International Journal of Concrete Structures and Materials
Vol.9, No.3, pp.267–281, September 2015
DOI 10.1007/s40069-015-0109-4
ISSN 1976-0485 / eISSN 2234-1315

Shear Strength of Prestressed Steel Fiber Concrete I-Beams

Padmanabha Rao Tadepalli[1),*], Hemant B. Dhonde[2)], Y. L. Mo[3)], and Thomas T. C. Hsu[3)]

(Received March 2, 2015, Accepted July 30, 2015, Published online September 2, 2015)

Abstract: Six full-scale prestressed concrete (PC) I-beams with steel fibers were tested to failure in this work. Beams were cast without any traditional transverse steel reinforcement. The main objective of the study was to determine the effects of two variables—the shear-span-to-depth ratio and steel fiber dosage, on the web-shear and flexural-shear modes of beam failure. The beams were subjected to concentrated vertical loads up to their maximum shear or moment capacity using four hydraulic actuators in load and displacement control mode. During the load tests, vertical deflections and displacements at several critical points on the web in the end zone of the beams were measured. From the load tests, it was observed that the shear capacities of the beams increased significantly due to the addition of steel fibers in concrete. Complete replacement of traditional shear reinforcement with steel fibers also increased the ductility and energy dissipation capacity of the PC I-beams.

Keywords: shear, steel fibers, prestress concrete, full-scale beams.

1. Introduction

When a concrete element is subjected to shear stress it causes principal diagonal tensile and compressive stresses in the element. Concrete starts cracking when the applied principal tensile stress exceeds the tensile strength of concrete. This cracking causes softening in the other principal direction and reduces the compressive strength of concrete. When the applied principal compressive stress exceeds the softened compressive strength, crushing of concrete occurs. This phenomenon is known as shear failure. This failure could be very brittle since tensile strength of concrete is much less than its compressive strength. Therefore, to enhance the behavior of concrete subjected to shear forces, one of the methods is to improve its tensile strength by adding steel fibers.

Steel Fiber Reinforced Concrete (SFRC) is conventional concrete reinforced with discrete fibers of a short length and small diameter. It has been extensively used by many researchers over the past two decades to improve the post cracking behavior of concrete. SFRC is important in seismic, impact and blast resistant structures due to its improved properties over conventional concrete. Full or partial replacement of mild steel reinforcement with steel fibers can also save considerable labor costs and time for construction. Although SFRC is mainly used along with mild steel bars as transverse reinforcement, studies have shown (Dhonde 2006) that presence of only steel fibers could enhance the shear behavior of concrete even in the absence of mild steel.

Dhonde (2006) found that most of the fiber reinforced beams have performed better in controlling shear crack widths than the beam with mild steel shear reinforcement. It is noted that the 4.2 % of shear steel, has its crack width greater than that of beam with 0.83 fiber factor that is the volume percentage multiplied by the aspect ratio of steel fibers. Study clearly indicated that the replacement of stirrups by steel fibers plays an important role in the crack control of the beams. It was found that the steel fiber reinforced beam had higher shear strength and greater ductility than the control beam.

In fully prestressed beams with a fiber volume fraction of 1.5 %, Padmarajaiah and Ramaswamy (2001) found an increase of shear strength up to 20 % at the first crack as well as at the peak. They also found that the fiber inclusion alters the a/d ratio that divides the flexure and shear critical failure mode. Thomas and Ramaswamy (2006) observed similar results in increment of shear strength when fibers were added to concrete. Additionally, they found that high strength concrete benefits more from fibers. That may be attributed to better bond characteristics between fibers and high strength concrete.

Meda et al. (2005) conducted a series of experiments on prestressed SFRC beams and concluded that the beams reinforced only with steel fibers show a similar, or even better, post cracking behavior than the beams with minimum amount of transverse reinforcement. The study also showed that the addition of steel fibers to replace conventional

[1)]American Global Maritime, Houston, TX 77079, USA.
*Corresponding Author; E-mail: tvvssprao@gmail.com
[2)]Civil Engineering Department, Vishwakarma Institute of Information Technology, Pune 411048, India.
[3)]Department of Civil and Environmental Engineering, University of Houston, Houston, TX 77204, USA.

transverse reinforcement improved the shear strength significantly. Steel fibers were also found to reduce the width of shear cracks more effectively than conventional steel reinforcement, thus improving durability of concrete.

In another study (Cho and Kim 2003) conducted on SFRC beams, it was found that fibrous concrete beams eventually collapsed from the severely localized deformations at one or two major cracks regardless of the failure mode.

Tan et al. (1995) showed through his experiments that inclusion of steel fibers enhanced the ultimate shear strength of partially prestressed concrete (PC) beams. The load at which the first shear crack appeared increased with an increase in steel fiber content. The study also found that stirrups may be replaced by an equivalent amount of steel fibers without significantly affecting the behavior and strength in shear of partially prestressed concrete beams.

Langsford et al. (2007) tested 13 fully prestressed steel fiber reinforced concrete beams without stirrups under shear loading. The shear span to depth ratio was varied from 1.5 to 2 and volume fraction of hooked end steel fibers from 0.5 to 1.2 %. They found out that addition of 0.5 % volume fraction of steel fibers increased the shear carrying capacity by 30 and 25 % at an a/d ratio of 2.0 and 1.5, respectively. The increase is 50 and 33 % with addition of 1.2 % volume fraction of steel fibers for both cases, respectively.

Narayanan and Darwish (1987) carried out 36 shear tests on simply supported rectangular prestressed concrete beams, containing steel fibers as web reinforcement. It was found that the failure modes for beams with prestress and without prestress are similar. Ultimate shear strengths increased up to 95 % when steel fibers were added to concrete.

Abdul-Wahab and Al-Kadhimi (2000) found similar results as mentioned above even with unbounded tendons. They also observed that increase in shear strength is more significant at lower a/d ratios. Junior and De Hanai (1999) found steel fibers are more effective when used in combination with traditional stirrups.

With regard to the behavior of steel fiber concrete under shear, when a concrete element is subjected to pure shear stress, it imposes tension and compression on the element in principal directions as shown in Fig. 1. In the case of normal concrete, the cracking due to tension in one direction causes the concrete to soften in the orthogonal direction. When steel fibers are added to concrete they contribute to shear strength in three ways.

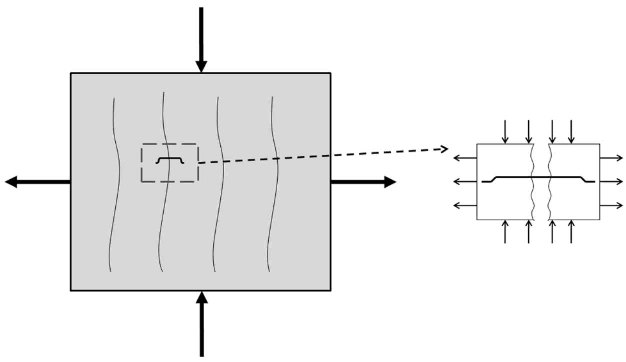


Fig. 1 Fiber concrete under principal tension and compression.

i. By improving the post cracking tensile behavior of concrete, which in turn reduces the cracking and hence the softening of the compression direction.
ii. Addition of steel fibers also improves the compressive behavior by confining the lateral strain. Many researchers found that addition of steel fibers improves the compressive strength of concrete up to 15 %.
iii. The compression in the normal direction acts as a confinement for the steel fibers as shown in Fig. 1, improving their bond with concrete and hence the tensile behavior of SFRC.

2. Research Significance

Addition of deformed steel fibers improves the shear behavior of prestress concrete. However, limited studies are available on large or full-scale prestressed concrete specimens with high strength concrete. This research focuses on full-scale testing of fully prestressed SFRC beams made with high strength concrete to reduce or completely eliminate transverse steel reinforcement and aiming towards the development of rational and simple shear design provision for SFRC structures. Furthermore, the replacement of stirrups with steel fibers will potentially reduce the labor and construction costs.

3. Experimental Program

The test specimens consisted of Texas Department of Transportation (TxDOT, Texas-USA) Type-A beams (prestressed I-Beams). Six 7.6 m (25 ft) long beams (R1–R6) were fabricated with Prestressed Steel Fiber Concrete (PSFC) to study the behavior of the beams in web-shear and flexure-shear mode of failure under monotonic loading. Steel fibers, which have double hooked ends and are collated, were chosen to produce the PSFC beams. The beam cross section is shown in Fig. 2. The primary testing variables investigated were the amount of steel fiber (fiber factor) and the mode of shear failure (i.e., shear span-to-effective depth ratio, *a/d*). No traditional transverse rebars (stirrups) were used in any of the beams; the shear reinforcement consisted solely of steel fibers. Beams R1 through R4 were designed to fail in web-shear with *a/d* ratio of 1.6, while Beams R5 and R6 were designed to fail in flexure-shear with *a/d* ratio of 4.2.

