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Effect of steel fibre profile on the fracture characteristics of steel fibre reinforced concrete beams

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ABSTRACT

The present study investigated the mechanical characteristics of different types of steel fibre substituted high strength concrete. Influence of steel fibre volume fraction and its complex profile characteristics on the strength properties of various fibre reinforced concretes had been systematically studied in slender concrete beam sections. Reinforcing efficiency of concrete incorporating four types of steel fibres having the same aspect ratio with varying fibre profile - single hooked ends, crimped, double hooked ends, and kinked had been experimentally analyzed in flexural bending and fracture studies. Test results showed higher flexural post peak toughness (23.48 N-m) and fracture toughness (39.62MPa√mm) for double hooked steel and crimped steel fibres substituted concretes. Steel fibre reinforced concretes containing double hooked and kinked geometry exhibited higher overall performance index. Also, high volume steel fibre substitutions (1.5% V_f) in slender concrete beams showed improved fracture toughness characteristics.

Keywords: Steel fibre reinforced concretes, Ductility, Post crack toughness, Residual strength, Fracture toughness.

INTRODUCTION

Fibre reinforcements in concrete are a well-known technique for transforming a brittle concrete into a ductile composite and thereby provide better mechanical properties of concrete. Fibre geometry and orientation plays a crucial role on the effective reinforcing mechanism in concrete, as fibres bridge cracks and transfer the stress effectively to matrix (Balaguru et al., 1992). Research studies proved the significance of fibre addition in concrete to improve the strain hardening properties of concrete (song et al., 2004). The mechanical performance of steel fibre reinforced concretes was found to be dependent on the fibre volume fraction, aspect ratio, and its spatial distribution (Soufeiani et al., 2016). Reinforcing efficiency of fibres in concrete depends on the interfacial fibre-matrix bonding properties and frictional sliding of fibres during loading (Banthia et al., 1994). Crack bridging effect of fibres was dependent on the surface bonding of fibres and the effect of steel fibre profile on mechanical improvement of concretes was well documented (Jong et al., 2017). Research studies indicated that post peak characteristics of concrete were dependent on the controlled crack widening with sufficient fibre availability. Fibre addition in concrete had consistently improved the energy absorption capacity after matrix cracking and its performance characteristics are measured by various parameters such as toughness indices, equivalent flexural strength, residual strength, and fracture energy (Barros et al., 1999, 2005). The post cracking response of steel fibre reinforced concretes was systematically investigated using different sizes of notched concrete specimens and size effect due to specimen geometry (Kooiman et al., 2000). The fracture properties of concrete were dependent on specimen size, fibre orientation in orthogonal direction, and fibre availability at the crack front. In another study it was reported that aligned fibre distribution provided maximum tensile stress capacity of the ultra-high performance concrete (Duque et al., 2017). Hybrid fibre combinations at low volume fraction (0.5%) in high strength concrete exhibited higher residual strength after first cracking and a marginal increase in compressive strength of concrete was reported (Sivakumar et al., 2007).

Fracture based studies in fibre reinforced concretes had drawn wider attention owing to realistic prediction of fracture process zone and the stress intensity factor at the crack tip. The presence of inherent micro flaws acts as nucleus for crack origination (stress concentration) and further unsteady propagation leading to failure of concrete (Zhang et al., 2004). Fracture based test methods using notched beams were used for the quantitative assessment of crack parameters (Jenq et al., 1994). Fracture studies on high volume fibre reinforced concretes showed significant improvements on the post crack properties of concrete (Zhang et al., 1999). The fracture mechanics concept had been adopted to study the effects of different notch depths on the different size effects of concrete beams (Bazant et al., 1987). Experimental determination of the fracture parameters such as strain energy release rate, fracture toughness, and fracture energy is essentially used to define the crack propagation in concrete (Appa rao et al., 2005).

Energy absorption capacity of fibre reinforced concrete was better assessed using fracture based studies and CMOD measurements were used to evaluate the stress crack width relationship of concrete (Shi, 2015; Gopalaratnam et al., 1991). The fracture process of fibre reinforced concrete involves crack bridging of fibres followed by fibre pullout or rupture leading to a crack control mechanism. The formation of fracture process zone behind the crack opening undergoes an inelastic deformation (Hillerborg, 1980). Tensile strength of steel fibre reinforced concretes primarily depends on the volume fraction, anchorage efficiency, failure strain, and distribution of fibres (Lee et al., 2010). The critical review of various studies revealed that the cracking properties of fibre reinforced concretes need to be further explored to understand the reinforcing efficiency of fibres in the concrete matrix. Also, the effects of different steel fibre profile on the flexural and fracture characteristics of fibre reinforced concretes need to be identified to estimate the overall performance of steel fibres in the composite.

Therefore, the present experimental study focused on the reinforcing effects of different types of steel fibre profiles, namely, single hooked, double hooked, crimped, and kinked steel fibres. The mechanical strength improvements in high strength concrete (45MPa) with the addition of steel fibres having same aspect ratio (63) for four types of steel fibre profiles were investigated. The reinforcing efficiency of steel fibre addition in plain concrete at different volume fraction (0.5, 1.0 and 1.5%V_f) was tested to evaluate the compressive, flexural, and fracture characteristics of the composite. Flexural studies on un-notched concrete specimens were conducted to characterize the bending strength, residual strength, and post crack toughness properties. Fracture tests were conducted on notched concrete specimens to evaluate the load versus crack mouth opening displacement (CMOD) to estimate the stress crack width relationship, fracture toughness, and fracture energy. From the experimental studies conducted, the overall comparative assessments on the various types of fibre reinforced concretes are presented.

MATERIALS AND EXPERIMENTAL METHODS

Concrete Materials

The binder used in this investigation consisted of ordinary Portland cement conforming to ASTM- Type I (ASTM, 2017), locally available fine aggregate – river sand and crushed granite stone as coarse aggregates. The properties of cement and concreting materials used in this study are provided in Table 1.

Table 1. Physical properties of materials used in this study.

