurred. In Panel C1C, loss of aggregate interlock occurred before rupture of transverse reinforcement. Nevertheless, similar to the fiber-reinforced panels, stresses in the concrete



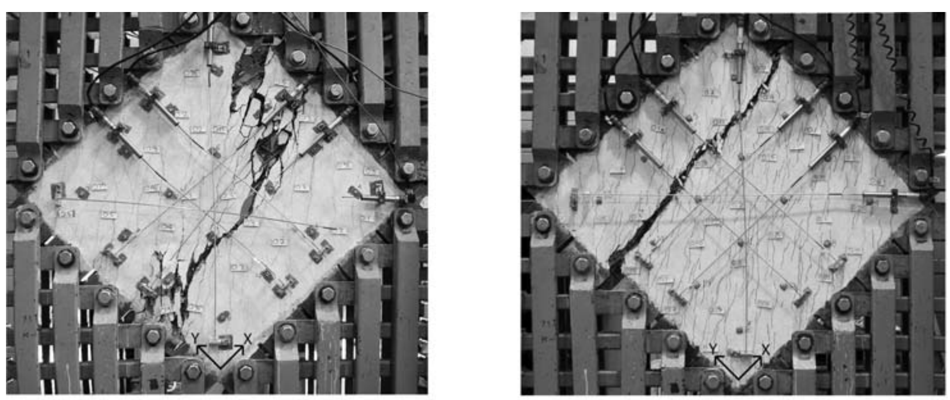
C1C – 9.53 ksi (66 MPa), Plain

C1F1V1 – 7.45 ksi (51 MPa), 0.5% RC80/50-BP

C1F1V2 – 7.75 ksi (53 MPa), 1.0% RC80/50-BP

C1F1V3 – 7.21 ksi (50 MPa), 1.5% RC80/50-BP

Fig. 5—Representative failure modes observed in test panels: normal-strength concrete.



C2C – 13.13 ksi (91 MPa), Plain

C2F3V3 – 8.99 ksi (62 MPa), 1.5% RC65/35-BN

Fig. 6—Representative failure modes observed in test panels: high-strength concrete.

and longitudinal reinforcing steel were well below their respective ultimate strengths.

Crack width and spacing

An important property of FRC is its ability to control crack propagation. Crack width control is important in ensuring adequate concrete shear resistance as narrow, closely spaced cracks enable better transmission of shear stress through aggregate interlock than large, widely spaced cracks. In conventionally reinforced concrete, crack control is provided by bonded reinforcing steel. In FRC, the fibers will bridge the cracks, controlling crack propagation and allowing tensile stresses to be transmitted across the cracks. As a result, the behavior of the concrete is significantly improved.

Figures 7 and 8 compare the maximum crack widths and crack spacings observed in the test panels, respectively. It can be observed that when normal-strength concrete was used, the fiber-reinforced panels exhibited smaller crack widths than the control panel under the same shear stress. This again illustrates the ability of fibers to control crack propagation. Due to the smaller crack widths, higher stresses could be transmitted across the cracks and, thus, the integrity of the cracked panels was maintained. An exception was Panel C1F1V1. Although it initially exhibited smaller crack widths than the control panel, it was incapable of controlling