Table 1 summarizes the test variables for Beams R1–R6. Beam R1 with a fiber factor of 0.4 was designed to fail in web-shear. Beams R2, R3 and R4 were made using fiber factor of 0.55, 0.83 and 1.23, respectively and were also designed to fail in web-shear. Beams R5 and R6 with a fiber factor of 0.4 and 1.23, respectively were designed to fail in flexural-shear.

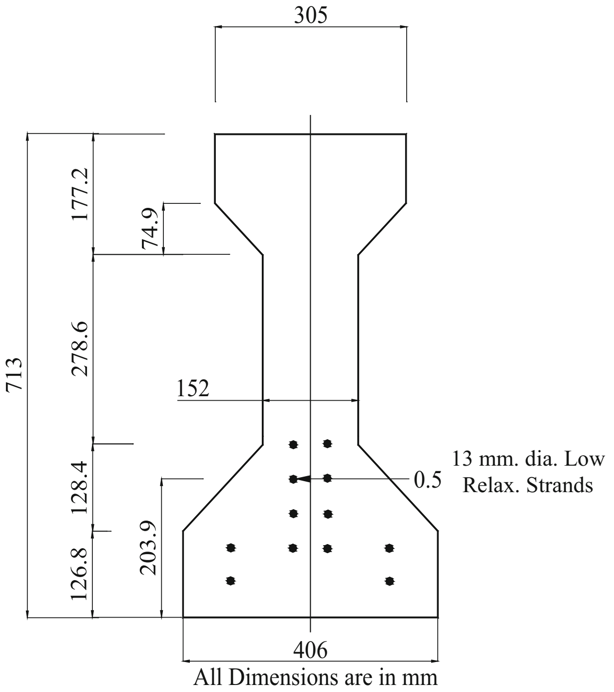


Fig. 2 Cross section of PSFC I-Beam. *Note* 1″ = 25.4 mm.

3.1 Details of PSFC I-Beams

The cross-section of the TxDOT Type A beam is shown in Fig. 2. The total height of the beam was 713 mm (28 in.) and the widths of the top and bottom flange were 305 mm (12 in.) and 406 mm (16 in.), respectively. The width of the web was 152 mm (6 in.). The prestressing tendons in all beams were straight. The location of prestressing tendons is also shown in Fig. 2. Twelve 13 mm (0.5-inch) diameter, 7-wire, low-relaxation strands were used as prestressing steel to resist flexure. The prestressing strands had an ultimate tensile strength of 1863 MPa (270 ksi). The total length of the beams tested was 7.62 m (25 ft) while the test span-length was 7.32 m (24 ft). The W, V, R and Y rebars (Fig. 3) were installed to resist the end zone bearing, spalling and bursting stresses. The sizes of the rebars are as follows: # 4 for R bars and V rebars, # 5 for W rebars, and # 6 for Y rebars. A view of the beam prestressing bed just before installing the formwork is also shown in Fig. 4.

3.2 Materials and Mix Design

Two types of steel fibers were used to cast the PSFC I-Beams. The type of steel fibers, selection of optimum and practical fiber dosage, and suitable fiber factors to cast the beams were selected based on previously carried out experiments at the University of Houston (Tadepalli et al. 2013). The steel fibers were 'trough' shaped with hook at both ends and were collated together with water soluble glue. The long fiber (LF) is shown in Fig. 5a and the short fiber (SF) is shown in Fig. 5b. The long fibers had a length of 60 mm, a diameter of 0.75 mm (aspect ratio of 80) and had a tensile strength of 1040 MPa. The short fibers were 30 mm long and 0.55 mm in diameter (aspect ratio of 55) and had a tensile strength of 1100 MPa.

Table 2 gives the details of the steel fibers used in this experimental study. The steel fibers were relatively stiff and glued into bundles, i.e., collated. The glue dissolved in the water during mixing, thus dispersing the fibers in the mix as shown in Fig. 6.

Table 3 shows the details of different constituent materials of concrete used to cast the PSFC I-Beams. Locally available materials were used to prepare the high strength fibrous concrete mixes.

3.2.1 Cement

High early strength cement was used in all the mixes, since it was necessary to develop high release strength at an early age in the PSFC I-Beams. Portland cement (Type-III) conforming to ASTM-C150, and fly ash (Class-F) conforming to ASTM-C618, were the only powder materials used. Fly ash was added to the mix to enhance workability, curtail rise in temperature and reduce cost.

3.2.2 Coarse and Fine Aggregates

The mixes utilized uniformly-graded, rounded, river-bed, coarse aggregates of 3/4 inch nominal size (AASHTO-T27, 1996) and well-graded, river-bed sand (AASHTO-M43, 1998).

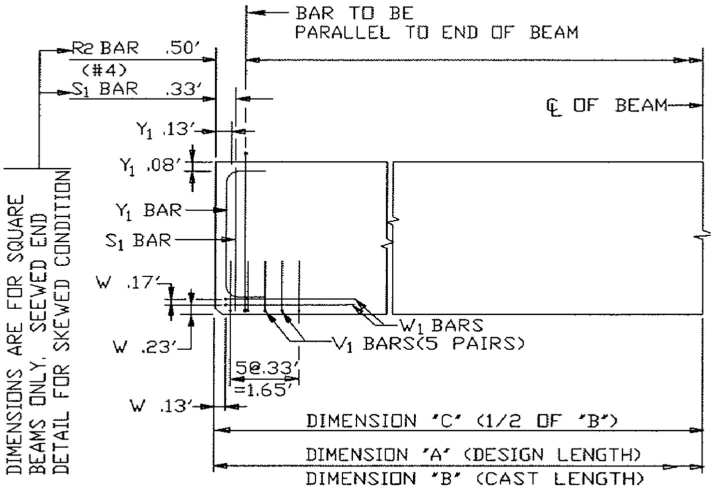
3.2.3 Admixtures

A Polycarboxylate-based High Range Water Reducing (HRWR) agent conforming to ASTM C 494-1999, Class-F was used to achieve workable concrete mixes. A retarder

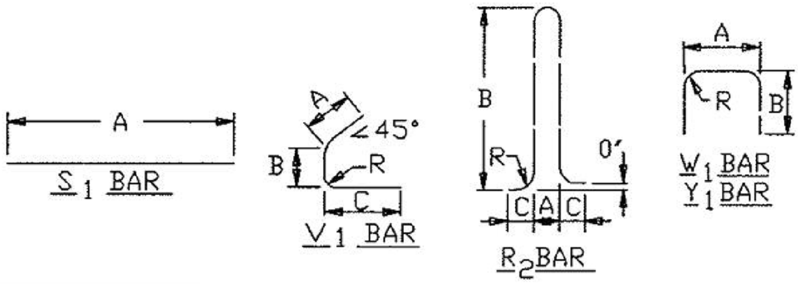
Table 1 Test variables of PSFC I-Beam.

Beam ID	Designed mode of failure	Concrete compressive strength (MPa) ksi	Volume of steel fiber reinforcement V_f	Fiber factor $[(L_f/D_f)V_f]/100$
R1	Web shear	86.9 (12.6)	0.5 % LF	0.40
R2	Web shear	90.4 (13.1)	1 % SF	0.55
R3	Web shear	82.1 (11.9)	1.5 % SF	0.825
R4	Web shear	73.1 (10.6)	1.5 % SF + 0.5 % LF	0.825 + 0.40 = 1.225
R5	Flexural shear	84.2 (12.2)	0.5 % LF	0.40
R6	Flexural shear	88.3 (12.8)	1.5 % SF + 0.5 % LF	0.825 + 0.40 = 1.225

LF long fibers, L_f/D_f 80, *SF* short fibers, L_f/D_f 55, L_f length of steel fiber, D_f diameter of steel fiber.



(a) **End Zone Reinforcement**



BEAM MARK						
BAR	SIZE	A	B	C	D	R
R2	#4	3 1/4"	2'-8"	4"	——	1"
S1	#5	——	–STRAIGHT–		——	——
W1	#5	8 1/2"	3'-0"	——	——	1 1/4"
Y1	#6	1'-10"	1'-0"	——	——	3"
V1	#4	6 3/4"	3"	11 1/4"	——	1"

(b) **Reinforcement: Layout and Schedule**

Fig. 3 Details of end zone reinforcement in PSFC I-Beams. *Note* $1'' = 25.4$ mm.