S.No	Property	Value
Cement – ordinary Portland cement (53-grade)		
1.	Normal Consistency	32%
2.	Initial Setting Time	47 minutes
3.	Final Setting Time	195 minutes
4.	Specific Gravity	3.12
5.	Fineness of Cement	1.34
Fine aggregate – River sand –less than 4.75mm		
1.	Specific Gravity	2.61
2.	Fineness Modulus	2.78
3.	Uniformity coefficient	3.31
4.	Coefficient of curvature	0.91
5.	Bulk density	2560 Kg/m^3
Coarse aggregate –Crushed granite –12.5mm(angular)		
1.	Specific Gravity	2.65
2.	Fineness Modulus	5.70
3.	Uniformity coefficient	1.22
4.	Coefficient of curvature	1.20
5.	Bulk density	2370 Kg/m^3

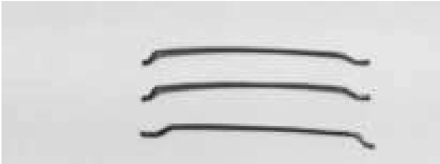
Steel Fibres

Steel fibres were used as reinforcement in concrete and four types of steel fibres were used, namely, single hooked ends steel fibre, double hooked ends steel fibre, crimped steel, and kinked steel fibres. Among these types, single hooked steel fibres were bundled together by water soluble glue for easy dispersion in concrete, whereas double hooked end steel fibres were available in loose form. Kinked steel fibres and crimped steel fibres were fabricated from the hooked end steel fibres in the laboratory using a simple mechanical pressing device. The properties of various steel fibres used are given in Table 2 and its profile characteristics are shown in Figure 1.

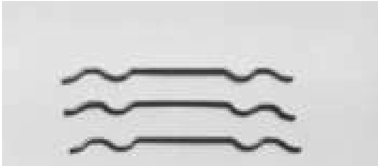
Table 2. Properties of various steel fibres used in this study.

Type of fibre	Length (mm)	Diameter (mm)	Aspect ratio (l/d)	Tensile strength (MPa)	Elastic modulus (Gpa)	Effective end anchorage length (mm)
Single hooked steel (SHS)	50	0.8	63	1100	210	8
Double Hooked steel (DHS)	50	0.8	63	1100	210	30
Crimped Steel (CS)	50	0.8	63	1100	210	*
Kinked steel (KS)	50	0.8	63	1100	210	20

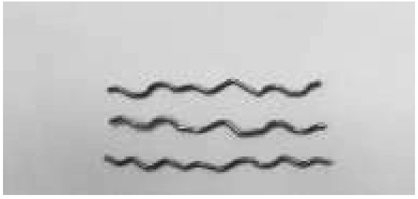
Note: * denotes the entire crimped fibre surface are bond length without having specific end anchorage.



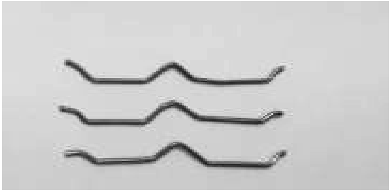
(a) Single hooked ends steel



(b) Double hooked ends steel



(c) Crimped steel



(d) Kinked steel

Fig. 1. Various steel fibre profiles used in this study.

Superplasticizer

Fibre reinforced concretes mixes tested in this study showed a reduction in workability, which necessitated the addition of superplasticizer to obtain a workable mix. A target concrete slump of 90-100mm was desired for all fresh concrete mixes. The loss in workability with fibre addition was compensated by varying the superplasticizer addition up to 1.5% to obtain a desired slump range in all fibre reinforced concretes mixes. A carboxylate ether based superplasticizer (CONPLAST SP430**)** was used to obtain the desired workability of all concrete mixes. The specific gravity of the superplasticizer was 1.25 with a solids content of 40%.

Concrete mix proportioning and specimen preparation

High strength concrete of 45 MPa was chosen for this study and trial concrete mix proportions were arrived based on ACI 211.4R (ACI, 2008) mix design specifications. A total of 13 different concrete mixes were tested in this study, out of which is the reference plain concrete mix without fibres and the remaining 13 consisted of fibre reinforced concretes mixes as given in Table 3**.** The various batches of concrete mixes were then prepared using a concrete pan mixer of 100 litres capacity rotating in horizontal axis at 60 revolutions/min. Initially, dry mixing of various concrete ingredients was carried out in a pan mixer and the required mix water with superplasticizer dosage for the concrete was added into the mixer. Later, the calculated dosage of steel fibres was added into the mixer and the total mixing was maintained for 6 minutes. The homogeneous dispersion of steel fibres in concrete was ensured and assessed for workability using slump cone method. Batch mixing of concrete was carried out systematically in order to produce a uniform concrete mix.

Table 3. Plain and fibre reinforced concretes mix proportions used in this study.

Type of concrete mix	Mix ID	Cement (kg/m³)	Sand (kg/m³)	Crushed stone 12.5mm (kg/m³)	Water (kg/m³)	SP (% by weight of cement)	Volume fraction# of steel fibres (%)	Fibre Dosage (kg/m³)
Plain slender concrete	PSC-1	474	664	1095	166	0.5	-	-
Single hooked steel fibre reinforced concretes	SHS-2	474	664	1095	166	1.0	0.5	12
	SHS-3	474	664	1095	166	1.25	1.0	24
	SHS-4	474	664	1095	166	1.5	1.5	36
Double hooked steel fibre reinforced concretes	DHS-5	474	664	1095	166	1.0	0.5	12
	DHS-6	474	664	1095	166	1.25	1.0	24
	DHS-7	474	664	1095	166	1.5	1.5	36
Crimped steel fibre reinforced concretes	CS-8	474	664	1095	166	1.0	0.5	12
	CS-9	474	664	1095	166	1.25	1.0	24
	CS-10	474	664	1095	166	1.5	1.5	36
Kinked steel fibre reinforced concretes	KS-11	474	664	1095	166	1.0	0.5	12
	KS-12	474	664	1095	166	1.25	1.0	24
	KS-13	474	664	1095	166	1.5	1.5	36

Note: # *denotes the volume fraction of steel fibres in terms of volume of concrete used in the mix*

Glued steel fibres were initially soaked in water and added into the concrete mixer immediately leading to easy separation of individual fibres (due to water soluble glue) in wet concrete mix during its rotary action. This provided a uniform dispersion of steel fibres in the matrix without the occurrence of fibre balling. Later, the fresh concrete from the concrete mixer machine was transferred into a fabricated wooden moulds of size100x150x1000mm^3. The inner sides of the wooden moulds were initially wrapped with polythene sheet to obtain a smooth surface finish as well as for easy remolding of hardened concrete. In addition, the polyethylene sheet lining provided good resistance to penetration of water from the fresh concrete to the wooden moulds. The fresh concrete mixes were placed in three layers from the bottom of wooden moulds and the needle vibration was provided subsequently on the top surface of the wooden moulds. This ensured the fibres aligning in the direction of the beam axis due to wall effect. The hardened concrete specimens were remolded after 24 hours and were cured under concealed conditions using polyethylene sheet wrapping (as shown in Figure 2). The details on the various types of concrete specimens casted in this study are provided in Table 4**.**

Fig. 2. Concealed curing of concrete specimens.

