Research Article

Investigating Flexural Behaviour of Prestressed Concrete Girders Cast by Fibre-Reinforced Concrete

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The main objective of this research was to investigate the effect of adding polypropylene and steel fibres on flexural behaviour of prestressed concrete girders. Although the construction industry is frequently using prestressed concrete to increase the load-carrying capacity of structures, it can be further enhanced by using fibres. In this paper, experimental work was carried out to encourage the construction industry in utilizing fibres in prestressed concrete members to improve the mechanical properties of these members. As past investigations on fibre-reinforced prestressed beams were limited, the present work was done on small-scale fibre-reinforced I-shaped prestressed concrete girders. Six small-scale prestressed concrete girders were cast comprising a control girder, a hybrid girder, two girders with varying percentages of steel fibres, and two girders with varying percentages of polypropylene fibres. These girders were tested by centre point loading up to failure. It was concluded that, by the addition of small volume fraction of fibres, not only the ductility but also the tensile strength and flexural strength of FRC girders could be improved. It also altered the failure pattern positively by enhancing large strains in concrete and steel. Steel fibre-reinforced concrete showed higher energy absorption and deflection at ultimate loads in comparison to other specimens.

1. Introduction

Steel reinforcement is used in the concrete members in various forms; for example, steel bars in conventional reinforced concrete (RC), steel wires in prestressed concrete (PC), and steel fibres as a reinforcing material in steel fibre-reinforced concrete (SFRC). Baston et al. [1] investigated the use of steel fibres and found it effective against shear failure instead of shear reinforcement. Then, various researches were carried out showing that plain concrete's ductility could be significantly increased by using steel fibre-reinforced concrete. ACI 318-11 included a specific provision that permits steel fibres as an alternate of shear reinforcement, if at least 0.75% of volume fraction was used [2]. This provision was considered not only for prestressed concrete but also for reinforced concrete members. Steel fibres helped in the significant improvement of concrete's postcracking tensile resistance and toughness [3]. Hooked steel fibres showed better performance of flexural toughness

and ultimate flexural strength when they were compared with the straight fibres [4]. As fibre content was increased in concrete, the concrete strength and density of laterized fibre-reinforced concrete were also enhanced. In laterized concrete, 1.5% fibres by volume was considered as the optimum fibre content [5]. The strength of the hybrid fibre-reinforced concrete was greater than that of control mix [6].

Flexural strength is the capacity of a flexural member, i.e., slabs or beams to resist the failure in bending. Ziad et al. [7] investigated the flexural behaviour of polypropylene fibre-reinforced concrete (PPFRC) by varying the length and volume fraction of polypropylene fibres and characterized the postpeak load resistance of the material in flexure under the four-point loading test. They found that, for enhancing the postpeak resistance, 19 mm or 3/4 in long fibres were more favourable for volumes less than or equal to 0.3 percent. Twelve millimetres-long fibres were more effective for 0.5 percent volume. Alhozaimy et al. [8] found out the effects of different volume fractions of polypropylene fibres

on the toughness and flexural strength of concrete fibre composites using two-point loading test designed by ASTM C78-10. He concluded that fibres had no effect on the flexural strength at lower volumetric fractions of FRC. The toughness in flexure was appreciably affected by the polypropylene fibres.

Patil and Sangle [9] worked on the prestressed and non-prestressed concrete beams using monofibres and concluded that load-carrying capability of steel fibre-reinforced concrete (SFRC) beam was greater than that of plain concrete beam by approximately 30–50%. Steel fibres helped to increase crack resistance both in flexure and shear. Hussain et al. [10] worked on the flexural response of carbon fibre-reinforced prestressed concrete beams and concluded that, for steel prestressed beams, the increase in the concrete strength slightly decreased the deformation capacity and improved the ultimate load capacity.

Yoon et al. [11] worked on the behaviour of pretensioned concrete beams using steel fibre-reinforced concrete and concluded that the presence of fibres affected the flexural cracking load of steel fibre-reinforced concrete (SFRC). The maximum flexural strength of the SFRC beams was at the most 10.0% greater than that of normal concrete. Anthony et al. [12] worked on optimal polypropylene fibre content for enhanced flexural and compressive strength of concrete. They concluded that the flexural strength of concrete increased by as much as 65% when low percentage fractions (0.25%) of fibres were added. It recorded a slight increase of 0.5% in concrete strength and then a decrease when 0.75% and 1% of fibres were added in concrete matrix. From this study, it was observed that the optimum dosage of polypropylene fibre was between 0.25% and 0.5% for both compressive and flexural strengths. Kang et al. [13] studied the shear-flexure coupling behaviour of steel fibre-reinforced concrete (SFRC) beams and validated the test results by using nonlinear modeling techniques considering the shear-flexure coupling effects. Soroushian and Bayasi [14] carried out compressive strength experiments and concluded that crimped and hooked steel fibres enhanced the postpeak behaviour and compressive strength more effectively when compared to straight steel fibres. Hajrah et al. [15] studied shear behaviour on fibre-reinforced I-section pretensioned concrete beams. They investigated shear properties by application of cyclic loading and concluded that use of fibres improved load-carrying capacity and ductility. The role of fibres initiated after cracks, and they actively increased the tensile properties of specimens. Hwang et al. [16] worked on shear deformation of prestressed concrete beams cast by varying percentage of steel fibres by volume. They concluded that using steel fibres, shear cracking, and shear strength of prestressing beams was considerably increased. Kang et al. [17] found increased flexural ductility in SFRC beams. The authors' findings were based on test results of twelve reinforced concrete and steel fibre-reinforced concrete beams subjected to four-point loading. Kwak et al. [18] studied the behaviour of SFRC beams without shear reinforcement. The test results showed that nominal stress at shear cracking and the ultimate shear strength increased with increasing fibre volume.