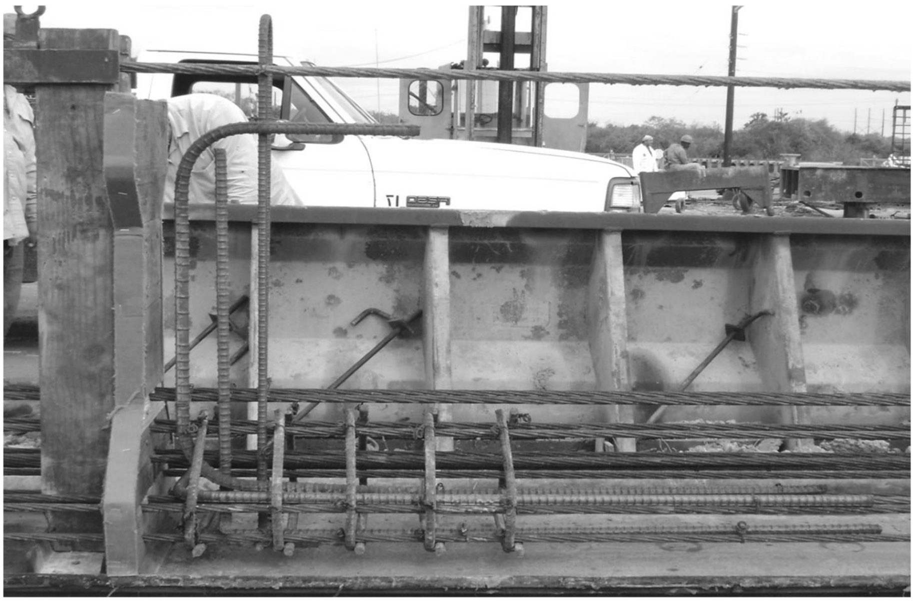
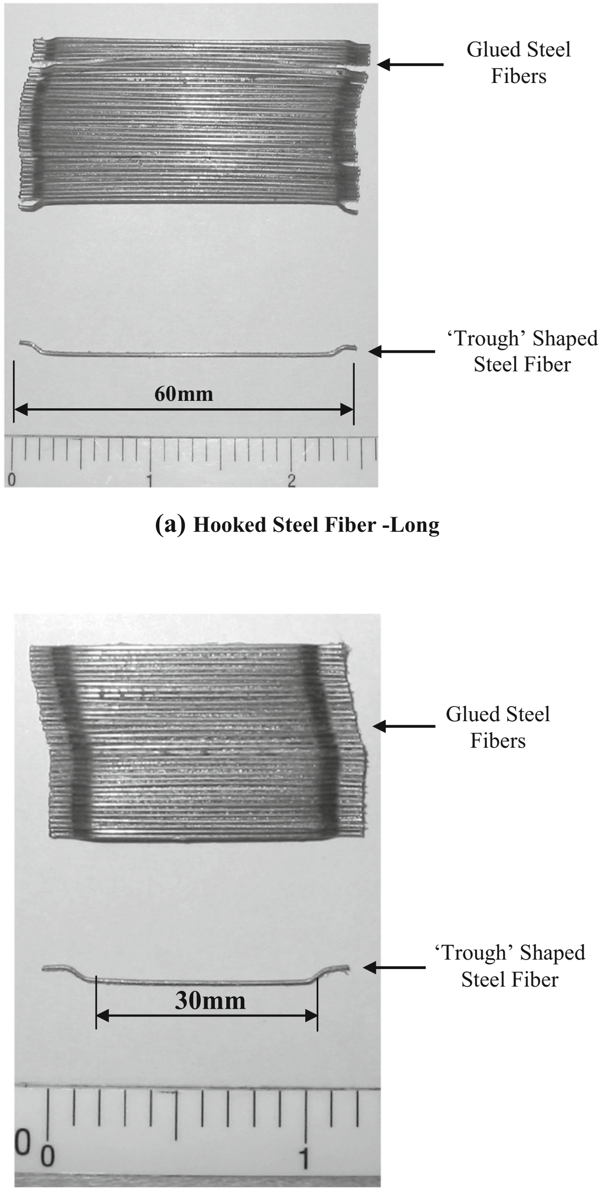


Fig. 4 Picture of end zone reinforcement.



(a) Hooked Steel Fiber -Long

(b) Hooked Steel Fiber -Short

Fig. 5 Steel fibers used in PSFC I-Beams.

conforming to ASTM-C494/C494 M, Class-B was added to the mixes as required to delay the initial *setting* of the mix. Concrete mix design used to cast each of the PSFC beam is given in Table 4. The amount of fibers used in a concrete mix can also be reported as its fiber-factor, which is the product of the aspect ratio of the fibers and the volume of fibers in the mix, i.e., $(L_f\text{ha}/D_f)V_f$.

3.3 Fabrication of PSFC I-Beams

All steel fiber concrete mixes were mixed in a 4.6 m^3 (6 yd^3) drum mixer at the Texas Concrete Company's (Victoria, Texas) precast plant. 1.5 m^3 (2 yd^3) of concrete was mixed for each beam. The six PSFC I-Beams were cast in two groups on two different days. Beams R2, R3 and R6 were first cast concurrently in a long-line prestressing bed using Type-A steel formwork. The strands were prestressed by hydraulic jacks against the prestressing bed ends. The second group of three Beams R1, R4 and R5 were cast 1 week after the first group. Concrete for both the groups was prepared in the plant's mixer, transported to the casting location (prestressing bed), and placed into the formwork using a mobile hopper. During concrete placement, spud vibrators were used for compacting the fibrous concrete.

Casting and compaction of PSFC I-Beams was relatively fast and easy in comparison with the conventional I-beams, even when the mix used large dosage of steel fibers. This was because transverse reinforcement in the beams was totally absent, causing no hindrance to the compaction of the fiber reinforced mix. Thus, fiber reinforced concrete was found to be relatively easy to compact in the absence of any traditional reinforcement. Just after mixing the steel fiber concrete (i.e., before casting the beams), slump tests were carried out for all the mixes.

It has been found out that the true workability of SFC cannot be ascertained by the slump tests as the fibers significantly affect the rheology of fresh concrete. At lower fiber factors concrete was more workable than at higher fiber factors based on the laboratory and field casting experiences. During casting of full-scale beam specimens it was interesting to observe that the compaction energy required for SFC was almost same as for non-fibrous concrete. This was evidently due to the absence of the traditional rebars resulting in lower hindrance to compact fresh concrete in the beams. A satisfactory level of workability and finish was achieved for all the fibrous mixes used in casting the beams.

Curing of the PSFC I-Beams was carried out until a minimum concrete compressive strength of 28 MPa (4000 psi) was obtained in the beams, sufficient for release of prestress. One day after casting, the prestressing strands were slowly released and the beams were de-molded.

3.4 Test Setup

The PSFC I-Beams were placed in a vertical loading system at the University of Houston and were subjected to vertical load up to their maximum shear capacity, until failure. The testing system was a specially built steel loading

Table 2 Properties of steel fiber used in PSFC I-Beams.

Fiber type		Length (mm) L_f	Diameter (mm) D_f	Aspect ratio L_f/D_f	Tensile strength (MPa)
Hooked end, collated	Long fiber (LF)	60	0.75	80	1040
	Short fiber (SF)	30	0.55	55	1100

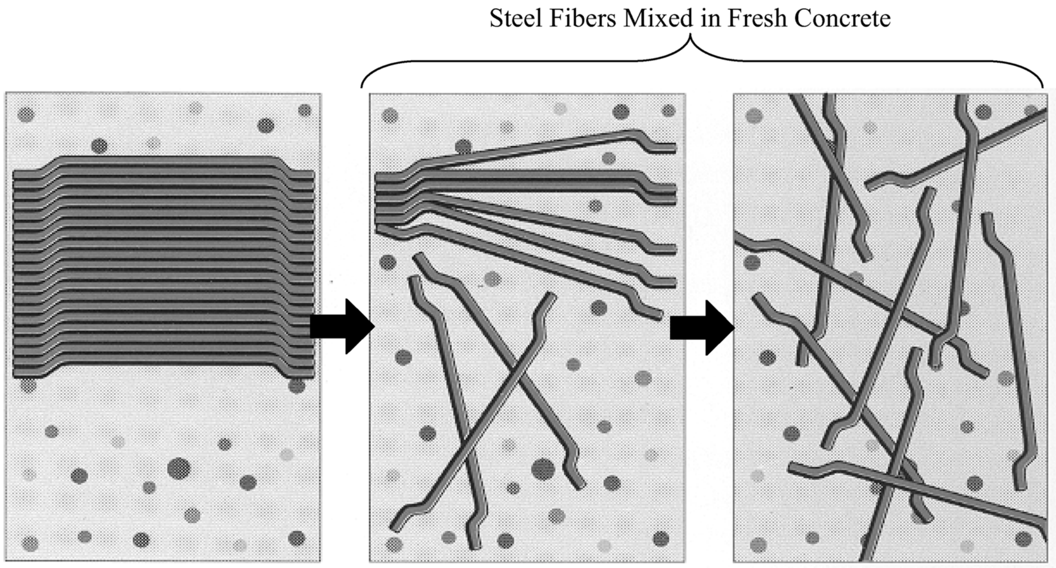


Fig. 6 Dispersion of glued (collated) steel fibers in concrete.

Table 3 Materials used in steel fiber concrete.

Material	Type
Cement	ASTM C150 Type-III
Fly ash	ASTM C618 Class F
Coarse aggregate	AASHTO T27
Fine aggregate	AASHTO M43

frame with four actuators as depicted in Fig. 7. Two of the four actuators (namely actuator B and actuator C) attached to the steel frame were used to apply vertical loads on the beams. Each of the actuators had a maximum load capacity of 1420 kN (320 kips). Details regarding the design, layout and capabilities of the loading system can be found in Laskar (2009).

