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Overview on the Theoretical Prediction of Shear Resistance of Steel Fibre in Reinforced Concrete Beams

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Abstract. This paper presents the experimental results and theoretical study on the shear strength of reinforced steel fibre concrete beams. Failure of reinforced concrete beams in form of diagonal tension failure has been very complex to predict accurately. The inclusion of steel fibre substantially increased the ductility of the concrete and improves the shear behaviour of the beams. Due to bridging effects provided by the steel fibre, it reduces the brittle shear failure of plain concrete. Previous test data were categorized by the influence of steel fibre in reinforced concrete beams. All test results showed that steel fibre has a significant influence on the ultimate shear strength of reinforced concrete beam and control the development of crack propagation. A discussion on the contribution of steel fibre on the shear strength is also presented, with reference to the past researcher formula and RILEM provisions. The shear strength prediction for steel fibre reinforced concrete beams were compared with those obtained by the experimental test. The comparison shows that the mean value of the ratio of the experimental ultimate shear strength to predicted ultimate shear strength for the beams tested was about 0.59 to 1.96.

1. Introduction

Many researchers have also established that the use of steel fibres in reinforced concrete lead to the increase in the ultimate shear capacity and improved ductility. Steel fibres improved concrete quality and the post-crack performance as well as reduced the brittle behaviour of normal and high strength concrete [1]. Steel fibre in concrete transferred stress across cracks, thus, provided post cracking diagonal tension resistance to reinforced concrete beams.

Shear stresses across small cracks can be transferred by the aggregate interlock mechanism, and this effect dissipates as the crack width increases. The shear transfer abilities across the cracks are influenced by the strength of concrete, crack width, volume of coarse aggregate and the mechanical properties of concrete. When the principal tensile stresses within the shear domain of a reinforced concrete beam exceeded the tensile strength of concrete, diagonal cracks developed and propagated, which resulted in sudden failure of the beam [2]. The main function of fibre is resisting the formation and growth of cracks, thus, resulted in higher ultimate strain for fibre reinforced concrete compared to that of plain concrete [3]. The shear strength increased up to 45% as compared to that of specimens with conventional shear reinforcement. Furthermore, the hooked-end type fibres in concrete increased the shear strength between 45% and 70% [4].

Many researchers have introduced a simple general theory for predicting the shear strength of steel fibre reinforced concrete members either with or without shear reinforcement. Sharma [5] proposed an equation to calculate the ultimate shear at failure. The proposed equation is a function of the concrete



tensile strength and the shear span-to-depth ratio. The equation was validated with 41 other tests on steel fibre reinforced concrete beams. However, the equation ignored some important parameters that contributed to the shear strength, such as fibre volume, aspect ratio and tensile reinforcement ratio.

The effect of shear span-to-effective depth ratio on the shear behavior of SFRC beams has been extensively studied by various researchers. The beams with a smaller span-to-effective depth ratio can resist higher shear. This is due to the arch action, which is basically the direct transfer of the load to the support through a compressive strut. Batson *et al.* [6] proposed a critical value of shear span-to-effective depth ratio of 3 for SFRC beams, similar to 2.5 for RC beams, as proposed by Zsutty [7]. The shear strength of the beam increases rapidly when the shear span-to-effective depth ratio is less than 2.0.

SFRC is conventional concrete reinforced with discrete fibres of short length and small diameter. Small steel fibres are distributed randomly into concrete to improve its shear strength. As stated in ACI 544, 3R-08 [8], fibre volume fraction used in SFRC should be within 0.5% to 1.5% because adding fibres may reduce the workability of concrete. The effect of fibres is that it enhanced post cracking behaviour and toughness. The steel fibre helps to form bridges through the crack development in concrete and therefore provided more resistance to the crack growth.

The objective of this article is to study the theoretical prediction of shear resistance of reinforced concrete beams containing steel fibres compared with experimental data. This also include the influence of fibre volume and shear span-to-depth ratio on the prediction of shear strength. The ability of steel fibre for limiting the failure and increase the shear capacity in the steel fibre reinforced concrete beams was also reviewed.