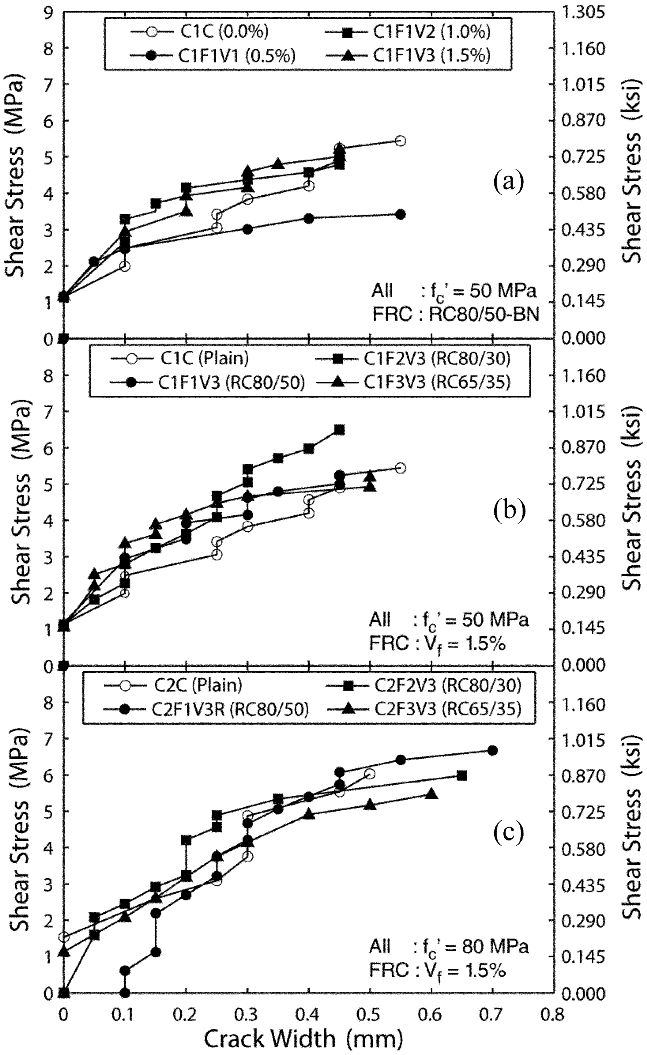


Fig. 7—Crack widths observed in test panels.

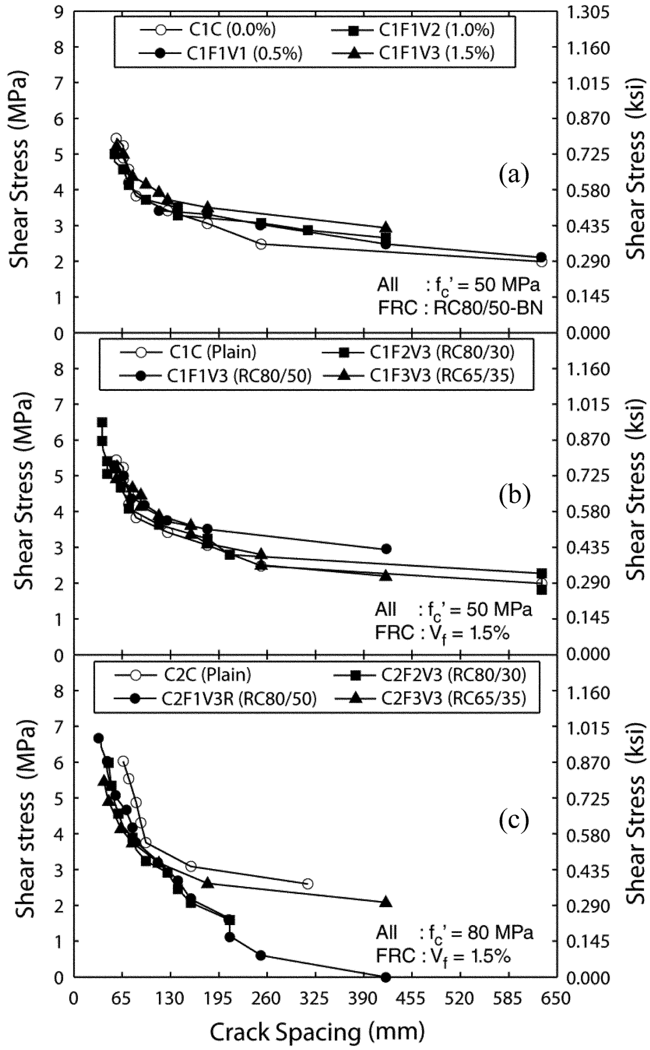


Fig. 8—Crack spacings observed in test panels.

the cracks at later load stages. The large crack widths that ensued limited the magnitude of shear stress that could be transmitted across the cracks, and hence led to the relatively poor performance of Panel C1F1V1.

The influence of fiber volume content on crack control characteristics can be observed in Fig. 7(a). A fiber volume of 0.5% was found to be too low to guarantee good crack control, as illustrated by the wider cracks of Panel C1F1V1 than those of the control panel under the same shear stress. Increasing the fiber content to 1.0% significantly reduced the crack widths by a factor of 3.0 when compared to those measured in Panel C1F1V1. Increasing the fiber content from 1.0% to 1.5% resulted in only a slight additional improvement in crack behavior.

The influence of the fiber type on the crack control characteristics is illustrated in Fig. 7(b) and (c). As indicated in the figures, the panels containing high aspect ratio fibers exhibited smaller crack widths than panels containing low aspect ratio fibers.

The effectiveness of fibers to control crack width was less pronounced in high-strength concrete panels than in normal-strength concrete panels. Although the high-strength, fiber-reinforced panels still exhibited remarkable crack control ability in the absence of transverse reinforcement, the level of improvement over the corresponding conventionally reinforced concrete panel was less than that in the normal-strength fiber-reinforced panels, as indicated in Fig. 7 and 8.

The influence of fiber addition on average crack spacing can also be observed in Fig. 8. The figure indicates that the crack spacing of the fiber-reinforced panels is comparable to or smaller than that of the control panel, even without the presence of the transverse reinforcement. This again suggests that fibers are as effective as conventional reinforcement in controlling cracks when sufficient fiber content is added. Note that Specimen C2F1V3 was prematurely cracked during the mounting of the panel into the machine. It is believed, however, that the existence of a precrack only altered the initial response of the panel. The response of the panel at high shear stresses is believed to have been unaltered.