Additionally, the researchers observed that the inclusion of steel fibres changed the failure mode from shear to flexure.

There was limited research found on flexural behaviour of prestressed concrete beams using monofibres and hybridized fibres. Furthermore, most of the investigations discussed earlier were conducted, focusing on shear behaviour only. In this research, an effort was made to fill up those gaps by testing fibre-reinforced concrete girders. This experimental program was likely to give some useful information regarding different mechanisms of failures and behaviour of fibre-reinforced prestressed concrete girders when tested as flexural members. Moreover, other properties of FRC such as compressive strength, tensile strength, load vs deflection responses, stress vs strain response, flexural strength, cracking patterns, ductility index, and energy absorption were evaluated.

2. Materials and Methods

2.1. Materials. Ordinary Portland cement (ASTM C-150-14, Type-I) was used to cast prestressed concrete girders. The source of fine aggregates used in this experimental program was locally available Lawerencepur brand sand with fineness modulus of 2.12. The source of coarse aggregates was Margallah hills near Islamabad, the capital city of Pakistan. Two types of coarse aggregates were used with two different percentages in preparing aggregate blend, conforming to ASTM C-33 [19]. Aggregates passing from 20.0 mm sieve were 80%, while aggregates passing from 9.50 mm sieve were 20% in the blend. The main ingredients of concrete-like cement, sand, and aggregates were mixed in the ratio of 1 : 1 : 2. The absolute quantities of all the materials in a one-meter cube of concrete are shown in Table 1. Compressive strength of mix at the age of 28 days determined as per ASTM C39-10 [20] was 35 MPa.

All girders were pretensioned by using 7 wired strands of 9.5 mm diameter at bottom and 5 mm wires at top, as shown in Figure 1. For shear linkages, 5 mm steel wire was used to provide shear reinforcement as given in detailing of girders (Figure 1). Ordinary tap water was used in preparing concrete mixes for girders. Polypropylene fibres (PPF) were used with two different volume percentages in PPFRC. Hooked-end steel fibres were also used having rounded ends with two different percentages in SFRC. The hybridization of fibres was also practiced in this research. For increasing strength and workability of fresh fibre-reinforced concrete, a superplasticizer was used by 1.2% of the weight of cement as per ASTM C 494-14. Superplasticizer had capability to reduce permeability and improve workability, ultimate strength, and durability of concrete.

2.2. Casting of Specimens. Casting of I-shaped girders was done in the casting yard of Bannu Mukhtar Pvt. Limited located at Hattar 14 km away from the place of research. The size of each girder was similar to some previous researches [18, 21, 22] and was adjusted to meet with the lab requirements and casting yard capabilities. The concrete mix ratio in all specimens was kept same as achieved in the

TABLE 1: Absolute quantities in a one-meter cube of concrete.

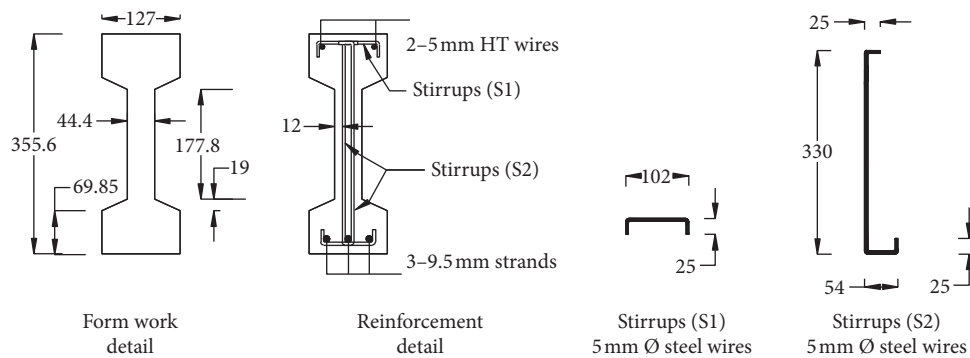| Sr. No.	Specimen	Cement (kg/m³)	Water content (Liters)	Sand (kg/m³)	Coarse aggregates (20 mm passing) (kg/m³)	Coarse aggregates (9.5 mm passing) (kg/m³)	Admixture (superplasticizer) (kg/m³)	Steel fibres (kg/m³)	Polypropylene fibres (kg/m³)
1	B1-CM							0	0
2	B2-HyFRC							58.50	4.55
3	B3-PPFRC-1							0	3.64
4	B4-PPFRC-2	520.78	223.93	520.78	729.09	312.47	6.25	0	5.46
5	B5-SFRC-1							50.70	0
6	B6-SFRC-2							66.30	0

FIGURE 1: Cross sections showing dimensions and reinforcement details of a typical I-shaped girder.

concrete mix design for 28 days compressive strength, i.e., 35 MPa. The complete length of the girders bed was 95 meters with separators to cut the girders in desired length. The compaction of concrete was done by using external vibrator. Formwork was removed after 24 hours, and curing activity was performed for 28 days by using wet burlap covering. Young's Modulus values for HT, ST wires, and strands are 200, 200, and 195 GPa respectively.

The pretensioning was done for full length of strands and later on shuttering plates were fixed. During pretensioning, the bars were stretched before casting of specimens. Long-line method was adopted for pretensioning of girders. In this method, prestressing was done for full length of prestressing bed before casting and then desired length of girder was cut off. 70 kN and 27 kN forces were applied on 9.5 mm strands and 5 mm high strength tensile wires (HT wire), respectively. Shear linkages consisting of 5 mm steel wire were used to provide enough shear strength to girders. These linkages were closely spaced near supports, and spacing was gradually increased towards centre of girder as shear capacity is always maximum near supports. Detailed properties of prestressing wires and effective prestressing force are given in Table 2. Cross sections of I-shaped girder with details of prestressing force and shear reinforcement are shown in Figures 1 and 2, respectively.