Table 4. Details of concrete specimens tested for each mix.

Type of concrete specimen	Mix type	Size of moulds (mm)	No. of. Specimens tested	Age at testing	Concrete Parameters measured
Cube	PSC-1 and all fibre reinforced concretes mixes	150×150×150	5	28	Compressive strength, residual strength, fibre efficiency
Cylinder	PSC-1 and all fibre reinforced concretes	150×300	2	28	Elastic modulus in compression
Un-notched Beam	PSC-1 and all fibre reinforced concretes	100×150×1000;	3	28	Flexural strength, residual strength, flexural strain, elastic modulus, post crack toughness
Notched Beam	PSC-1 and all fibre reinforced concretes	100×150×1000; Notch depth - 20mm; width - 4mm	3	28	Fracture Load, CMOD, Stress–crack width, fracture toughness and fracture energy

EXPERIMENTAL METHODOLOGY

Flexural strength

Flexural bending tests were conducted on un-notched concrete beam specimens as per the standard specifications given in ASTM C1609 (ASTM, 2012). Flexural tests were conducted using a servo controlled digital flexural testing machine of 100KN capacity (Figure 3). The load was applied at a rate of 0.2mm/min and a three-point bending arrangement was considered to evaluate the flexural properties. The total span of the beam was 1m with a loading span of 900mm and a sensitive mechanical dial gauge of 0.01mm accuracy was fixed at the midpoint of beam axis to measure the mid-span deflection.

Fig. 3. Flexural bending test setup in three-point loading.

The mid span deflection was calculated without any extraneous deflections in order to obtain true load-deflection plots. From the experimental test, various flexural parameters are calculated as given below:

3.1.1 Ultimate flexural strength was calculated from the following formula

$$f_u = (3Pp\, L)/(2\, b\, d^2) \quad (1)$$

where

P_p – peak failure load (N)

L – loading span of concrete beam (mm)

b – cross sectional width of concrete beam (mm)

d – cross sectional depth of concrete beam (mm)

3.1.2 Residual strength was calculated from loading the concrete specimens until 95% of ultimate load (just before failure) and then unloaded completely. Further, the specimens are reloaded till failure and the load corresponding to it is called the residual load. The corresponding stress and strain obtained are called the residual stress and residual strain, respectively.

3.1.3 Flexural strain was calculated from the following formula:

$$\varepsilon_f = 6\delta d/L^2 \quad (2)$$

where

δ – mid-span deflection at of beam (mm)

d – depth of tested beam (mm)

L – loading span (mm)

3.1.4 Residual compressive strength index was obtained from the ratio of residual strength to ultimate strength of concrete specimens.

3.1.5 Fibre reinforcement efficiency was determined from the ratio of difference in ultimate strength of fibre reinforced concretes and plain concrete to that of plain concrete.

3.1.6 Post crack toughness was measured from area under the load deformation curve from the first crack load until 4mm deflection.

Fracture strength

Fracture studies were conducted as per RILEM TC – 1985 (RILEM, 1985) recommendations using three-point bend tests in notched concrete beam specimens (shown in Figure 4). The test was carried out with a standard concrete beam specimen having a single central notch of 20mm depth and 4mm width. The fracture test was conducted using a servo controlled digital flexural loading machine with a pace rate of 0.2mm/min. The applied load versus crack mouth opening displacement (CMOD) was measured during the test and further used to determine the various fracture parameters of concrete specimens. The displacement was measured at the crack mouth using two mechanical dial gauges of 0.01mm accuracy attached to either sides of clip arrangement. The clip arrangement was rigidly fixed into notch opening and the readings from both dial gauges were used to measure the total crack displacement. From the experimental test, the following fracture parameters were evaluated.

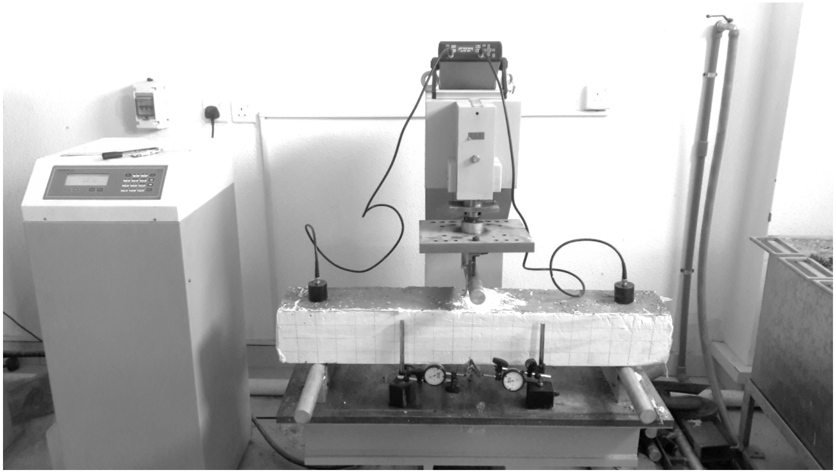


Fig. 4. Fracture test setup with mechanical dial gauges – third point loading.

3.2.1 Fracture strength (N/mm^2) was calculated from peak load at failure

$$f_p = (3P_f\,L)/(2\,B\,(D-a)^2) \quad (3)$$

P_f – fracture load (N)

L – loading span (mm)

B – width of beam (mm)

D – depth of beam (mm)

a – notch depth (mm)

3.2.2 Fracture toughness is defined as the resistance of the material against fracture. It was determined from three-point bend tests on concrete specimen and calculated from the equations as given below:

a) Bowers fracture model [Bower, 2009]

$$K_I = \frac{4P}{B}\sqrt{\frac{\Pi}{W}}\,[1.6\left(\frac{a}{W}\right)^{1/2} - 2.6\,(\frac{a}{W})^{3/2} + 12.3\,(\frac{a}{W})^{5/2} - 21.2(\frac{a}{W})^{7/2} + 21.8(\frac{a}{W})^{9/2}] \quad (4)$$

where

K_I - fracture toughness (MPa√mm)

P – applied load (KN)