Load application points and support locations for PSFC I-Beams are shown in Fig. 8. Support bearings beneath the beams were located 6 in. from each beam end. The applied loads from actuators B and C were 0.92 m (3 ft) away from each of the supports for Beams R1, R2, R3 and R4, and at 2.44 m (8 ft) from each of the supports for Beams R5 and R6. Actuator loads were applied on the beam via a steel roller and bearing plate assembly. This assembly consisted of two steel rollers of 51 mm (2 in. diameter and 305 mm (12 in.) long, sandwiched between two steel bearing plates 152 mm wide × 305 mm long × 51 mm thick (6 in. × 12 in. × 2 in.). This ensured a uniform and frictionless load transfer from the actuators to the top surface of the beam.

A freely movable roller assembly (roller-support) and a fixed roller assembly (hinged-support) were provided at the North and South beam ends, respectively. This enabled free rotation and longitudinal movement of the simply supported beam during test. All the steel bearing plates and rollers were heat-treated to maximum hardness in order to minimize local deformations. Lead sheets were also used between the load bearing plates and beam surface to help distribute the load evenly.

Beam displacements and concrete strains at important locations on the beam were measured continuously throughout the load test using Linear Variable Differential Transformer Linear Variable Differential Transformer s (LVDTs). A group of seven LVDTs was used at either end and on each side of the beam to measure smeared (average) concrete strains within the beam-web. The LVDTs were arranged in a rosette form as shown in Fig. 9. Each rosette consisted of two vertical, three horizontal, and two diagonal LVDTs. The rosettes were mounted on the beam adjacent to the loading points where the web-shear or flexure-shear failure was anticipated (Figs. 8a, b, 9).

A total of six LVDTs were used to continuously monitor and measure the vertical deflections of the beam. LVDTs were placed under each beam support (North and South ends) on either sides of the beam (West and East). Two pairs of LVDTs were positioned under the beam at each of the two loading points. These LVDTs were used to measure the

Table 4 Concrete mix design for PSFC I-Beams.

Component (kg/m^3)	R1 and R5	R2	R3	R4 and R6
Cement	364.0	364.0	364.0	364.0
Fly ash	121.5	121.5	121.5	121.5
Cementitious material	485.6	485.6	485.6	485.6
Water	146.3	146.3	146.3	146.3
Water/cement ratio (w/c)	0.4	0.4	0.4	0.4
Water/cementitious ratio	0.3	0.3	0.3	0.3
Coarse aggregate (CA)	1125.1	1125.1	1125.1	1125.1
Fine aggregate (FA)	596.5	596.5	596.5	596.5
CA/FA ratio	1.88	1.88	1.88	1.88
HRWR/superplastisizer (gm/100 kg)	688	688	688	688
Fibers	39.5 LF	79 SF	118.5 SF	119 SF + 39.5 LF
Retarder (gm/100 kg)	250	250	250	250

1 kg/m^3 = 1.686 lb/yd^3.
LF long fibers, *SF* short fibers.

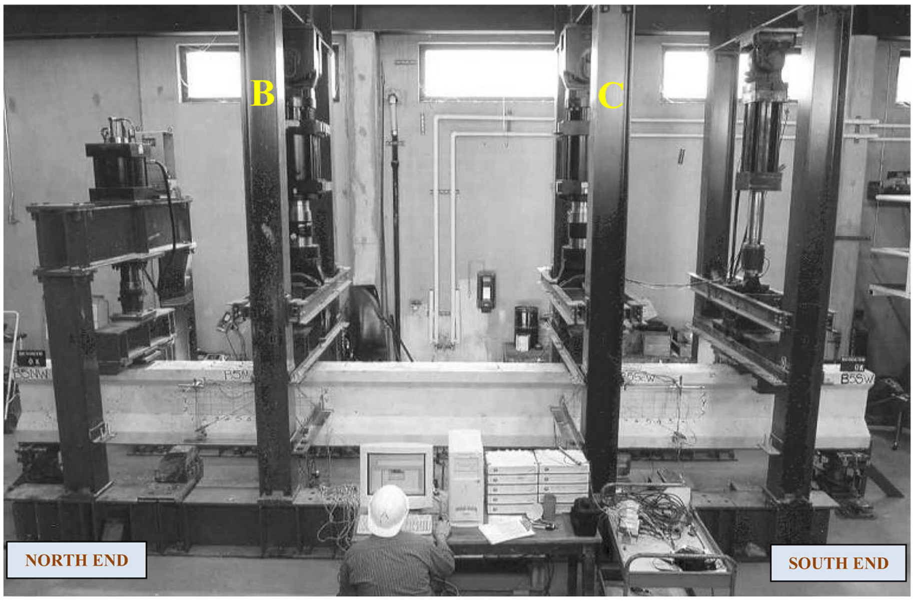


Fig. 7 Test set-up at University of Houston.

deflections of the beam. An additional set of LVDTs was used to monitor potential lateral displacements of the beam.

Two 2225 kN (500 kips) capacity load cells were used to monitor support reactions at each beam-end. Two load cells, attached to the loading actuators (B and C), were used to measure the applied load on top of the beam. During a test, force equilibrium between the applied loads (actuators B and C) and the measured reactions (load cells) was always verified.

Non-stop measurement of all the experimental data (beam deflections, strains, loads, and support reactions) from the above sensors were continuously monitored and stored by data acquisition system, during a load test. Shear cracks, which formed on the beam web during a load test, were regularly marked on a grid, as shown in Fig. 9. The crack widths were measured using a hand-held microscope having a 0.025 mm (0.001 in.) measuring precision.

The two hydraulic actuators were precisely controlled in force or displacement modes by a servo-controlled system. Actuators B and C were initially used to apply shear force on the beam in force control mode at a rate of 22 kN/min (5 kips/min). During a test, the shear load–displacement curve for a beam was continuously monitored visually on a display screen. When the slope of this load–displacement curve started to decreasing (flatten-out), the control mode of the actuators was switched to displacement control with a rate of

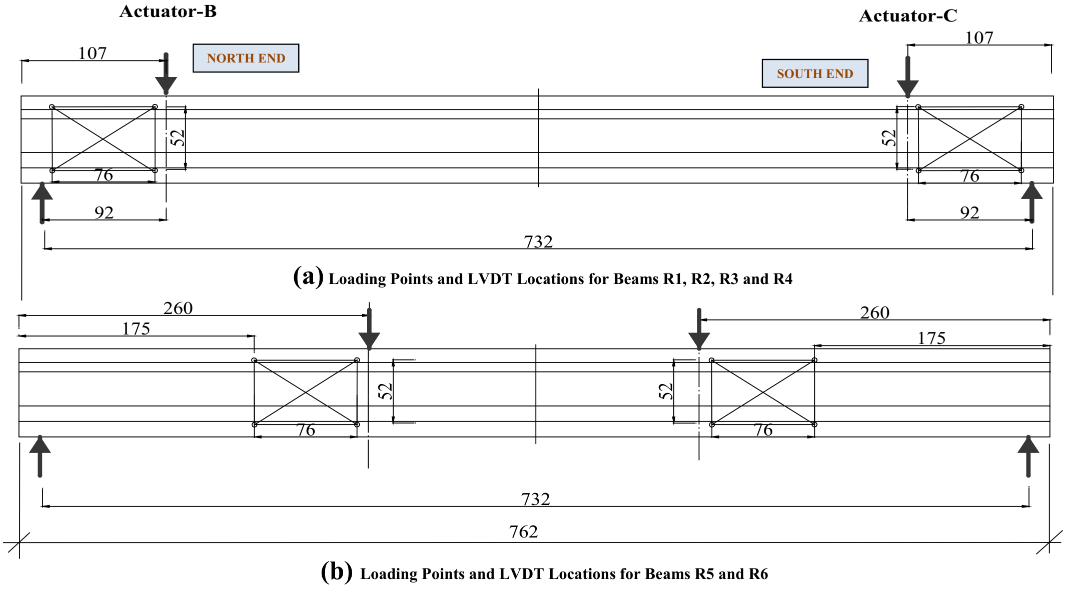
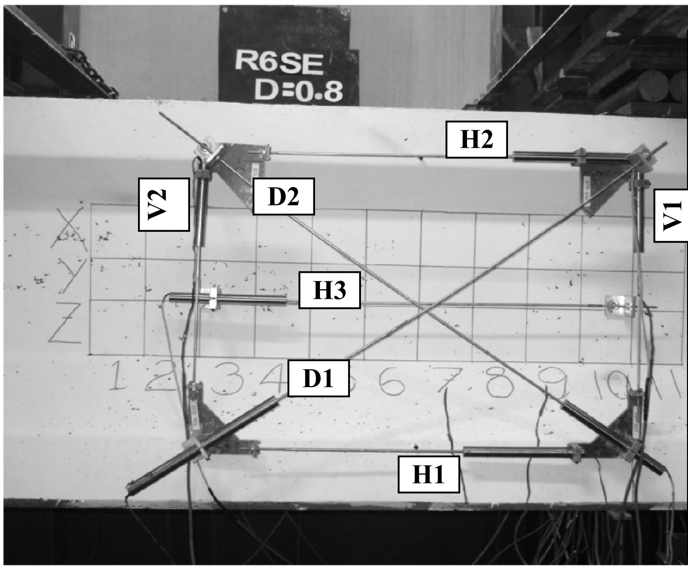


Fig. 8 Loading and support locations in PSFC I-Beams. *Note* All dimensions in cm: 1″ = 25.4 mm.