Two different types of fibres in varying ratios were used in all girders, as shown in Table 3. The addition of fibres in concrete usually results in severe problem of fibre flocculation in the concrete matrix causing balling action. Special mixing techniques were employed to overcome the balling action of steel fibres or flocculation of polypropylene fibres. In this context, steel fibres were mixed in coarse aggregates and polypropylene fibres were mixed in fine aggregates, both in dry state. Mixing was done manually by gloved hands for each batch of concrete for weighted amounts of all ingredients tabulated in Table 1.

2.3. Flexural Testing of Specimens. The test method covers the determination of flexural strength of prestressed concrete girders with centre-point loading. Grids were drawn on vertical faces of girders to examine crack patterns (Figure 3). Two deflection gauges were installed at the bottom of each girder to measure deflection at the midspan of girders. Average of the two deflection gauge values was taken. Three strain gauges were installed at the points of maximum possibility of cracks, i.e., near centre, below neutral axis of the girders. Point load was applied at the midspan of girders, and maximum deflection of the girder was measured using deflection gauges, schematically as shown in Figure 4. The girder was loaded through a 25 × 25 mm square steel rod to simulate point load. Experimental strains were recorded by using the concrete surface strain gauges connected to the P3-strain indicator and recorder box. Actual installation of dial gauges and strain gauges is shown in Figure 5.

TABLE 2: Properties of prestresssing wires and detail of prestressing force.

| Sr. No.	Nominal diameter	Exact diameter (mm)	Weight (kg/m)	Area (mm²)	Yield strength (MPa)	Ultimate strength (MPa)	Elongation (%)	Effective prestress (P_e) kN (% of ultimate strength)
1	9.5 mm	8.407	0.436	55.484	0	1906	3.13	70 (66)
2	strands	8.382	0.433	55.471	0	1903	4.69	
3	5 mm HT wire	5.004	0.155	20	1569	1772	3.19	27 (76)
4		5.004	0.155	20	1569	1769	3.15	
5	5 mm ST wire	5.791	0.207	26.451	470	584	17.19	—
6		5.791	0.207	26.451	452	553	12.5	—
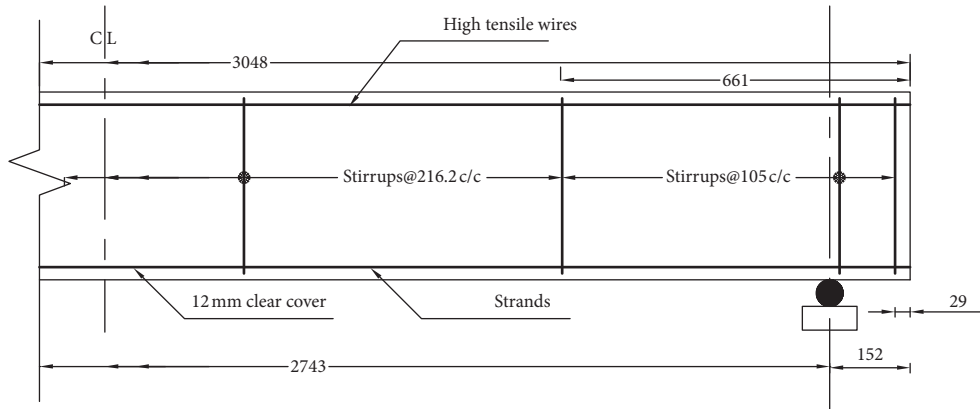
FIGURE 2: Detail of shear reinforcement in full length of an I-shaped girder.

TABLE 3: Fibre contents in all six concrete mixes.

Sr. No.	Name of the specimen	Fibre contents (by percentage of volume of concrete)		Fibre contents (by percentage of weight of concrete)	
		Polypropylene fibres	Steel fibres	Polypropylene fibres	Steel fibres
1	B1-CM	—	—	—	—
2	B2-HyFRC	0.50%	0.75%	0.20	2.52
3	B3-PPFRC-1	0.40%	—	0.16	—
4	B4-PPFRC-2	0.60%	—	0.24	—
5	B5-SFRC-1	—	0.65%	—	2.18
6	B6-SFRC-2	—	0.85%	—	2.86

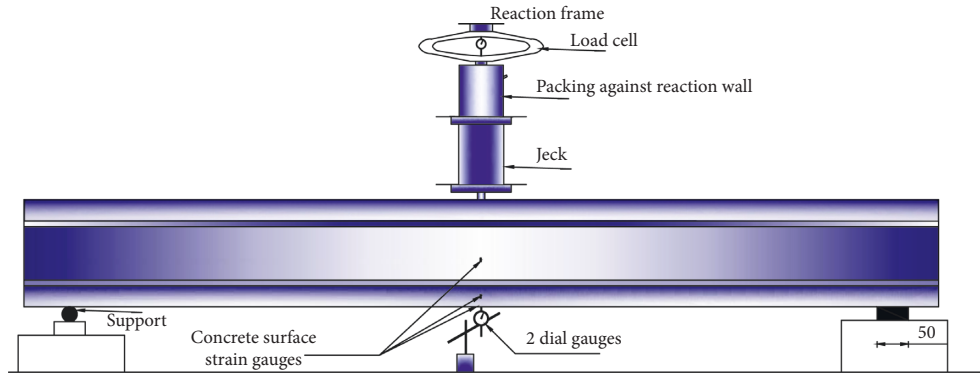FIGURE 3: Actual test setup using centre point loading.

FIGURE 4: Schematic view of arrangement for the flexural test of girders by the centre point loading method (ASTM C293-02).

FIGURE 5: Installation of strain and deflection gauges (arrows indicate the position of strain gauges).