B – Thickness of specimen (mm)

a – crack/notch depth (mm)

w – width of specimen (mm)

b) ASTM E1290-08 fracture model [ASTM, 2017]

$$K_I = \frac{6P}{BW} a^{0.5} \, Y \quad (5)$$

where

$$Y = \frac{1.99 - a/W\,(1 - a/W)\,(2.15 - 3.93a/W + 2.7\,(a/W)^2)}{(1 + 2a/W)\,(1 - a/W)^{3/2}} \quad (6)$$

3.2.3 Fracture energy (N/m) was calculated from the energy required to open a cracked concrete and is given by the equation

$$G_I = K_I^2/E \quad (7)$$

where

G_I –fracture energy (N/m)

K_I^2 – fracture toughness (MPa√mm)

E - elastic modulus (N/mm^2)

3.2.4 Characteristic Length [Gettu et al., 1998; Zhang et al., 2004] was calculated to estimate the concrete brittleness and given by the equation

$$L_{ch} = (E\ G_f)\ /\ ft \quad (8)$$

where

L_{ch}– characteristic length (mm)

E – elastic modulus of the concrete (GPa)

G_f - fracture energy of concrete (N/m)

ft – flexural tensile strength (MPa)

TEST RESULTS AND DISCUSSIONS

Flexural properties

Fibre substituted concretes showed favorable improvements on flexural bending, which can be evidently seen from the load – deflection plots provided in Figure 5. The significance of steel fibre reinforced concretes on the flexural parameters had been discussed in further sections.

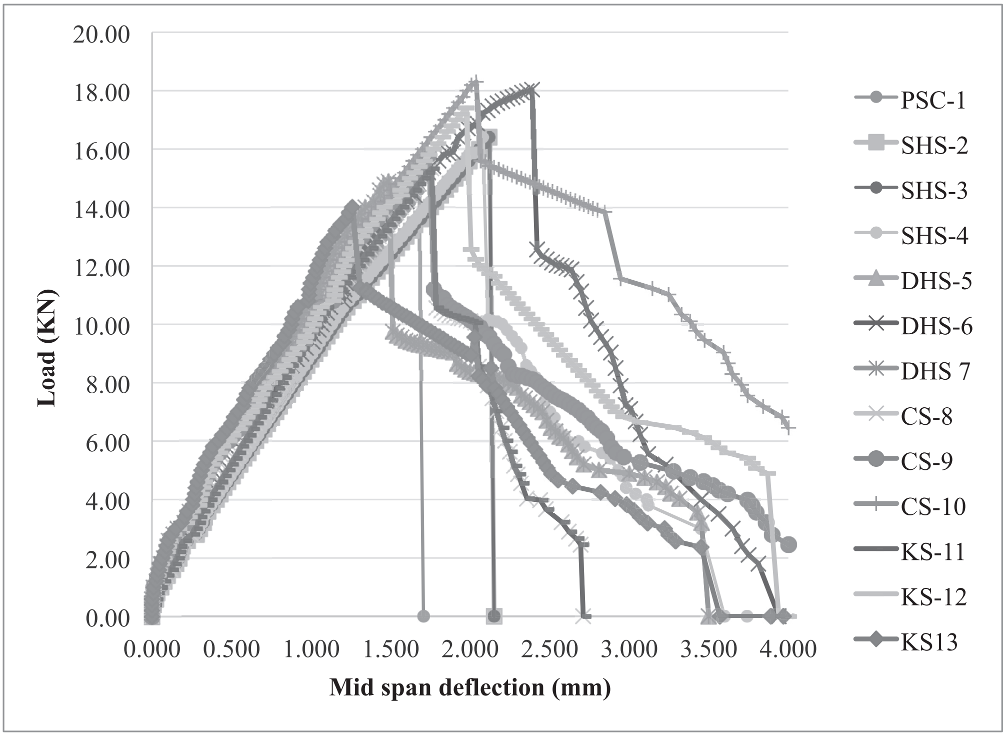


Fig. 5. Load-Deflection plots for various concrete specimens in flexural bending.

Ultimate flexural strength

The flexural performance of various concrete mixes as obtained from the load-deflection plots is provided in Table 5. Among the different fibre reinforced concretes types tested, the effect of fibre profile and volume fraction was found to influence the overall flexural performance of fibre reinforced concrete mixes. All fibre substituted concretes showed favorable flexural strength enhancement compared to plain concrete specimens (PSC-2). The bending resistance of plain concrete was not appreciable as the failure occurred suddenly upon reaching the ultimate load. However, fibre reinforcements in concrete provided improved bending resistance due to the linear profile of steel fibres, which were capable of stress transfer mechanism in the matrix resulting in maximum flexural strength. The concrete specimens containing double hooked steel fibres (DHS-7) and kinked steel fibres (KS-13) showed a maximum flexural strength of 10.94 N/mm^2 and 10.60 N/mm^2, respectively. Test results obtained were reportedly in good agreement as that of the study by Banthia et al. (1992) in which a maximum flexural strength of 10.2 N/mm^2 was reported for deformed steel fibre reinforced concrete. The flexural strength properties of concrete were favorably improved for various fibre types tested in this study. Low volume fibre addition at 0.5% had shown an average flexural strength gain up to 6.2% compared to plain concrete (PSC-1). However, the flexural performance was found to improve appreciably with the increase in fibre volume fraction from 1.0% to 1.5% V_f owing to large fibre availability in the concrete system. This would have possibly reduced the inter-fibre spacing and was effective in delaying the crack origination. Load deformation plots shown in Figure 5 indicated the steeper pre-peak hardening region for fibre reinforced concretes as compared to plain concretes. However, the type of fibre profile showed significant influence on the pre-peak hardening. The crack bridging effect of double hooked steel fibres just after reaching the ultimate load can be seen in Figure 6. At low fibre volume fraction, sudden drop in load carrying capacity after reaching ultimate load was noticed for all fibre reinforced concretes. However, this drop in load was controlled with increasing fibre volume fraction for all steel fibre substituted concretes. Most notably the fibre anchorage and interfacial fibre matrix bond in the case of double hooked, kinked, and crimped steel fibres were effective in crack bridging. This was evidently seen with reduced crack widths for high volume fibre substituted concrete specimens.

Table 5. Flexural bending properties of various concrete mixes investigated.