LVDT ID	Measured Smeared Strain in Concrete
H1	ε_{H1}
H2	ε_{H2}
H3	ε_{H3}
V1	ε_{V1}
V2	ε_{V2}
D1	ε_{D1}
D2	ε_{D2}

Fig. 9 Typical LVDT rosette used to measure smeared/average concrete strains in PSFC beams.

5 mm/h (0.2 in./h). This displacement control mode was maintained until the failure occurred at either end of the beam. The displacement control feature was essential in capturing the ductility/brittleness behavior of the beam as it failed in shear.

4. Experimental Results

Table 5 shows the experimental ultimate strengths at failure for the six beams tested (R1–R6). During the test, although application of load and support arrangements were symmetric for all the beams; only in the case of Beam R2 web-shear failures occurred simultaneous at both the ends. In all the other beams, the weaker end failed first. Even though Beam R3 ultimately failed in flexure, the shear load at failure at both the ends was close to the web-shear capacity, as indicated by the spalling of concrete struts in the web region of this beam.

While testing Beam R4 it was found that the shear capacity was surprisingly increased beyond the anticipated value due to the use of higher fiber-factor. Hence, the beam would have failed in flexure instead of the desired web-shear failure mode. Therefore, Beam R4 was reinforced with FRP sheets (installed on the beam soffit at the bottom flange) to increase its flexural capacity. The beam was then tested using a shorter span of 4.26 m (14 ft), which failed in web-shear. This test is denoted as 'R4-Short' hereafter in the discussion.

Beams R5 and R6 failed near a region adjacent to the loading point (i.e., at one third span of the beam) in flexural-shear and flexure failure mode, respectively. As a result, both these beams did not have a sufficiently long undamaged

Table 5 Experimental ultimate strengths at failure for PSFC I-Beams.

Beam ID and failed end	Steel fiber by volume (%)	Fiber factor	Concrete compressive strength (MPa)	Failure mode	Ultimate shear capacity (kN)	Ultimate moment capacity (kN/m)	Max. shear at ultimate moment (kN)	Max. moment at ultimate shear (kN/m)
R1-North	0.5 % LF	0.40	86.9	Web-shear	1175	–	–	1078
R2-North	1 % SF	0.55	90.4	Web-shear	1250	–	–	1146
R2-South	1 % SF	0.55	90.4	Web-shear	1313	–	–	1205
R3	1.5 % SF	0.825	82.1	Flexure/web-shear	–	1191	1299	–
R4-North (short beam)	1.5 % SF + 0.5 % LF	1.225	73.1	Web-shear	1540	–	–	–
R5-South	0.5 % LF	0.40	84.2	Flexural-shear	472	–	–	1153
R6	1.5 % SF + 0.5 % LF	1.225	88.3	Flexure	–	1243	507	–

1 MPa = 145 psi, 1 kN = 0.225 kip, 1 kN/m = 0.735 kip-ft.
LF long fibers, *SF* short fibers.

length for another re-test in flexure-shear mode. Hence each of these two beams could provide only one failure capacity. Beam R5 failed on the South side without any prior warning. The sudden brittle failure of beams subjected to flexure-shear was explained by Kani (1964). When the strength of concrete "teeth" formed between the flexural cracks is smaller than the remaining arch, the beam fails suddenly as soon as the strength of teeth is compromised. Specimen R6 apparently failed in flexure mode instead of the targeted flexure-shear mode. Beam R6 demonstrated much higher web-shear capacity than expected, owing to the use of higher fiber-factor.

The comparison of shear strength of PSFC I-Beams tested in this work (Table 5) shows that shear capacity of beams can be significantly increased due to the addition of steel fibers in concrete. The beam test results reveal a good co-relation between the fiber-factor and shear strength. The general trend detected was that with an increasing fiber-factor, shear strength also increased. The failure of beam R5 suggested that a fiber-factor of more than 0.4 (0.5 % by volume of LF) may be necessary to serve as minimum shear reinforcement. ACI 318 code recommends a minimum of 0.75 % by volume of steel fibers as minimum shear reinforcement.

The ACI provisions are basically formulated on experimental studies of non-PC beams and majority of them had a cylinder compressive strength less than 42 MPa (6000 psi). Nevertheless, in a prestressed concrete beam, the beneficial effect from prestressing forces could further relax the minimum required fiber volume fraction. Further, concrete compressive strengths much higher than 42 MPa (6000 psi) are commonly used in PC for higher structural performance. The achievement of high shear strength in the PSFC beams in this study suggests a minimum amount fiber volume less than 0.75 % is feasible for replacing traditional shear reinforcement in PC members. Therefore, the current ACI 318 requirement may hamper the use of SFC in structures with PC members made of high strength concrete. However, further research would be required to establish the minimum fiber-factor required for PSFC members to avoid brittle failure.

The crack pattern and photograph at failure of all the PSFC I-Beams are shown in Fig. 10. The web-shear failures in beams R1–R4 were noticeably along a single shear crack which formed between the support and loading points at failure. Studying the failure photographs closely, it can be observed that the damage to the beams with web-shear failure mode (R1–R4) was less pronounced in comparison to the damage in beams with a destructive flexure-shear mode of failure (R5 and R6).

From the shape of the load–deflection curves of the PSFC I-Beams, shown in Fig. 11, it can be seen that the beams which failed in web-shear mode (R1–R4) demonstrated higher shear capacities compared to the beams that failed in flexural-shear mode (R5 and R6). It is therefore evident that the shear span-to-effective depth ratio (a/d) has a significant effect on the web-shear and flexure-shear strengths of PSFC I-Beams. Laskar (2009) reported similar results for traditional TxDOT PC I-Beams. The PSFC I-Beams that failed in flexural-shear or flexure mode displayed higher ductility than the beams which failed in web-shear mode.

The advantageous effect of steel fibers on shear strength of PSFC I-Beams can be observed by examining Fig. 11. The values of shear force plotted in this figure were obtained from the load cells under the beam's end-supports and were also verified by the load equilibrium computations. The net deflection was obtained from the difference in readings of LVDT placed under the beam at the particular actuator location and the readings of LVDT placed at the corresponding support. Hence, the beam total deflection values were subtracted by the support settlement and then used to plot the load–deflection curves (Fig. 11).

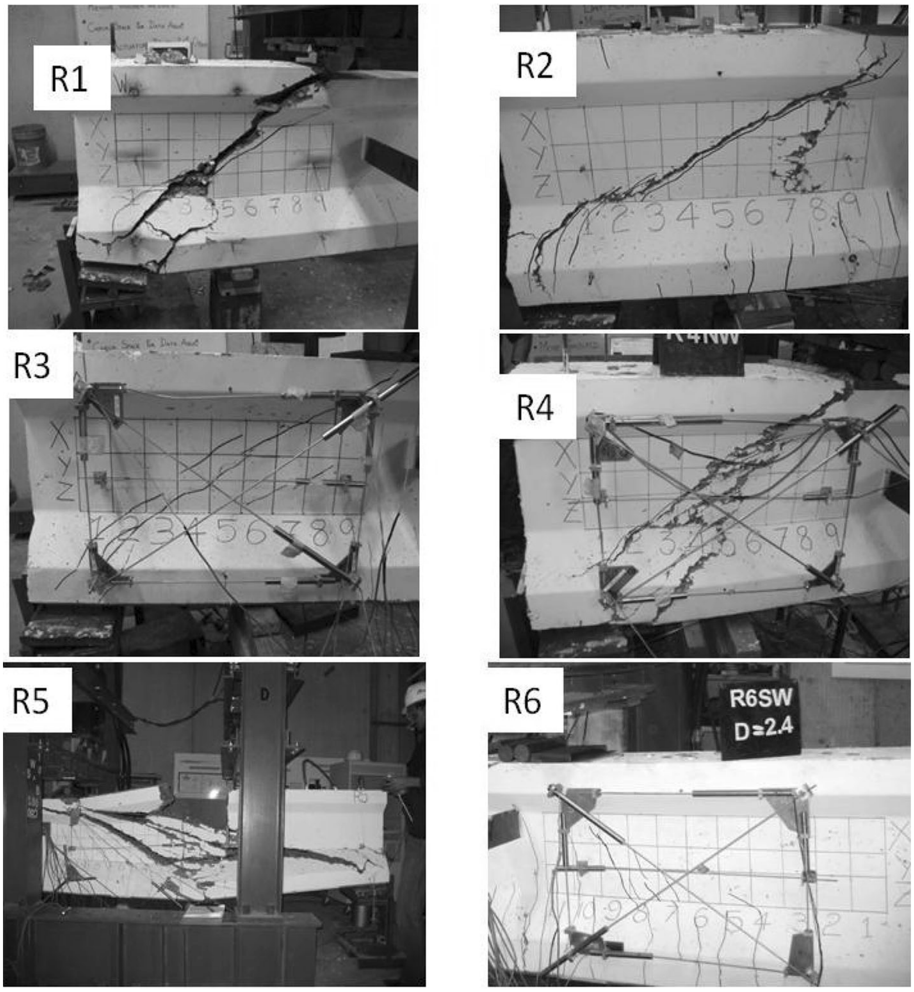


Fig. 10 PSFC I-Beams at failure.