2. Prediction of the shear strength

2.1 Shear analysis of reinforced concrete beams containing steel fibres

Shear in a reinforced concrete beam without shear reinforcement causes cracks on inclined planes near the support as shown in Figure 1. The cracks are caused by the diagonal tensile stress. The shear failure mechanism is complex and depended on the shear span-to-effective depth ratio (a_v/d). Shear span, a_v is defined as the distance between the support and the major concentrated load acting on the span. Fibres improved concrete toughness, control crack development and increase the ultimate shear strength. Other mechanisms are also improved by the presence of fibres which are confinement of the aggregate interlock and dowel action of the longitudinal reinforcement at the bottom chord. The total shear forces can be written as follows:

$$V = V_c + V_a + V_d + V_f \quad (1)$$

where V_c is the shearing force across the compression zone, V_a is the interlocking force, V_d is the dowel action force and V_f is the vertical components of the fibre pull-out force along the inclined crack.

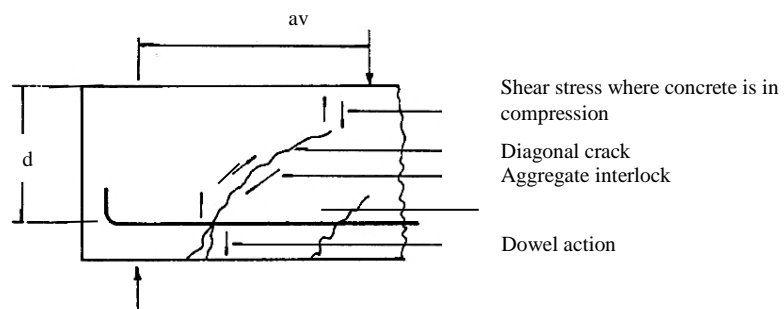


Figure 1. Different actions contributing to shear strength [9].

2.2 RILEM Model

The first model was published by RILEM TC-162-TDF [10]. The design value of the shear capacity of beams with longitudinal reinforcement but without stirrups is given by:

$$V_{Rdf} = V_{Rdc} + V_{fd} \quad (2)$$

where V_{Rdc} is the shear capacity of the beam reinforced with longitudinal reinforcement but without stirrups as given in BS EN 1992-1-1. V_{fd} is the shear resistance provided by the fibres given as:

$$V_{fd} = 0.7 k_f k_1 \tau_{fd} b d \quad (3)$$

k_f is the factor for taking the contribution of the flanges of a T-section given as:

$$k_f = 1 + n \left(\frac{h_f}{b_w} \right) \left(\frac{h_f}{d} \right) \quad \text{and} \quad k_f \leq 1.5 \quad (4)$$

h_f is the height of the flanges (mm), b_f is the width of the flanges (mm) and b_w is the width of the web (mm). The relationship between h_f , b_f and b_w together with k_1 is given in the following Eq. (5) and Eq. (6) as:

$$n = \frac{b_f - b_w}{h_f} \leq 3 \quad \text{and} \quad n \leq \frac{3b_w}{h_f} \quad (5)$$

$$k_1 = 1 + \sqrt{\frac{200}{d}} \quad \text{and} \quad k_1 \leq 2 \quad (6)$$

d is the effective depth (mm) and τ_{fd} is the design value of the increase in shear strength due to the fibres given as:

$$\tau_{fd} = 0.12 f_{R4,k} \quad (7)$$

$$\tau_{fd} = 0.18 f_{R4,m} \quad (8)$$

$f_{R4,k}$ is the characteristic value of the residual flexural strength for Crack Mouth Opening Displacement (CMOD) = 3.5 mm, while $f_{R4,m}$ is the average value of the residual flexural strength for CMOD = 3.5 mm.

The recommendations from RILEM TC 162-TDF [10] to the equivalent flexural tensile strength is replaced by the residual flexural tensile strength; the equivalent flexural tensile strength is derived from the contribution of steel fibres to the energy absorption capacity (area under the load-deflection curve), while the residual flexural tensile strength is derived from the load at a definitely CMOD or mid-span deflection (δ_R). The value which is used for the ULS is $f_{Rk,4}$ (CMOD = 3.5 mm or $\delta_{R,4} = 3.0$ mm) is related to the strain of 2.5%.