Mix ID	Volume fraction of steel fibres (%)	Flexural Stress* - N/mm² @ 28 days		Flexural strain (x 10^{-3} mm/mm)		Maximum deflection @ ultimate load	Flexural modulus (x 10^3 N/mm²)	Residual flexural strength ratio	Post peak toughness (N-m)
		Ultimate Strength	Residual strength	Ultimate strain	Residual strain				
PSC-1	0	6.20	0	2.36	0	2.12	2.60	0	0
SHS-2	0.5	6.94	3.32	2.31	0.47	2.08	3.08	0.48	9.22
SHS-3	1.0	7.54	3.49	2.65	0.56	2.38	2.84	0.46	14.29
SHS-4	1.5	8.19	4.83	2.62	0.56	2.41	2.93	0.59	17.66
DHS-5	0.5	7.81	5.12	1.65	0.55	1.48	5.35	0.66	15.82
DHS-6	1.0	9.30	5.98	1.93	0.58	1.74	4.81	0.64	20.29
DHS-7	1.5	10.94	6.92	2.26	0.56	2.03	4.83	0.63	23.48
CS-8	0.5	7.02	3.43	1.95	0.58	1.75	3.49	0.49	11.97
CS-9	1.0	7.73	3.89	2.20	0.56	1.97	3.24	0.50	16.47
CS-10	1.5	9.40	6.50	2.21	0.57	2.12	3.26	0.69	22.31
KS-11	0.5	7.47	5.37	1.39	0.59	1.25	6.35	0.72	13.71
KS-12	1.0	9.44	5.87	1.79	0.58	1.61	5.25	0.62	17.42
KS-13	1.5	10.60	6.97	1.87	0.56	1.68	5.65	0.66	22.78

Note: **Test results reported are an average of 3 specimens with coefficient of variance 0.043 and standard deviation of* $\pm$ *0.38 N/mm²*

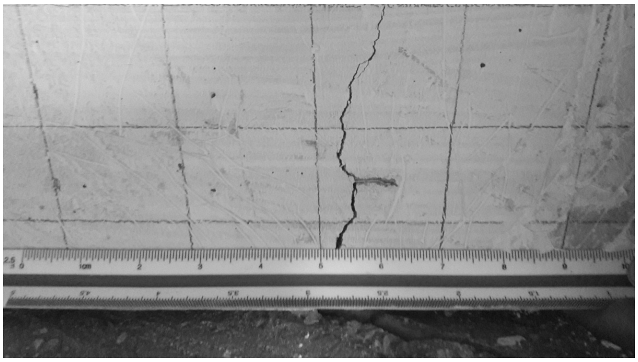


Fig. 6. Crack bridging of steel fibres - double hooked steel fibre reinforced concretes in flexural loading.

Residual flexural strength

Residual strength results of all fibre reinforced concretes demonstrated the strength retention capacity of fibre reinforced concretes in bending even after being preloaded up to 95% of ultimate load. Residual strength of plain concrete (PSC-1) was not significant as the concrete specimens failed immediately upon loading, since the plain concrete specimens had sustained multiple cracking internally and further lead to sudden failure upon reloading. However, the strength retention of fibre reinforced concretes was significant and more specifically the residual strength values of double hooked steel (DHS-7) and kinked steel (KS-13) showed higher value compared to other fibre reinforced concrete specimens. Test results implied that steel fibres were effectively sharing stresses and undergo self-straining. This could possibly minimize the matrix cracking without allowing the concrete for complete failure and had appreciable strength retention during reloading. Apparently, all fibre reinforced concretes showed higher strength retention capacity even it was preloaded initially. Furthermore, the residual strength gain was considerate at high volume fibre substitution in concrete and the residual strength values of fibre reinforced concretes were in the range of 0.48 to 0.72. It was also imperative that failure and residual strength properties of concretes discussed in compression were found to be different in flexural tension. Also, flexural results obtained from the various fibre reinforced concretes were indicative of the level of performance achieved with respect to type of fibre profile and its reinforcing efficiency in matrix.

Post peak toughness

The ability of fibres to bridge the unsteady crack growth after cracking resulting in energy absorption was measured in terms of post peak toughness. The toughness measurements of various concrete specimens are provided in Table 5 and a typical toughness measurement is represented in Figure 7.

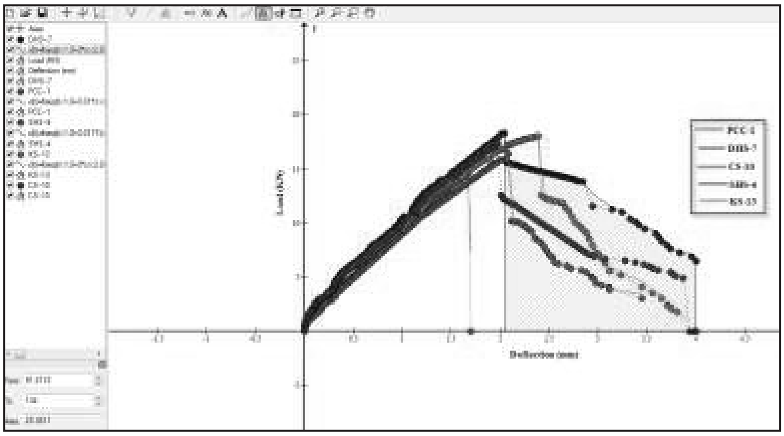


Fig. 7. Post peak toughness calculation for fibre reinforced concretes in flexural loading.

More specifically, the test results obtained were indicative of the crack bridging efficiency of various steel fibres at large crack widths. Compared to plain slender concrete (PSC-1) sections, all fibre reinforced concretes showed higher post peak toughness values. Among the various fibre reinforced concretes, the double hooked steel fibre reinforced concretes (DHS-7) exhibited maximum toughness value of 23.48N-m. Similarly, kinked steel fibre (KS-13) and crimped steel fibre (CS-10) concretes showed a maximum toughness value of 22.78 and 22.31 N-m, respectively. Also, toughness of fibre reinforced concretes was significantly higher with an increase in fibre volume fraction (1.5%) for all fibre types. However, single hooked steel fibres concretes (SHS-4) reportedly showed lower toughness value (17.66 N-m) than other fibre types. The sudden drop in load after ultimate load was essentially lower for long steel fibre substituted concretes as well as for high volume fibre reinforced concretes. The profile of steel fibres played a major role on the post crack bridging efficiency, as the end anchorages had been efficient in transferring large stresses without fibre pullout. This was evidently observed during flexural testing with a rupture sound of steel fibres and this behavior was specific for double hooked steel, kinked steel, and crimped steel fibre reinforced concretes specimens. In the case of kinked steel fibre reinforced concretes, a similar trend was noticed as the fibre rupture occurred at high volume fibre substituted concretes. The flexural performance of single hooked steel fibres was relatively lower than other fibre types, due to insufficient end anchorage resistance, which resulted in fibre pullout as seen in Figure 8. Test results were also indicative that single hooked steel fibre reinforced concretes specimens suffered a sudden drop in load after reaching ultimate load. This could be due to ineffective end anchorages of single hooked steel fibres compared to other fibre types, which had better end anchorages and due to complex fibre profile.