3. Results and Discussion

3.1. Testing of Standard Samples

3.1.1. Compressive Strength of Concrete Mixes. As per ASTM C39-10 [20], nine concrete cylinders (three for each curing age, i.e., 3, 7, and 28 days) were cast for each type of concrete mix to determine the compressive strength at specified ages. Dimensions of each cylinder were 150 mm diameter and 300 mm height. Average compressive strength values of three cylinders at different ages are shown in Figure 6.

Maximum compressive strength was shown for the mix with hybrid fibres (B2-HyFRC). The mix of B2-HyFRC increased its compressive strength by 18% in comparison to the control mix (B1-CM). However, there was not much difference in compressive strength of mixes with monofibres (i.e., mixes of B3-PPFRC-1, B4-PPFRC-2, B5-SFRC-1, and B6-SFRC-2) as compared to that of control mix.

3.1.2. Split Tensile Strength of Concrete Mixes. Split tensile strength test of hardened concrete cylinders was performed as per ASTM C496-12 [23] for each curing age, i.e., 3, 7, and 28 days. The average split tensile strength values are shown in Figure 7.

It is known from the latest research that the presence of fibres considerably increased tensile properties of concrete in comparison to normal concrete (ACI 544.1-R-96) [24]. Maximum tensile strength was found for the mix of girder B2-HyFRC because this girder had hybrid fibres which increased the compatibility of two different types of fibres
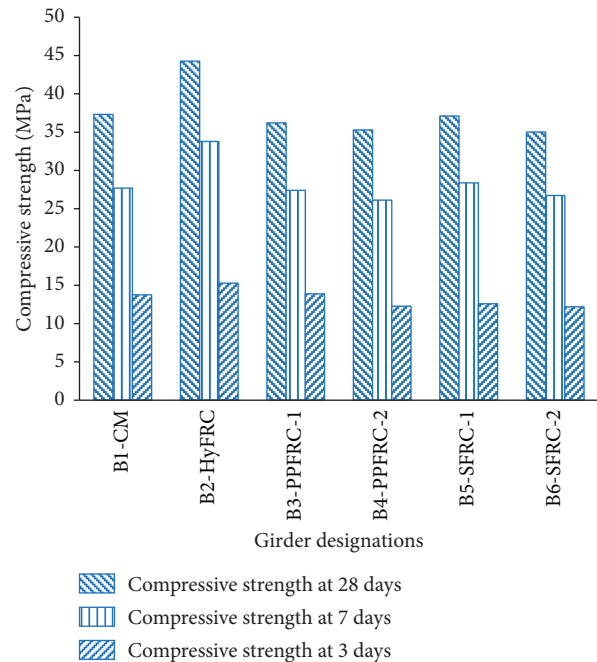

FIGURE 6: Average compressive strength at different curing ages.

within the concrete matrix. About 123% tensile strength was increased for mix of girder B2-HyFRC in comparison to the control mix. This is because of brittleness of normal concrete and ductile nature of fibres that bound the particles of concrete resulting in high strength concrete. Use of polypropylene fibres (B3-PPFRC-1 and B4-PPFRC-2) increased the tensile strength of concrete, but this increase in strength was quite low in comparison to that in case of specimen B2-HyFRC. The use of PP fibres in B3-PPFRC-1 increased the tensile strength up to 65% as compared to that of control mix (B1-CM).

Steel fibres bind concrete matrix firmly and increase its ductility considerably due to tensile nature of steel fibres. The roughness of steel fibres also plays an important role to bind the concrete matrix together. So, tensile strength of B5-SFRC-1 having steel fibre contents as 0.65% by volume of concrete was increased upto 90% in comparison to that of control mix (B1-CM). But the mix of B6-SFRC-2 having fibre volume as 0.85% showed less tensile strength in
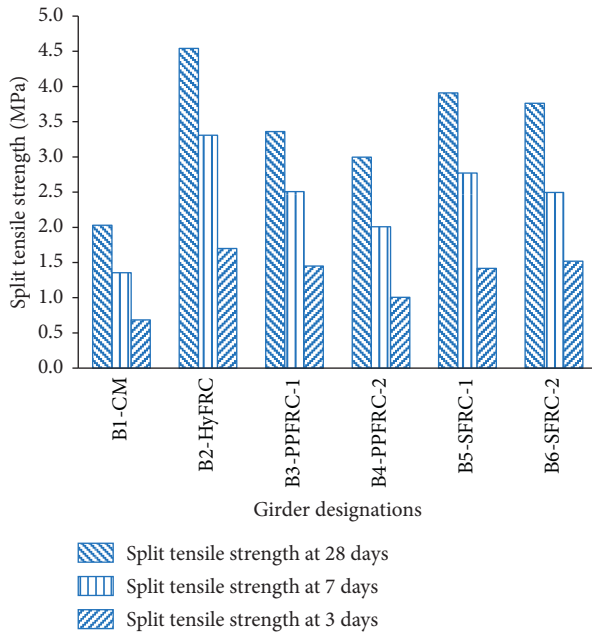
FIGURE 7: Average split tensile strength of all mixes at different curing ages.

comparison to mix of B5-SFRC-1, although it exhibited 85% more tensile strength than that of control mix (B1-CM).

3.2. Flexural Testing of Girder Specimens

3.2.1. First Crack Load. During application of load, continuous appearance of cracks was observed on the girders. The load at the first crack of all the girders was noted and then is plotted in Figure 8.

Although deflection of B2-HyFRC was more in comparison to other girders, the load at first crack was quite higher in B5-SFRC-1 with 0.65 percent of steel fibres. First crack loading capacity of the specimens using FRC increased from 0 to 1% at various volumes of polypropylene fibres in comparison to that of control girder. However, in case of steel fibres, the increase in first crack load was about 38% to 42%.