Since the compressive strength of concrete for various I-Beams tested were different, the beam's ultimate shear capacity was normalized with the corresponding compressive strength of concrete to better compare all beam results. Normalized shear was calculated as:

$$\text{Normalized Shear Force of PSFC I-Beam} = \frac{\textit{Shear Capacity}}{bd\sqrt{f_c}} \quad (1)$$

where experimental shear capacity is in N. f_c is in MPa., and b and d are in mm.

The normalized shear force versus net deflection curves for PSFC I-Beam are shown in Fig. 12. It can be clearly seen that the shear behavior of beams improves with increasing fiber-factor. The ductility in beams also increased with an increase in the fiber factor. This performance shows that the complete replacement of traditional transverse steel by steel fibers is very effective in resisting the shear force.

To understand the true effectiveness of steel fibers as shear reinforcement, the results of PSFC I-Beams are compared with the results of conventional beams (LB2 and LB4) having mild steel as shear reinforcement, tested by Laskar (2009). Laskar's beams had the same compressive strength of concrete, *a/d* ratios, test span and total prestressing force as the PSFC I-Beams. I-Beam LB2 had a transverse steel ratio of 1 % by volume of concrete and failed in web-shear mode, while LB4 had a transverse steel ratio of 0.17 % by volume of concrete and failed in flexure-shear mode. The comparisons of web-shear and flexural-shear failures for fibrous and non-fibrous PC beams are shown in Figs. 13 and 14, respectively.

Figure 13 shows that the PSFC I-Beams demonstrated superior shear performance when compared with the traditional PC I-Beam. Not only the shear strengths, but also the ductility and stiffness were greater in all the PSFC I-Beams in comparison with the PC I-Beam. In case of web-shear failure, the increase in shear strengths of PSFC I-Beams over the PC I-Beam due to addition of steel fibers ranged from 15 to 50 % corresponding to a fiber factor of 0.40–1.225, respectively. Hence, based on the limited work, the authors suggest an optum fiber factor of 0.40 for minimum shear strength and a fiber-factor of 1.225 for maximum shear strength based on workability and constructibility requirements in PC beams made with high strength concrete.

Figure 14 shows that the PSFC I-Beam also demonstrated superior flexure-shear performance when compared with the traditional PC I-Beams. Not only the flexure-shear strengths, but also the ductility was greater in all the PSFC I-Beams in comparison with the PC I-Beams. The increase in flexure-

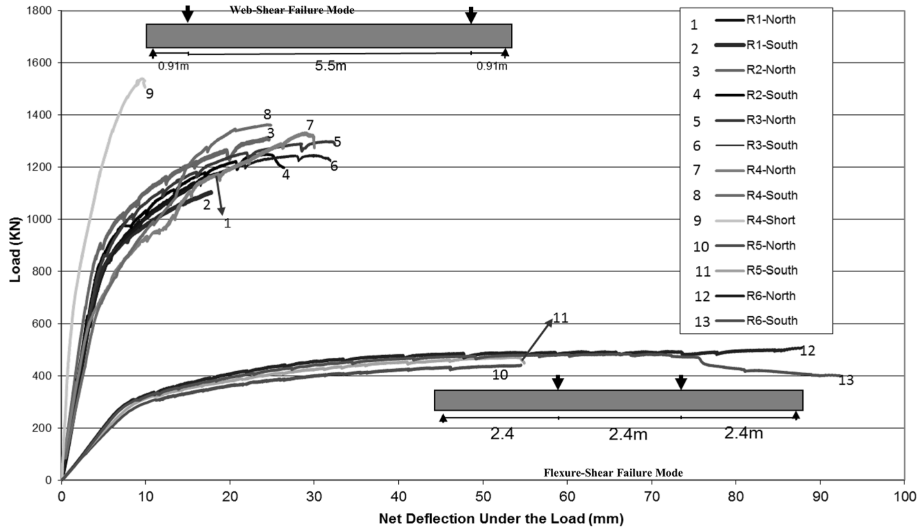


Fig. 11 Shear force versus net deflection curves for PSFC I-Beams. *Note* 1″ = 25.4 mm, 1 KN = 0.225 kip.

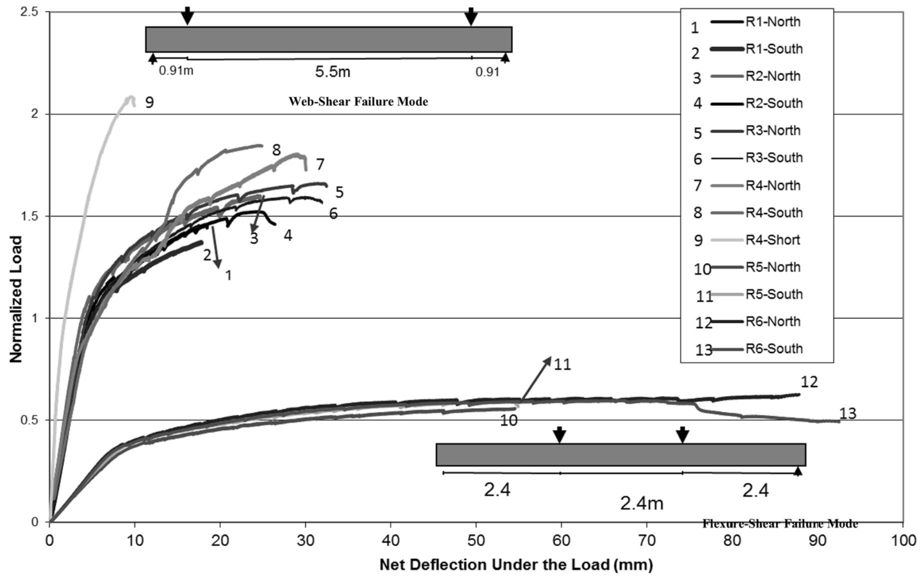


Fig. 12 Normalized shear force versus net deflection curves for PSFC I-Beams. *Note* 1″ = 25.4 mm, 1 KN = 0.225 kip.

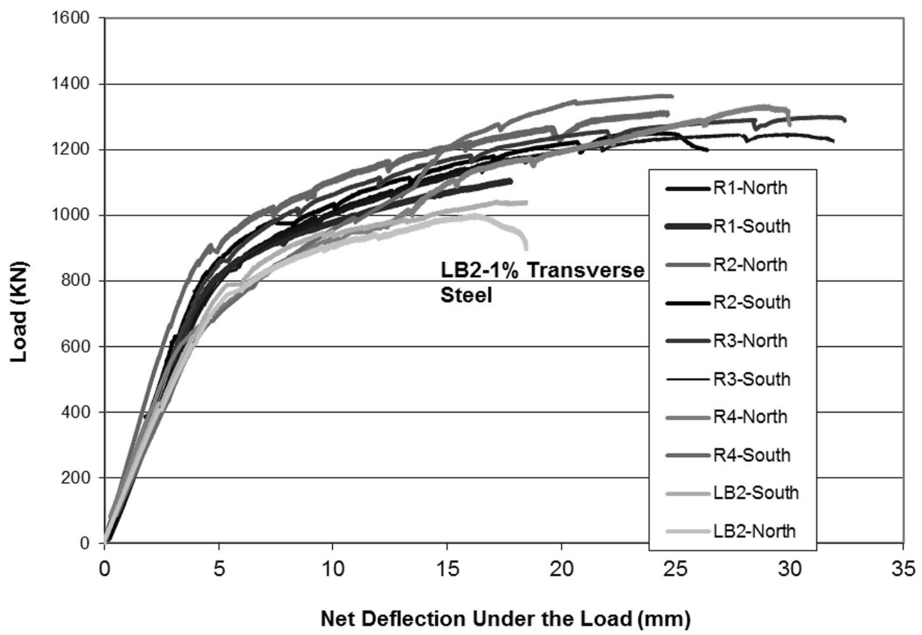


Fig. 13 Comparison of PSFC and PC I-Beams in web-shear failure mode. *Note* 1″ = 25.4 mm, 1 KN = 0.225 kip.