Fig. 8. Fibre pullout in single hooked steel fibre reinforced concrete specimens in flexural loading.

The sudden drop in peak load was witnessed for all fibre reinforced concretes due to sudden release of fracture energy accompanied by rapid crack growth and crack widening. Apparently, the crack bridging mechanism occurred as a result of intercepting fibres resulting in immediate stress transfer along the fibre length.

Fracture properties

Performance characteristics of fibres in high strength concrete were assessed from the fracture studies by measuring the load versus crack mouth opening displacement relationship. The various fracture parameters were evaluated to assess the relative improvements by steel fibres addition, discussed in various sections below. The relationship between fracture load and CMOD measurements is illustrated graphically in Figure 9.

Fracture strength

The experimental observations from fracture tests are provided in Table 6. Experimental observation indicated that the fracture in concrete specimens occurred by means of a single crack growth originated from the notch tip. Fracture strength of all fibre reinforced concrete beams was higher than plain concrete specimens (PSC-1). Similarly, the fracture strength was considerably improved for all types of fibre incorporated concrete specimens at high fibre volume fraction. Fracture strength of double hooked end steel fibre incorporated concretes (DHS-7) showed a maximum value (13.26 N/mm^2) compared to other fibre reinforced concrete specimens. Similarly, crimped steel fibre (CS-10) and kinked steel fibre (KS-13) concretes reported a similar increase in fracture strength at high volume fraction (1.5%). Test results denoted that fracture strength of fibre reinforced concretes was dependent on the effective fibre profile as in the case of double hooked steel fibre, crimped, and kinked steel fibres, since the matrix bonding in these fibres was efficient as a result of more anchorage length (as referred to in Table 2).

In addition, steel fibre performance was also dependent on the fibre volume fraction and preferential alignment of steel fibres normal to loading plane. Fracture growth of concrete was effectively controlled by crack bridging mechanism of intercepting steel fibres across the crack growth and thereby redistributes the stresses in matrix. It is better understood from the experimental results that complex steel fibre profiles such as double hooked ends, crimped, and kinked had direct influence on fracture strength due to effective fibre matrix bonding. Test results also indicated that the large steel fibre availability at the crack front can control the rapid propagation of cracks. Compared to relative flexural bending properties of fibre reinforced concretes discussed in earlier section, the findings from fracture strength revealed the effect of steel fibre profile during crack propagation.

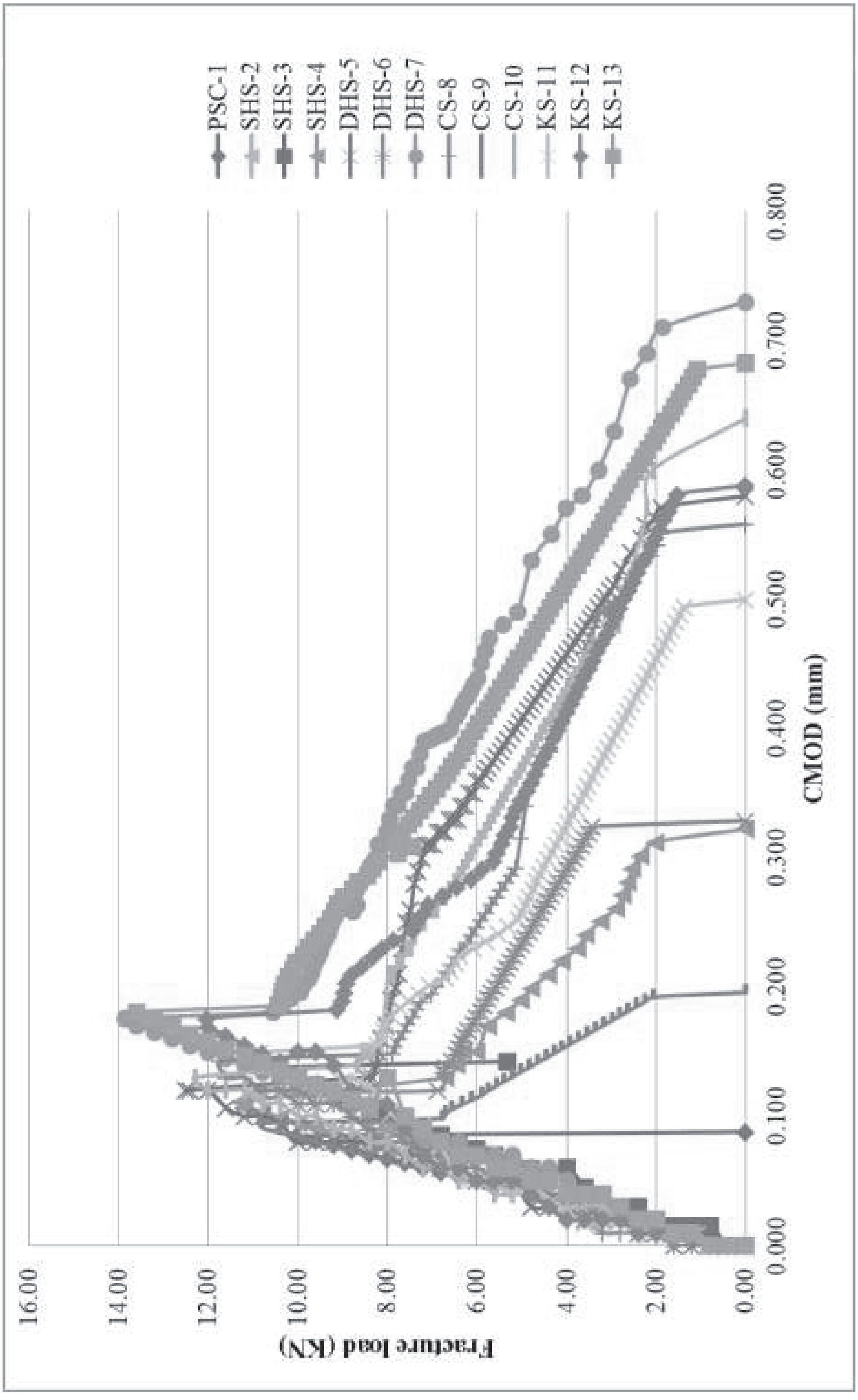


Fig. 9. Fracture load vs. CMOD curves for various concrete specimens tested.