3.2.2. Ultimate Load. The stage at which the specimen does not have enough capacity to carry further load is known as ultimate loading capacity. At this stage, deflections are increased considerably but the load-carrying capacity is totally reduced. Ultimate load-carrying capacity for each type of girder was noted and is plotted in Figure 9.

The specimen B5-SFRC-1 having steel fibre content of 0.65% by volume of concrete accommodated maximum failure load among all specimens. The specimen showed 48% higher ultimate load as compared to control mix (B1-CM). This behaviour may be attributed to better flexural capacity of steel fibres in comparison to that of polypropylene fibres. This increase in flexural capacity of the specimen was due to intrusion and binding of concrete matrix by steel fibres and acted as barrier to resist the crack
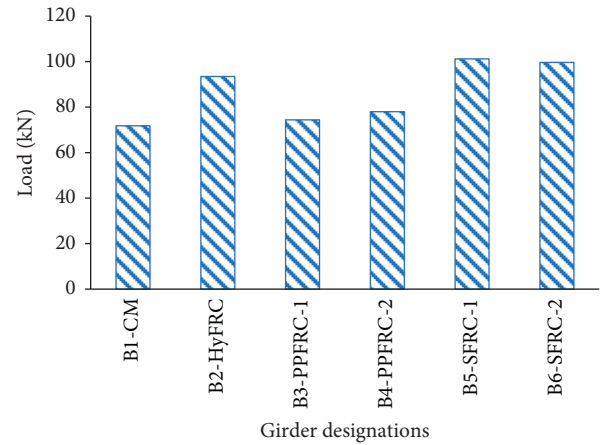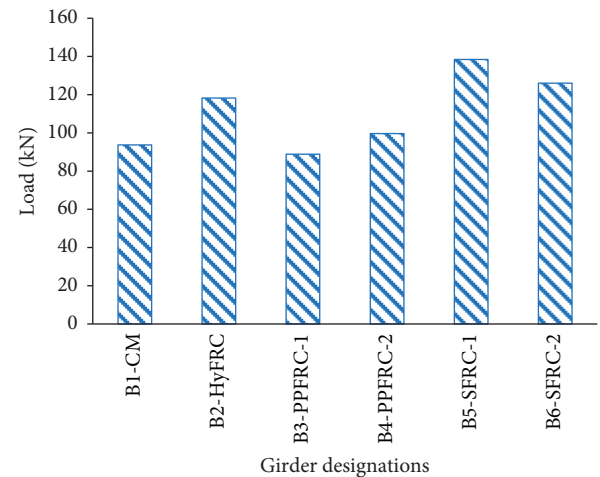

FIGURE 8: Load at the first crack.

FIGURE 9: Ultimate loads at failure.

propagation in flexure. The specimen B3-PPFRC-1 having polypropylene fibre content of 0.4% showed premature failure than that of control specimen but by increase in polypropylene fibre content to 0.60% as in the case of girder B4-PPFRC-2, the ultimate loading capacity was slightly increased about 6.4% in comparison to that of control specimen, i.e., B1-CM.

3.3. Load-Deflection Curves.

The load was applied at an interval of 0.5 mm deflection. To measure deflection at midspan of girders, two dial gauges were installed and average value of deflection of these two gauges was noted. Load was noted using load cell positioned above girders. Load-deflection curves of all specimens are plotted in Figure 10.

It was observed that load-carrying capacity of B2-HyFRC girder was quiet higher at ultimate load stage. At initial load stages, all curves are almost overlapping, but after elastic range, the load-carrying capacity was changed abruptly. This was because the fibre-reinforced concrete improved the postelastic properties of structural members.
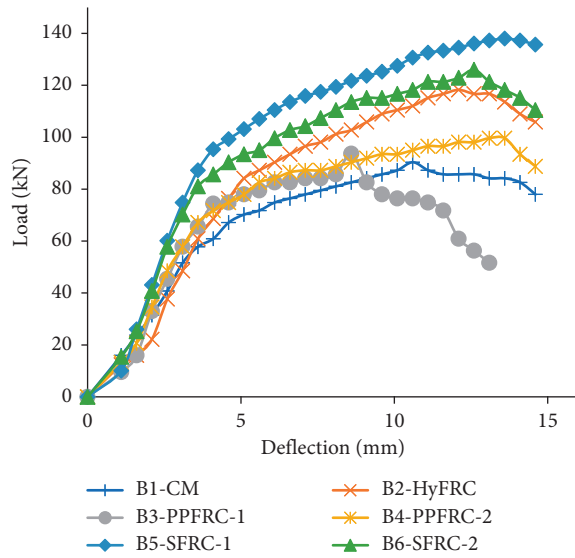
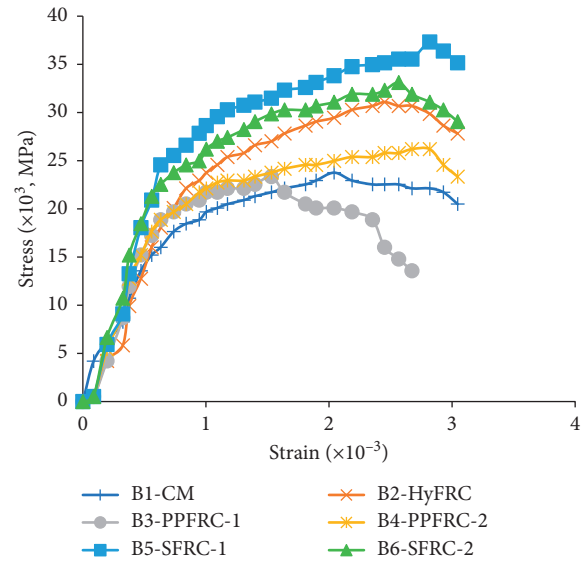FIGURE 10: Load-deflection curves of all girders.