(Note: 1”=25.4mm, 1KN=0.225kip)

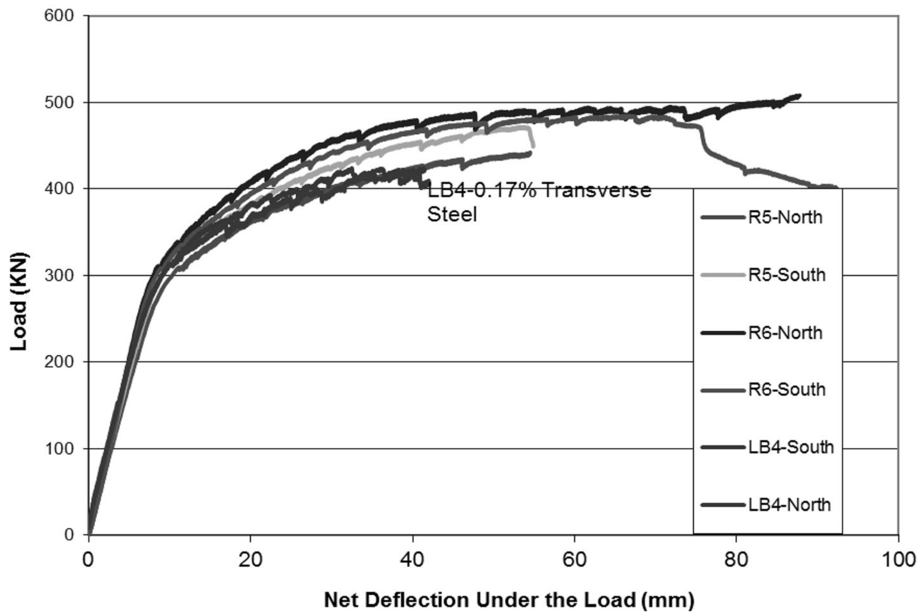


Fig. 14 Comparison of PSFC and PC I-Beams in flexure-shear failure mode. *Note* 1″ = 25.4 mm, 1 KN = 0.225 kip.

Table 6 Ultimate strains in specimens R1–R6 measured by LVDTs.

Beam ID -End side	Strains ($\times10^{-6}$)						
	ε_{V1}	ε_{V2}	ε_{H1}	ε_{H2}	ε_{H3}	ε_{D1}	ε_{D2}
R1-North E	−1980	−367	−1430	42.6	−2097	1890	−4000
W	−2300	−28	−981	97.2	−2052	1950	−2890
R1-South E	−1390	−1100	−821	234	−1296	−2660	1120
W	−1960	−1150	−807	109	−1256	−2260	1180
R2-North E	−1270	−655	−2920	140	−2388	2490	−5080
W	−2090	−484	−2510	−324	−2741	1870	−4920
R2-South E	−3320	−1190	−817	−169	−3066	−4680	2330
W	−3490	−2140	−867	−35.9	−2975	−4540	2360
R3-North E	−1290	−845	−1550	459	−1319	1870	−3360
W	−738	21	−1380	−694	−1797	1210	−3010
R3-South E	−2030	−444	−2120	275	−1504	−2910	875
W	−1900	20.9	−3500	116	−1684	−4890	2010
R4-South E	−2930	−1000	−1720	−109	−2900	3040	−5490
W	−1140	−1130	−3740	−850	−3326	6770	−7100
R5-North E	−3560	−673	−2900	771	−1590	−47.8	−3140
W	−3140	−1140	−3030	562	−1610	248	−3370
R5-South E	−578	−6110	−2660	712	−1260	−3700	560
W	−122	−5610	−2920	519	−1400	−2940	−42.5
R6-North E	144	−1210	−5020	687	−1208	−18.2	−10,600
W	−99.6	−436	−2610	874	−1192	428	−1870
R6-South E	−811	−6.62	−2395	572	−653	−1684	−9864
W	−183	−142	−1814	275	−527	−890	87

E East side, *W* West side.

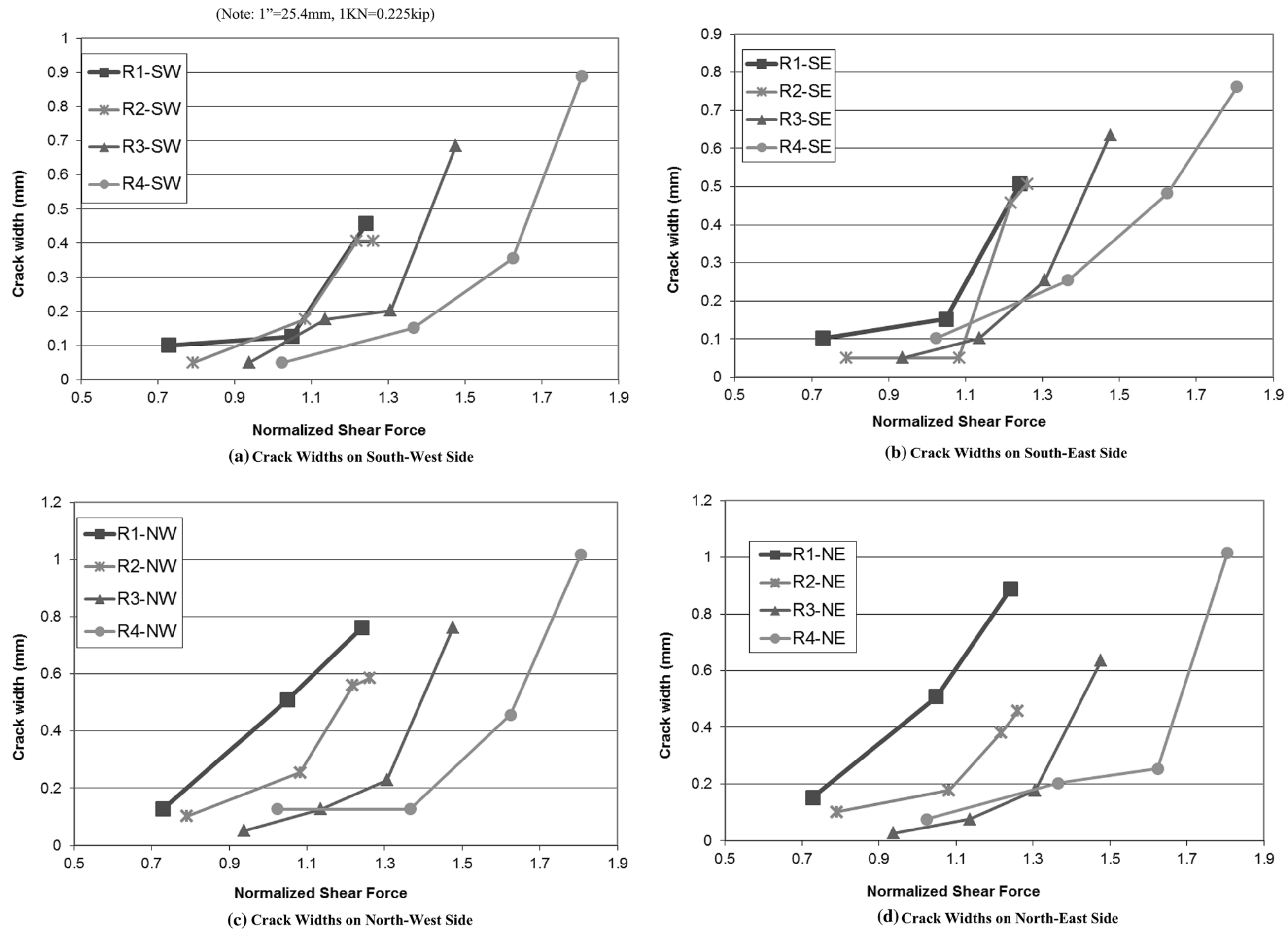


Fig. 15 Shear crack widths versus normalized shear force in beams R1–R4 (North). *Note* 1″ = 25.4 mm, 1 KN = 0.225 kip.

shear strengths of PSFC I-Beams over the PC I-Beam due to addition of steel fibers ranged from 15 to 24 % corresponding to a fiber factor of 0.40–1.225, respectively. It can be clearly observed from the Figs. 13 and 14 that the web-shear behavior is affected more than the flexure-shear behavior of PC beams owing to the addition of steel-fibers.

Table 6 gives the ultimate strains in Beams R1–R6 measured by LVDTs at failure. The LVDTs were located adjacent to the loading point as indicated in Fig. 8. A set of six LVDTs is shown in Fig. 9. Each set had two vertical, two horizontal and two diagonal LVDTs. Out of the two vertical LVDTs, the one that was situated closer to the load was named V2 (strain of ε_{V2}) while the other was named V1 (strain of ε_{V1}). The horizontal LVDT situated on the top flange was named H1 (strain of ε_{H1}) while the one on the bottom flange was named H2 (strain of ε_{H2}). The diagonal LVDT that was connected to the top flange near the load point was named D1 (strain of ε_{D1}) and was subjected to compressive strains during the loading of the beams. Diagonal LVDT D2 (strain of ε_{D2}) was connected to the lower flange near the load point and subjected to tensile stresses during loading of the beams. The tensile and compressive strains measured are more than what had been observed in conventional PC I-beam (Laskar 2009).