Table 6. Fracture properties of various concrete mixes tested.

Mix ID	Volume fraction of steel fibres (%)	Fracture load (KN)	Fracture strength of notched beam* (N/mm²)	Fracture Toughness, K_{IC} (MPa√mm)		Fracture Energy, $G_I = K_I^2/E$ (N/m)		Characteristic length (mm)
				Bowers model	ASTM E1290-08	Experimental σ-w curve	Bowers model	
PSC-1	-	6.67	5.33	23.27	28.59	172.15	181.93	99.45
SHS-2	0.5	7.80	6.23	26.48	35.68	192.45	178.83	114.98
SHS-3	1.0	9.13	7.29	29.87	37.43	276.04	185.96	123.02
SHS-4	1.5	10.12	8.08	35.27	38.38	304.15	266.33	261.58
DHS-5	0.5	9.41	7.52	32.61	35.48	234.90	192.78	221.52
DHS-6	1.0	10.72	8.56	34.24	36.16	312.56	272.76	277.83
DHS-7	1.5	13.26	10.59	39.62	41.82	367.78	431.86	338.58
CS-8	0.5	8.84	7.06	25.62	35.28	213.14	145.31	106.77
CS-9	1.0	9.12	7.29	33.73	39.96	246.20	314.17	218.45
CS-10	1.5	11.54	9.22	37.48	40.78	376.04	420.55	290.10
KS-11	0.5	9.15	7.31	31.41	34.12	203.16	150.73	197.69
KS-12	1.0	10.50	8.39	32.71	37.07	302.23	244.12	271.55
KS-13	1.5	12.86	10.27	36.04	39.22	345.67	423.05	267.28

Note: **Test results reported are an average of 3 specimens with coefficient of variance 0.051 and standard deviation of* $\pm$*0.25 N/mm²*

Fracture toughness

Crack growth resistance in brittle concrete systems was effectively measured in fracture test using fracture toughness (K_{IC}) and fracture energy (G_I). Energy released during work done in fracturing a material provides an indicative measure on the toughness of the composite. Comparative assessment of fracture toughness calculated from experimental observations using Bower's model (24) and ASTM model (25) is provided in Table 6. The calculated toughness values from both the models were almost consistent and represented better interpretation. Among the various fibre reinforced concretes tested, maximum fracture toughness values of 39.62MPa√mm and 37.48MPa√mm were obtained for double hooked steel fibres (DHS-7) and crimped steel fibre (CS-10), respectively, for concretes at high fibre volume fraction. The fracture toughness value of plain concrete (23.27 MPa√mm) obtained in this study was in good agreement as obtained in another study (Kursat et al., 2005). Test results also indicated higher toughness value for all fibre reinforced concretes types compared to plain concrete specimens. As anticipated, the high volume fibre (1.5%V_f) substitution showed higher toughness of the composite and provided a steady crack growth from the notch during the test. Better fibre-matrix interfacial bonding and efficient end anchorages had shown improved fracture properties in double hooked, crimped, and kinked steel fibre profile types.

Experimental results duly confirm that fibres present across the crack growth were efficient in crack bridging properties resulting in the development of larger fracture process zone. Fibres present in this zone could possibly provide the required inelastic deformation with steady crack growth upon reaching the yield strength. Test results confirm that the fracture properties are appreciably improved in high volume fibre substituted concretes resulting in an efficient crack bridging mechanism leading to controlled failure process. Most notably the crack widths of high volume hooked steel fibre substituted concretes were found to be thinner than single hooked steel fibre reinforced concretes [(Figures 10(a) and 10(b)].

Fig. 10(a). Thin crack width of high volume fibre reinforced concretes specimens (DHS-7).

Fig. 10(b). Thick crack width of steel fibre reinforced concretes specimens (SHS-4).

Fracture energy

The total energy required for growth of cracks till the energy released upon complete failure of concrete specimen provides an essential measure on the fracture energy of the composite. Test results provided in Table 6 showed maximum fracture energy in the case of all fibre reinforced concretes compared to plain concrete. Among the various fibre reinforced concretes, highest fracture energy of 376 N/m and 367N/m was obtained for crimped steel (CS-10) and double hooked steel fibre (DHS-7) concretes. Similarly, the fracture energy of kinked steel fibre reinforced concretes (KS-13) was on par with that of other high volume fibre reinforced concretes. The computed fracture energy from the experimental results was also validated using Bowers fracture model (2009), which provided consistently similar fracture values. The fracture properties of fibre reinforced concretes were essentially controlled by the type of fibre profile, fibre volume fraction, and preferential fibre alignment normal to loading plane.

Characteristic length

Characteristic length value determined in this study iswas used to define the degree of concrete brittleness and was measured from the fracture energy. From the characteristic length values provided in Table 6, it can be noted that a maximum value of 338mm was observed in the case of double hooked steel fibre reinforced concretes specimens. Similarly, the characteristic length of crimped steel fibres and kinked steel fibre reinforced concretes was found to be higher - 290mm and 267mm, respectively. Most notably, the characteristic length of fibre reinforced concretes increased with the increasing fibre volume fraction indicating the improvement on concrete ductility. It can be drawn that the brittleness of concrete reduced with the increase in characteristic length value. Hence, it can be clearly justified from the tabulated results that the characteristic value of plain concrete was found to be lower than other fibre reinforced concretes. This showed the significant transformation of brittle plain concrete to ductile composite with a corresponding increase in steel fibre substitution in plain concrete.

OVERALL FIBRE PERFORMANCE INDEX

From the various test results obtained in this study, the summative comparative analysis given in Table 7 provides the relative fibre performance in various fibre reinforced concretes as compared with that of plain concrete (PSC-1). The overall fibre performance index of different steel fibre substituted concretes exhibited superior flexural bending and fracture resistance properties compared to plain concrete sections.

Table 7. Comparative assessment on the mechanical properties of various fibre reinforced concretes.