FIGURE 11: Stress-strain curves of all girders.

The curve for B3-PPFRC-1 had abrupt failure at ultimate load. Although B3-PPFRC-1 showed an increase in the initial load stage, it reached at ultimate load stage prior to that of control girder because of low polypropylene fibre content. The deflection of control girder at ultimate load was 24% higher than that of B3-PPFRC-1. Load-carrying capacity of B4-PPFRC-2 was 8.6% higher than that of control mix (B1-CM). This improvement is due to the increased percentage of polypropylene fibres, i.e., from 0.40% to 0.60% by volume of concrete, and their resulting uniform distribution in concrete matrix. It was also observed that B4-PPFRC-2 showed improved postelastic behaviour. The girder exhibited elastic behaviour for longer time as compared to B3-PPFRC-1. The higher percentage of fibre content resulted in the uniformity of concrete matrix. B5-SFRC-1 showed enhanced load-carrying capacity and failed in more ductile manner in comparison to control girder. Deflection at ultimate failure of B5-SFRC-1 was increased by about 30% in comparison to that of control specimen. B5-SFRC-1 showed maximum deflection and load-carrying capacity among all tested specimens. The girder B6-SFRC-2 showed almost similar behaviour to that of B5-SFRC-1, but less load-carrying capacity and deflection at ultimate failure. The girder B6-SFRC-2 showed 34.5% higher loads than that of control mix. This was because the fibre content increased the tensile properties of girders up to some extent beyond which there was a reduction in strength due to flocculation of fibres. Deflection at ultimate failure was increased by about 24% in comparison to that of control girder because of improved tensile strength of steel fibre-reinforced girder.

3.4. Stress-Strain Curves.
Three strain gauges were installed near the centre of girders below neutral axis to examine the stress-strain behaviour of girders. Average of these three gauges was taken to plot strain at horizontal axis and stresses on vertical axis. Stress-strain curves of girders in comparison to the control girder are shown in Figure 11. Curves shown in Figure 11 are almost similar in shape to the load-deflection curves plotted in Figure 10, as load is proportional to stress and deflection is proportional to strain.

3.5. Ductility Ratio.
Ductility index of flexural members is defined by Azizinamini et al. [25] as the ratio of maximum displacement (Δ_{max}) at midspan to first yield displacement (Δ_y) of member. The first yield displacement comes out by the intersection of tangents to load displacement curve at origin and maximum displacement. First yield displacements of all girders are plotted in Figures 12(a)–12(f). Ductility index of each girder is calculated in Table 4 and plotted in Figure 13.

Ductility index of B1-CM was the lowest in comparison to that of other girders as this girder showed brittle behaviour and failed just within the deflection of a few millimetres after elastic limit. Maximum ductility index was observed in B6-SFRC-2 as this girder showed more ductile behaviour in comparison to those of other specimens. Ductility index for this specimen was 46.7% higher than for control mix. The load-deflection curve of this girder was smooth and failed after considerable deflection in comparison to the other girders. PPFRC girders showed slight increase in ductility index, i.e., 1% to 8% for B3-PPFRC-1 and B4-PPFRC-2, respectively. The tensile nature of steel fibres ultimately improved its postelastic behaviour.

3.6. Energy Absorption.
Energy absorption is the indication of toughness of flexural members calculated from the load-deflection curves as the area under those curves. Absorbed energy depends upon both the maximum load and deflection achieved by the specimen at ultimate failure state. The maximum energy was absorbed by B5-SFRC-I, i.e., in comparison to other specimens. In comparison to the control specimen, B5-SFRC-1 absorbed about 69% more

$y = 14.996x - 0.2467$

$y = 3.1x + 55.955$

(a)

$y = 20.763x - 9.5464$

$y = 3.0988x + 81.377$

(b)

$y = 22.693x - 9.7308$

$y = 3.0988x + 60.77$

(c)

$y = 3.0988x + 62.474$

$y = 15.541x - 0.3837$

(d)