Figure 2 shows that the ability of fibres to provide additional post-cracking capacity which occurred during the early stages of failure when the crack widths are small. When plain beams are first cracked, there is an immediate reduction in load together with a large crack opening.

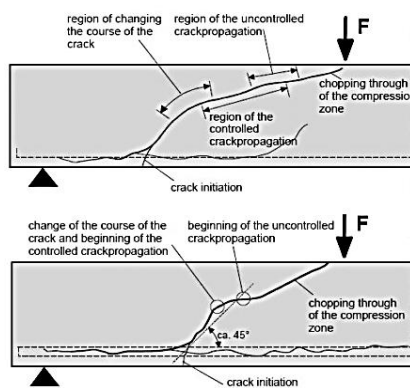


Figure 2. Failure mechanism observed at beams with and without steel fibres, respectively and without stirrups [10].

2.2.1 Comparison with experimental results and Theoretical Rilem Model

The beams were designed to fail in shear according to Eurocode 2 (EN 1992-1-1). There were no stirrups in the beams. The experimental parameters were the shear span-to-depth ratio (a_v/d), fibre type and fibres dosage, V_f . For each concrete composition, three different (a_v/d) were used: 0.5, 1.5 and 2.5. Beams with a very small (a_v/d) were used to check the limitation of the prediction models. The beams were tested under a four-point bending setup to compare the experimental ultimate shear capacity with the analytical models published by RILEM. The steel fibre (UN) is an undulated fibre, the steel fibre with a conical end anchorage (CF) and the macro-synthetic fibres (Sy) are made of a blend of polypropylene and polyethylene. Volume fractions, V_f were 0.26% (20 kg/m³) and 0.51% (40 kg/m³) for the steel fibres and 0.49% (4.5 kg/m³) for the macro-synthetic fibre. For the concrete mixtures used in the study, the increase of ultimate shear capacity ranges from 48% (UN20) to 82% (CF40).

The experimental results are given in Table 1, which indicates that the fibre volume fraction increases the shear capacity. The use of fibres with a volume fraction $V_f = 0.26\%$ gave an increase on the shear capacity of minimum 48% for (a_v/d) between 0.5 and 2.5. The ultimate shear capacity increase in ranges from 48% (UN20) to 82% (CF40) for (a_v/d) = 2.5 and up to 134%. The predictions on the shear capacity are very close to experimental value with an average, $V_{Rdf,Rilem} / V_{Rdm,exp} = 0.83$.

Table 1. Comparison between average experimental results and theoretical models [11].

Series	VRdm,exp (kN)	VRdf,RILEM (kN)	VRdf,RILEM / VRdm,exp
B-2 ($V_f = 0.26\%$)	64.3	64.3	1.00
CF20-2 ($V_f = 0.26\%$)	101.6	82.7	0.81
UN20-2 ($V_f = 0.26\%$)	100	85.0	0.85
CF20-1 ($V_f = 0.26\%$)	111.7	88.9	0.80
UN20-1 ($V_f = 0.26\%$)	145.3	89.3	0.61
Sy4.5-1 ($V_f = 0.49\%$)	150.4	89.9	0.60
Sy4.5-2 ($V_f = 0.49\%$)	111.2	90.2	0.81
CF40-2 ($V_f = 0.51\%$)	100.1	94.8	0.95
CF40-1	95.1	94.8	1.00

($V_f = 0.51\%$) UN20CF20-2	106.1	94.7	0.66
($V_f = 0.51\%$) UN20CF20-1	116.9	95.2	0.84
($V_f = 0.51\%$) UN40-2	113.9	102.8	0.88
($V_f = 0.51\%$) UN40-1	106.1	104.5	0.98
($V_f = 0.51\%$)			