5. Shear Crack Widths and Crack Patterns

As mentioned earlier, shear cracks were continuously tracked and measured during the load tests of the beams. A grid was marked on the beam-web at both the beam-ends to facilitate easy identification and location of the shear cracks. Hand-held microscopes were utilized to precisely measure the shear crack width with an accuracy of 0.025 mm (0.001 in.). Figure 15a–d shows the plot of the normalized shear force and corresponding shear crack width in Beams R1–R4 (having web-shear mode of failure) measured on four different sides of the beams, during the test. The represented shear crack widths for a given beam were the maximum crack widths recorded along the most dominating shear crack in a beam during the test.

The onset of shear crack formation in all the beams initiated at the mid height of the beam web and was oriented along a line joining the loading and support points. Shear cracks of this nature are referred to as “diagonal tension cracks”, because the general direction of principal tension is perpendicular to this crack. The ligament of concert formed between adjacent diagonal tension cracks is referred to as a concrete compression strut. In the conventionally reinforced PC beams, the applied shear force is resisted by tension in transverse rebars and compression in the concrete strut

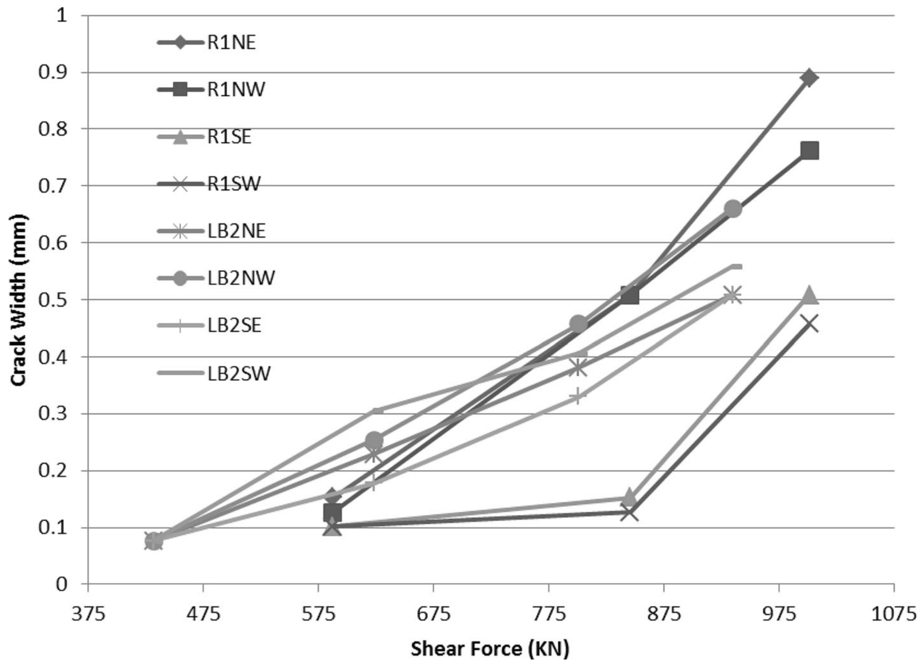


Fig. 16 Shear crack widths versus shear force in beams R1 and LB2. *Note* 1″ = 25.4 mm, 1 KN = 0.225 kip.

(Schlaich et al. 1987). In the case of PSFC girders, diagonal tension is resisted solely by the steel fibers. In the test beams, the initial diagonal tension crack did not generally progress to form the failure surface, but as the load increased, other cracks appeared and further developed into a failure surface with a single dominant failure shear crack (see Fig. 10).

Steel fibers were clearly observed to restrict the width of the shear cracks, as shown in Fig. 15. Generally, it was observed that as the fiber-factor increased the shear crack width for a given load decreased. Also, the load at which first visible shear crack appeared increased as the fiber-factor increased. This can be attributed to the fact that with the use of higher fiber-factor, more steel fibers are available in bridging and intersecting the shear crack. The stresses across the shear crack will therefore be shared by a larger number of steel fibers, thereby reducing the tensile strain across the crack. As the strains across the crack and in the steel fibers are reduced, the crack widths will be less.

To better understand the effectiveness of steel fibers in controlling the shear crack widths in PC beams, Fig. 16 is plotted depicting the crack widths of fibrous (Beam R1) and non-fibrous (Beam LB2) beams. It can be seen from Fig. 16 that the onset of shear cracking for beams with steel fibers occurred at a higher normalized shear force than those without steel fibers. This indicates that the addition of steel fibers in beams is helpful in preventing the development and growth of initial shear cracks. This property of steel fibers can be helpful particularly at service load level in PC highway-bridge beams.

The above discussion signifies that the replacement of traditional transverse rebars with steel fibers enhances the shear crack resistance in PC beams. The test results demonstrated that steel fibers more effectively delayed the opening of cracks beyond the service load level in the PSFC I-Beams in comparison with the traditionally reinforced PC beams.

6. Conclusions

The shear behavior of PSFC beams was critically examined by full-scale tests on six TxDOT Type-A I-beams with web-shear or flexural-shear failure modes. From the experimental results of six PSFC I-beams, it was observed that with increase in the amount of steel fibers (fiber factor) the shear capacity of the beam was increased.

The amount of steel fibers had an effect on first crack load and crack widths in shear span of the beam. As the amount of steel fibers increased, the first crack load increased and crack widths at a given load were decreased. The optimum fiber-factor is dependent on many parameters such as the desired minimum and maximum shear capacities, the required degree of workability to cast full-scale beams, compressive strength of concrete and mode of failure based on a/d-ratio. Based on the results of this limited study, the authors suggest an optum fiber factor of 0.40 for minimum shear strength and a fiber-factor of 1.225 for maximum shear strength based on workability and constructibility requirements in PC beams made with high strength concrete.

From the above observations it can be concluded that steel fibers were found very effective in resisting the shear loads and arresting the shear cracks. In order to replace shear reinforcement with steel fibers in PC beams, however, further research would be required on the serviceability and strength of PSFC members considering a reliable margin of safety.

Acknowledgments

This research was funded by the Texas Department of Transportation. The researchers would like to thank the Texas Concrete Company, Victoria, Texas, for continued cooperation during this project. The researchers are grateful to Bekaert Corporation (USA) for supplying the steel fibers for this research.

References

Abdul-Wahab, H. M. S., & Al-Kadhimi, S. G. (2000). Effect of SFRC on shear strength of prestressed concrete beams. *Magazine of Concrete Research, 52*(1), 43–51.

Cho, S. H., & Kim, Y. I. (2003). Effects of steel fibers on short beams loaded in shear. *ACI Structural Journal, 100*(6), 765–774.

Dhonde, H. (2006). Steel fibers and self-consolidating concrete in prestressed concrete beams. PhD. Dissertation, Department of Civil and Environmental Engineering, University of Houston, TX.

Junior, S. F., & De Hanai, J. B. (1999). Prestressed fiber reinforced concrete beams with reduced ratios of shear reinforcement. *Cement and Concrete Composites, 21*(3), 213–221.

Kani, G. N. J. (1964). Riddle of shear failure and its solution. *American Concrete Institute Journal, 61*(4), 441–467.

Langsford, R. P., Lloyd, N., & Sarker, P. K. (2007). Shear strength of steel fibre reinforced prestressed concrete beam. In *Proceedings of the 4th International Structural Engineering and Construction Conference (ISEC-4)—Innovations in Structural Engineering and Construction* (pp. 441–446). Melbourne, Australia: Taylor & Francis/Balkema.

Laskar, A. (2009). Shear behavior and design of prestressed concrete members. PhD Dissertation, Department of Civil and Environmental Engineering, University of Houston, TX.

Meda, A., Minelli, F., Plizzari, G. A., & Riva, P. (2005). Shear behaviour of steel fibre reinforced concrete beams. *Materials and Structures, 38*(3), 343–351.

Narayanan, R., & Darwish, I. Y. S. (1987). Shear in prestressed concrete beams containing steel fibers. *The International Journal of Cement Composites and Lightweight Concrete, 9*(2), 81–90.

Padmarajaiah, S. K., & Ramaswamy, A. (2001). Behavior of fiber-reinforced prestressed and reinforced high-strength concrete beams subjected to shear. *ACI Structural Journal, 98*(5), 752–761.

Schlaich, J., Schafer, K., & Jennewein, M. (1987). Toward a consistent design of structural concrete. *PCI Journal, 32*(3), 74–150.

Tadepalli, P. R., Mo, Y. L., & Hsu, T. C. (2013). Mechanical properties of steel fibre concrete. *Magazine of Concrete Research, 65*(8), 462–474.

Tan, K. H., Paramasivam, P., & Murugappan, K. (1995). Steel fibers as shear reinforcement in partially prestressed beams. *ACI Structural Journal, 92*(6), 643–652.

Thomas, J., & Ramaswamy, A. (2006). Shear strength of prestressed concrete T-beams with steel fibers over partial/full depth. *ACI Structural Journal, 103*(3), 427–435.