$y = 22.058x + 0.6686$

$y = 1.9629x + 111.41$

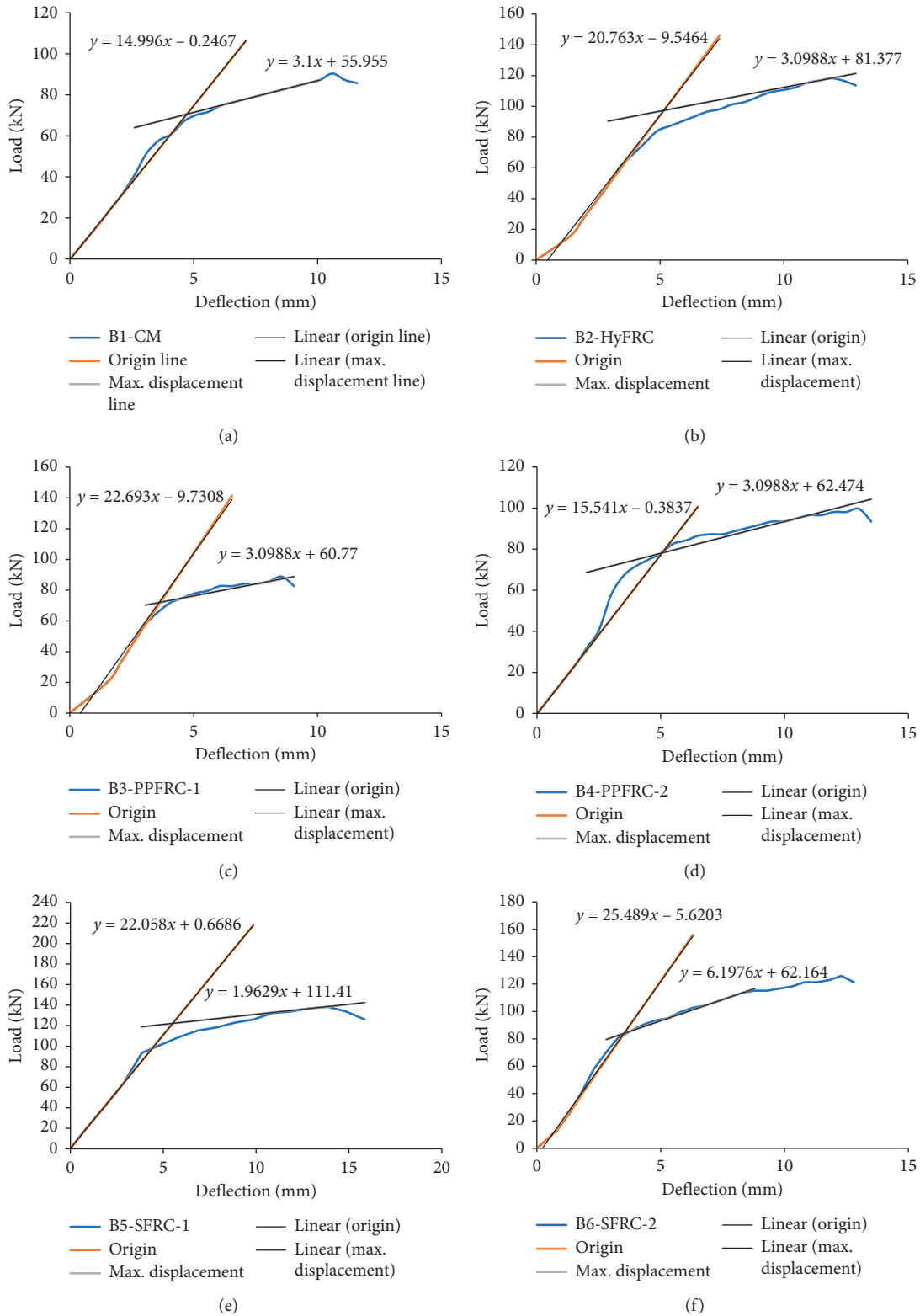(e)

$y = 25.489x - 5.6203$

$y = 6.1976x + 62.164$

(f)

FIGURE 12: First yield displacements of all girders.

energy. Although load-carrying capacity of B3-PPFRC-1 was higher than that of B1-CM, it showed 16.67% less energy absorption due to low deflection at ultimate loads (Figure 14).

3.7. Crack Patterns. The use of fibre-reinforced concrete increases crack resistance and improves the ductility of members. On the contrary, an overdose of the fibre content causes negative effects on the efficiency of the members due

TABLE 4: Ductility index values of all girders.

Sr. No.	Specimen	Maximum deflection (Δ_{max})	First yield displacement (Δ_y)	Ductility index (Δ_{max}/Δ_y)
1	B1-CM	11.6	4.68	2.47
2	B2-HyFRC	12.9	5.147	2.51
3	B3-PPFRC-1	9.05	3.598	2.51
4	B4-PPFRC-2	13.5	4.54	2.97
5	B5-SFRC-1	15.85	6.12	2.59
6	B6-SFRC-2	12.8	3.515	3.64
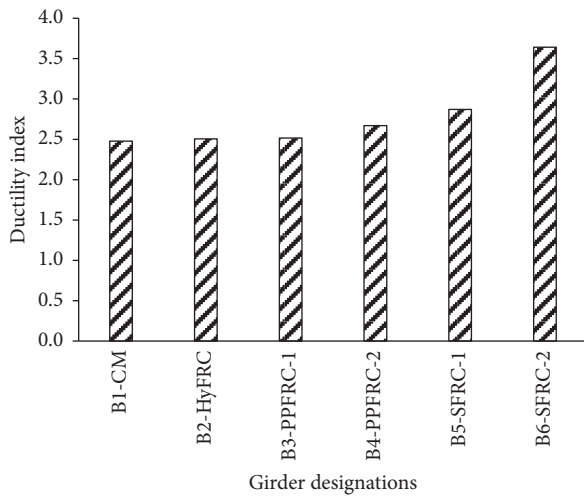
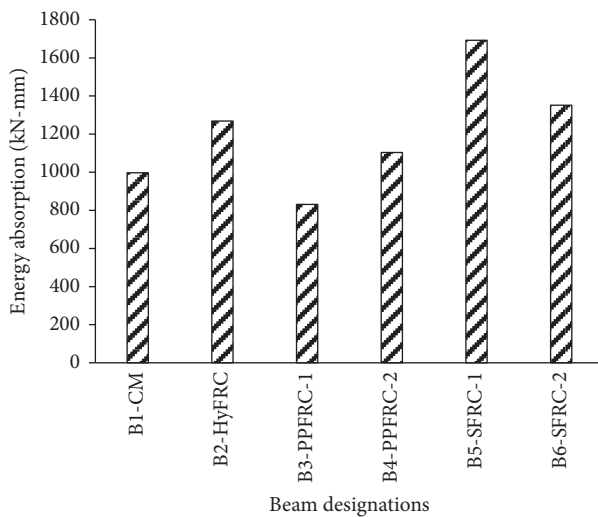FIGURE 13: Ductility index of all girders.

FIGURE 14: Energy absorption by all girders.

to flocculation of fibres. The crack patterns along with the failure modes observed during testing of girders are explained separately as follows.

3.7.1. Crack Patterns of B1-CM. Crack patterns in B1-CM are shown in Figure 15. Although overall crack pattern in control specimen was flexural in nature, near the supports on one side of the girder, these appeared as shear cracks and propagated towards top flange of the girder. Before failure of

FIGURE 15: Crack patterns of B1-CM.

the girder, a reduction in loading capacity was noted, resulting in widening of cracks.