DISCUSSION

Five parameters are known to affect the effectiveness of steel fibers in improving the behavior of concrete: fiber volume content, fiber aspect ratio, fiber length, fiber tensile strength, and concrete compressive strength. The influences of these parameters on the shear resistance of the FRC panels are examined and discussed in the following.

Effect of fiber content

Figure 3(a) compares the shear stress-shear strain response of panels having similar concrete strengths and fiber type, but different amounts of fibers. Panel C1C was the control panel with zero fiber content, whereas Panels C1F1V1, C1F1V2, and C1F1V3 contained 0.5%, 1.0%, and 1.5% by volume of RC80/50-BN steel fibers, respectively. The responses depicted indicate strong similarities in behavior prior to cracking; however, the similarities diminished as the panels were subjected to higher shear stresses. All fiber-reinforced panels except Panel C1F1V1 exhibited a gradual softening in the response until failure occurred. The gradual softening was a result of the development of multiple cracks and slippage of fibers through the cracks as the load was increased. The control panel, C1C, also exhibited softening in the response, but at a lower stiffness due to the fewer and wider cracks than those in the fiber-reinforced panels. Panel C1F1V1, having only 0.5% of fibers by volume, exhibited a plateau in the response soon after the first crack appeared. The very low fiber content was found to be incapable of controlling crack propagation; however, owing to fiber slippage through the crack, the transmission of tensile stresses across the crack was still enabled, resulting in the flat response. Its response remained flat until the panel failed at a considerably lower shear resistance and ductility level than those of the control panel.

It is evident that an increase in the fiber content led to an improvement in concrete performance. Doubling the fiber volume content from 0.5% to 1.0% resulted in a 46% increase in the shear resistance and a 94% increase in the maximum shear deformation. Further increasing fiber volume from 1.0% to 1.5%, however, resulted in additional improvement of less than 4%. It is suspected that a fiber volume content of 1.5% might exceed the optimum fiber volume content for this particular concrete mixture, as suggested by Rossi.[21]

The examination of the principal tensile stress-principal tensile strain response of the panels further clarified the influence of the fiber content on the behavior of FRC. As indicated in Fig. 4(a), although the response of Panel C1F1V1 indicates improvement over the response of the control panel, it still exhibited a strain-softening behavior immediately after the maximum principal tensile stress was reached. By increasing the fiber volume content to at least 1.0%, substantial improvements in the concrete tensile behavior were observed, as indicated by the plateau in the principal tensile stress-strain responses and by the high residual post-cracking tensile strengths. Again, only a marginal increase in the tensile stresses that could be transmitted across the crack was obtained by increasing the fiber content from 1.0% to 1.5%, leading to the only slight improvement in performance of Panel C1F1V3 over Panel C1F1V2.

The results from these tests do not provide a basis for a rigorous examination of the principal compressive stress-principal compressive strain response because of the relatively low levels of compression stress achieved (refer to Table 4). Generally, however, there was no discernible influence of fiber content on the observed compression response. This supports the hypothesis that fiber addition minimally affects the prepeak compressive behavior of the concrete, particularly at low values of compressive stress and strain.[22,23]

Effects of fiber type

Figures 3(b) and (c) compare the shear responses of normal-strength and high-strength concrete panels having a similar concrete strength and containing 1.5% of fibers by volume, respectively. It is evident from these plots that the length of fibers used has significant influence on the response of the panels. The panels containing short fibers (Panels C1F2V3 and C1F3V3) exhibited a higher shear resistance than the panels containing long fibers (Panel C1F1V3). The principal tensile stress-strain responses plotted in Fig. 4(b) and (c) also suggest similar conclusions. It is presumed that this is due to the actual number of fibers present in the panels. Despite the same fiber content, the number of individual fibers was larger for short fibers than for long fibers due to the shorter length and the smaller diameter, leading to an increased possibility of the fibers intersecting microcracks. The progression of microcracks into macrocracks was better controlled, thus improving the shear performance.

A further examination of the responses plotted in Fig. 3 and 4 reveals the influence of the fiber aspect ratio on panel response. High-aspect-ratio fibers appear to be more effective in improving structural behavior than low-aspect-ratio fibers. The shear stress-strain responses of Panels C1F1V3 and C1F2V3, which contained fibers with aspect ratios of 81 and 79, respectively, showed better post-cracking ductility than Panel C1F3V3, which contained fibers with an aspect ratio of 64. Furthermore, the principal tensile stress-strain responses of Panels C1F1V3 and C1F2V3 indicate a plateau after the maximum principal tensile stresses had been attained, whereas the response of Panel C1F3V3 shows strain-softening behavior. These findings indicate that fiber aspect ratio has a greater effect on the effectiveness of fiber reinforcement than fiber length and agrees well with the conclusions of Johnston and Skarendahl.[24]

The tensile strength of the fibers did not have a significant influence on the response of the fiber-reinforced panels tested in this research program. The tensile stresses that were developed in the fibers were lower than the fiber ultimate tensile strength because no fiber fracture was observed. All activated fibers experienced ductile fiber slippage instead of brittle fiber fracture, which explains the ductile behavior of the fiber-reinforced panels. The tensile strength of the fibers does influence the moment capacity of the fibers, which affects the required force to straighten the hooks and pull the fibers out from the concrete; however, this influence was not evident in the panel responses observed.