Fibre type	Mix Id	V_f of fibres (%)	% increase in steel fibre reinforced concretes compared to plain slender concrete – PSC1				Overall performance index*
			Flexural strength	Fracture strength	Fracture toughness	Fracture energy	
Single hooked steel (SHS)	SHS-2	0.5	11.94	16.89	13.79	11.79	13.60
	SHS-3	1.00	21.61	36.77	28.36	60.35	36.77
	SHS-4	1.50	32.10	51.59	51.57	88.30	55.89
Double hooked steel (DHS)	DHS-5	0.50	25.97	41.09	40.14	36.45	35.91
	DHS-6	1.00	50.00	60.60	47.14	81.56	59.83
	DHS-7	1.50	76.45	98.69	70.26	113.64	89.76
Crimped steel (CS)	CS-8	0.50	13.23	32.46	10.10	23.81	19.90
	CS-9	1.00	24.68	36.77	44.95	43.01	37.35
	CS-10	1.50	51.61	72.98	61.07	118.44	76.02
Kinked steel (KS)	KS-11	0.50	20.48	37.15	34.98	18.01	27.66
	KS-12	1.00	52.26	57.41	40.57	75.56	56.45
	KS-13	1.50	70.97	92.68	54.88	100.80	79.83

Note: * *Denotes the average mechanical strength property of a particular fibre reinforced concretes mix compared to plain concrete (PSC1)*

However, the real merits of fibre type with respect to profile characteristics had witnessed the improvements on the composite mechanical characteristics. It can be clearly identified from Table 7 that the overall performance index of double hooked steel fibres (DHS-7) was relatively higher (89.76%) followed by kinked steel fibres (KS-13) and crimped steel fibres (CS-10). The overall test results were also indicative that the reinforcing efficiency of single hooked steel fibres was comparatively lesser than the other complex steel fibre profiles used in this study.

CONCLUSIONS

Based on the test results obtained from this study, some of the specific conclusions that can be drawn from this study are summarized below:

i) Mechanical performance of different steel fibres in high strength slender concrete sections had been systematically investigated with different steel fibre profile–single hooked, double hooked, crimped, and kinked steel fibres.

ii) Even though the four different types of steel fibres tested in this study had the same aspect ratio, the relative improved performance of fibres in the concrete matrix was dependent on its complex profile.

iii) Plain concrete specimens exhibited uncontrolled failure leading to sudden and brittle failure. However, fibre addition improved the flexural bending properties leading to improved strain softening characteristics after failure. Flexural properties of all fibre reinforced concretes were found to be maximum compared to plain concrete and also showed favorable improvements with increasing fibre volume fraction resulting in superior composite properties in terms of flexural strength, residual strength, and post crack toughness. However, maximum flexural tensile strength of 10.94 MPa and 10.6 MPa was noted for concretes containing double hooked steel fibre profile (DHS-7) and kinked steel fibre (KS-13) profile, respectively.

iv) Post peak toughness of all fibre reinforced concretes showed exceptionally higher value than plain concretes due to which the post elastic deformation capacity was dominated by the fibres bridging the crack opening at large crack width corresponding to large deformation. Among the various fibre types, maximum post crack toughness value of 23.48 N-m was obtained for concretes containing double hooked steel fibres (DHS-8).

v) The increase in fracture strength among various fibre reinforced concretes was realized at high fibre volume fraction up to 1.5%. Among the different steel fibres, the double hooked steel fibres (DHS-7) and kinked steel fibre (KS-13) incorporated concretes exhibited highest fracture strength of 10.59 and 10.27 N/mm^2, respectively, than other fibre reinforced concretes types.

vi) Fracture toughness properties of all fibre reinforced concretes were comparatively higher than plain concrete. Maximum fracture toughness of 39.62MPa√mm was obtained for double hooked steel fibre reinforced concretes (DHS-7) followed by crimped hooked steel fibre (CS-10) of 37.48 MPa√mm.

vii) Fracture energy of double hooked steel and crimped steel fibre reinforced concretes recorded maximum value of 376N/m and 367N/m, which indicates the consistent performance of fibres in the concrete matrix.

viii) Ductility and characteristic length were found to increase with the increase in fibre volume fraction. The test values denoted the effectiveness of fibre reinforced concretes in transforming a brittle concrete into ductile composite.

ix) Test results obtained from this study are indicative of fibre performance in terms of effective fibre profile, preferential alignment of fibres in concrete normal to loading plane, and high volume fibre addition up to 1.5% V_f. Compared to single hooked steel fibres, the relative performance of double hooked, crimped, and kinked steel fibre profiles had been highly appreciative in terms of mechanical strength properties.

x) Overall performance index of fibre reinforced concretes indicated that double hooked steel fibre reinforced concretes achieved maximum reinforcing efficiency (89.76%) in high strength concrete followed by kinked steel fibre (79.83%) and crimped steel fibres (76.02%). Also, the performance characteristics of fibres in concrete were purely dependent on the post crack–bridging efficiency as a result of effective steel fibre profile.

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تأثير الألياف الفولاذية على عارضة الخرسانة المسلحة وخصائص التصدع

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الخلاصة

ترتكز هذه الدراسة في بحثها على الخصائص الميكانيكية لأنواع مختلفة من الخرسانة ذات الألياف الفولاذية عالية القوة. تمت دراسة تأثير حجم الألياف الفولاذية وخصائصها المعقدة على خواص القوة لمختلف الخرسانات المقواة بالألياف بشكل منهجي في أجزاء الخرسانة الضعيفة. وتم تعزيز كفاءة الخرسانة التي تشتمل على أربعة أنواع من ألياف الصلب التي لها نفس نسبة العرض إلى الارتفاع مع تباين المظهر الجانبي للألياف – نهايات خطافية مفردة، ونهايات معقوفة، ونهايات مزدوجة الخطاف، وتم تحليلها بشكل تجريبي من حيث الانحناء والكسور. وأظهرت نتائج الاختبار أعلى صلابة للإنحناء (23.48 نانومتر) وصلابة الصدع (39.62 ميجا باسكال) للصلب المزدوج والألياف الفولاذية المتعرجة. وأظهرت الخرسانة المسلحة بالألياف الفولاذية التي تحتوي على هندسة متوازنة ومزدوجة للانحناءات ارتفاعاً في مؤشر الأداء الكلي. وكذلك أظهرت بدائل ألياف الصلب ذات الحجم الكبير (V_f 1.5%) في العوارض الخرسانية الضعيفة تحسناً في ليونة الصدع ومدى قوة وجودة خاصيتها.