3.7.2. Crack Patterns of B2-HyFRC. The girder B2-HyFRC with hybrid fibres showed typical flexural cracks, and there was congestion of cracks at the midspan of girder due to application of point load at specific point as shown in Figure 16.

The congestion of cracks near midspan verified the efficient nature of fibre-reinforced concrete that binds the concrete matrix together, resulting in enhanced resistance against cracks. The cracks were observed after each 0.5 mm deflection, and it was noted that initiation of cracks occurred near the midspan of bottom flange and penetrated to the web of girder.

3.7.3. Crack Patterns of B3-PPFRC-1. Crack patterns of B3-PPFRC-1 are shown in Figures 17(a) and 17(b). This girder had polypropylene fibres at 0.40% by volume of concrete.

It showed brittle failure and number of cracks were greater than those in B2-HyFRC but less than those in the control specimen. Although it failed in flexure, some cracks initiated from the supports and moved towards point of application of the load. These showed hybrid nature of girder failure called dual failure, i.e., partly in flexure and partly in shear.

3.7.4. Crack Patterns of B4-PPFRC-2. As this girder had higher ratio of polypropylene fibres as compared to B3-PPFRC-1, it showed enhanced capacity to resist the cracks during loading. Cracks initiated from the web of girder and continued gradually towards the lower flange due to development of stresses at the bottom of specimen. The cracks of this pattern showed typical flexural behaviour of the

FIGURE 16: Crack patterns of B2-HyFRC.

(a)

(b)

FIGURE 17: Crack patterns of B3-PPFRC-1.

FIGURE 18: Crack patterns of B4-PPFRC-2.

FIGURE 19: Crack patterns of B5-SFRC-1.

FIGURE 20: Crack patterns of B6-SFRC-2.

member upto ultimate failure. Crack patterns of B4-PPFRC-2 are shown in Figure 18.

3.7.5. Crack Patterns of B5-SFRC-1. Crack patterns of B5-SFRC-1 are shown in Figure 19. This girder was cracked at the point of application of load, and due to presence of steel fibres, the shear resistance of cracks was considerably increased. The failure was started from the flange of girder where tensile stresses were higher. Some cracks started from the bottom of girder and vanished just after passing the neutral axis.

Crack density was considerably high at the midspan as compared to the region near supports. This indicated the enhanced flexural behaviour of specimen which complements the behaviour of steel fibres used in fibre-reinforced concrete.

3.7.6. Crack Patterns of B6-SFRC-2. Crack patterns of B6-SFRC-2 are shown in Figure 20. This girder had steel fibre content greater than that in B5-SFRC-1 and showed normal cracks within the elastic range initiating from the web of girder. Increase of fibre content in this girder resulted in reduction of the crack resisting ability due to flocculation of steel fibres. Although there were small cracks found near the
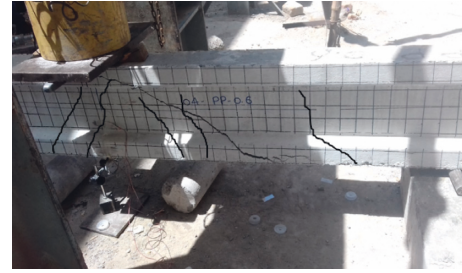
midspan of girder, there was also a widened crack that initiated from the supports and propagated towards the top of flange in shear style.

4. Conclusions

Based on the experimental results of plain and fibre-reinforced concrete girders subjected to center point loading, the following conclusions have been drawn:

(1) The use of steel fibre-reinforced concrete in specimen HyFRC increased the flexural strength by 47% and split tensile strength by 123%. This conclusion can allow use of smaller and lighter section with increased ductility.

(2) Use of steel fibres in concrete improved the ductility of girders, as in this study, the ductility index of girders containing steel fibres was 47% higher as compared to girders with polypropylene fibres.

(3) With comparison to control girder, the increase in load-carrying capacity of polypropylene fibres and steel fibres is 6% and 47%, respectively, due to high capability and intrusion of steel fibres with concrete matrix and high tensile strength as compared to that of fibrillated polypropylene fibres.

(4) The use of steel fibres and polypropylene fibres in concrete mix increases the crack resistance of girders.

(5) Ordinary concrete girder shows abrupt brittle failure and steel fibre-reinforced concrete girders show ductile failure.

(6) Steel fibre-reinforced prestressed concrete girders help to improve the flexural strength (+47%), crack resistance, energy absorption (+69%), and ductility (+47%).

(7) Hybridization of steel fibres and polypropylene fibres shows first crack load of girders at higher deflection of 6.4 mm as compared to those with individual fibres. Although deflection of SFRC girder was 13.85 mm at ultimate crack.

Data Availability

The data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

On behalf of all authors, the corresponding author states that there are no conflicts of interest.
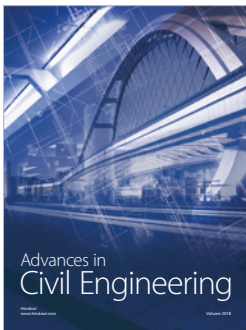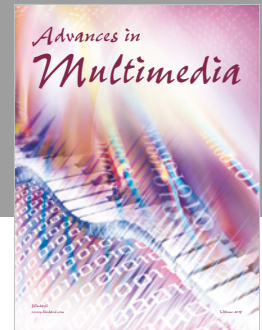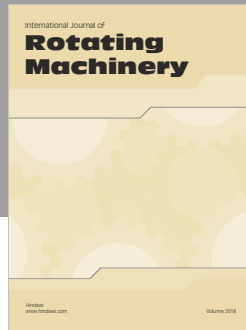
Authors' Contributions

All authors read and approved this research work

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