Effects of concrete matrix strength

Concrete strength did not have a substantial influence on the shear stress-strain response of the panels, as no significant correlations could be discerned from the observed behaviors. In the case of panels containing the RC80/50-BN fibers, a higher concrete strength resulted in a higher shear resistance; however, in panels containing the RC65/35-BN fibers, the opposite was observed: a higher concrete strength resulted in a lower shear resistance. It appears that the type of fibers used in the panel was more influential than the strength of the concrete matrix.

Another indication that the concrete strength did not significantly influence the behavior of FRC can be deduced from the principal tensile stress-strain responses (Fig. 4). The influence of concrete strength on panel behavior varied with the type of fibers used, but with no clear apparent pattern.

SUMMARY AND CONCLUSIONS

A series of panel tests was performed to assess the effectiveness of steel fibers as a possible replacement for conventional transverse reinforcing steel in concrete elements requiring low or minimum amounts of shear reinforcement. Specimen variables included fiber volume content, fiber length, fiber aspect ratio, fiber tensile strength, and concrete compressive strength. Conventionally reinforced control specimens were also tested for comparison. All panels were tested under in-plane pure-shear monotonic load conditions.

The test results obtained support the following conclusions:

1. Relative to conventionally reinforced concrete elements containing low or minimum amounts of shear reinforcement, FRC elements can achieve comparable shear strength and deformation response with improved crack control characteristics.

2. Fiber volume content was found to have a significant effect on shear behavior. A fiber volume content of approximately 1.0% was required to achieve satisfactory performance in terms of shear strength, deformation ductility, crack width, and crack spacing. Only a minor improvement was obtained, however, by increasing the fiber volume content from 1.0 to 1.5%, possibly due to fiber saturation. A fiber volume content of 0.5% was found to be insufficient to guarantee adequate shear resistance and shear deformation response.

3. The fiber type was also found to be influential. Fibers with a high aspect ratio resulted in much improved post-cracking deformation capacities and crack control characteristics compared to fibers with a low aspect ratio. Panels containing short fibers were found to exhibit a higher shear resistance and a higher maximum concrete principal tensile stress than panels containing long fibers.

4. The concrete compressive strength was found to have no significant influence on the shear response of the fiber reinforced panels for the range of concrete strengths examined. Likewise, the fiber tensile strength was found to have little influence on the shear response of the fiber-reinforced panels.

5. The addition of fibers significantly improved the post-cracking principal tensile stress-strain response of the concrete. Even with a low fiber volume content of 0.5%, a post-cracking principal tensile stress-strain response comparable to that observed in a conventionally reinforced panel was achieved. This demonstrates the ability of the fibers to transmit tensile stresses across a crack. Strain-hardening behavior was observed in those panels with a fiber volume content of 1.0% or higher.

6. The principal compressive stress-strain response was not significantly influenced by fiber volume content or fiber type.

NOTATION

A_s = cross-sectional area of reinforcement
d_b = diameter of reinforcement steel
d_f = diameter of steel fibers
E_s = modulus of elasticity of reinforcement
f_c' = concrete compressive strength
$f_{c1,cr}$ = principal tensile stress of concrete at onset of cracking
$f_{c1,max}$ = maximum principal tensile stress of concrete
$f_{c1,u}$ = principal tensile stress of concrete at failure
$f_{c2,u}$ = principal compressive stress of concrete at failure
f_{sx} = reinforcement stress in x-direction
f_{sy} = reinforcement stress in y-direction
f_{uf} = ultimate tensile strength of steel fibers
f_{us} = ultimate tensile strength of reinforcement
f_{ys} = yield strength of reinforcement
l_f = length of steel fibers
s_m = average crack spacing perpendicular to crack
V_f = volume content of steel fibers
v_{cr} = applied shear stress at onset of cracking
v_u = maximum applied shear stress
w_m = average crack width
ε_{us} = ultimate strain of reinforcement
ε_{ys} = yield strain of reinforcement
γ_{cr} = shear strain at onset of cracking
γ_u = shear strain corresponding to v_u
ρ_x = reinforcement ratio in x-direction
ρ_y = reinforcement ratio in y-direction

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