2.3 Proposed Shear Design Equations

Sharma [5] performed seven tests on SFRC beams and proposed an equation to calculate the ultimate shear at failure. The proposed equation is a function of the concrete tensile strength and the shear span-to-depth ratio. The equation was validated with 41 other tests on steel fibre reinforced concrete beams. However, the equation ignored some important parameters that contributed to the shear strength, which includes fibre volume, fibre aspect ratio and tensile reinforcement ratio. The tests have shown that a combination of stirrups and fibre reinforcement forms effective shear reinforcement in structural member. The design value for the shear resistance is given by:

$$V_{fd} = k f_t' \left(\frac{d}{a} \right)^{0.25} \quad (9)$$

where $k = 2/3$, (a/d) is the shear span-to-depth ratio, f_t' is the splitting tensile strength of concrete, $f_t' = 0.79 (f_c')^{0.5}$ MPa, if the tensile strength is unknown, and f_c' is the concrete cylinder compressive strength.

Mansur *et al.* [12] proposed an equation to calculate the ultimate shear at failure and tested 24 simply supported beams under two symmetrical point loads. In the model, the effect of fibres were included by considering a uniform stress block of average stress along the tension crack, thus, provide good predictions of the ultimate strength. However, for beams with short shear spans, the predicted shear capacities were highly conservative. The design value for the shear resistance is given by:

$$V_{fd} = \left(0.16 \sqrt{f_c'} + 17.2 \rho \frac{d}{a} + 0.41 \tau_f \right) b d \quad (10)$$

where f_c is the characteristic compressive strength of concrete, ρ is the longitudinal reinforcement ratio, b and d is the width and effective depth of beam, respectively, and a is the shear span.

Narayanan and Darwish [13] proposed equations for evaluating the cracking shear strength and ultimate shear strength of fibre reinforced concrete beams. The inclusion of 1% of volume fraction steel fibres in reinforced concrete beams resulted in a substantial increase of up to 170% of the ultimate shear strength. The test was designed on 49 beams with rectangular cross section of 85 mm \times 150 mm and tested under four symmetrically placed concentrated loads. The design value for the shear resistance is given by:

$$V_{fd} = e \left[0.24 f_{spfc} + 80 \rho \frac{d}{a} \right] + v b \quad (11)$$

where f_{spfc} is the computed value of splitting tensile strength of fibre concrete, and:

$$fcuf/(20 - \sqrt{F}) + 0.7 + 1.0 \sqrt{F} \quad (12)$$

where ρ is the flexural reinforcement ratio, F is the fibre factor (L_f/D_f) $V_f d_f$, e is the arch action factors: 1.0 for $a_v/d > 2.8$ and $2.8 d/a_v$ for $a_v/d \leq 2.8$, f_{cuf} is the cube strength of fibre concrete in MPa, L_f is the fibre length, D_f is the fibre diameter (D_f bond factor is 0.5 for round fibre, 0.75 for crimped fibre and 1.0 for indented fibre), v_b is the $0.41\tau_f$, and τ_f is the average fibre matrix interfacial bond stress, taken as 4.15 MPa.

Kwak *et al.* [14] proposed two empirical equations for ultimate shear strength of SFRC deep and slender beams. The nominal stress at shear cracking and the ultimate shear strength increases with the increasing concrete compressive strength, fibre volume, and decreasing shear span-to-depth ratio. The design value for shear resistance is given by:

$$V_{fd} = 3.7 \left(\frac{fc'}{20 - \sqrt{F}} + 0.7 + \sqrt{F} \right)^{\frac{2}{3}} \left(\rho \frac{d}{a} \right)^{1/3} + 0.8(0.41\tau F) \quad (13)$$

Swamy *et al.* [15] proposed equations to predict ultimate shear strength of lightweight and normal weight concrete beams defined in Eq. (14). The method proposed by the researcher was also valid to predict the shear strength of normal weight fibre concrete beams containing steel fibres as shear reinforcement. The fibres in concrete beams increases the flexural and shear strength at closer spacing than the corresponding concrete beams without fibres.

$$V_{fd} = 0.37\tau\sqrt{f'_c} V_f \frac{L_f}{d_f} + v_c \quad (14)$$

Dinh *et al.* [1] proposed a simple model to estimate the effectiveness of the fibre as shear reinforcement in a beam without stirrup reinforcement. The use of hooked-end type steel fibres in volume fraction equal or greater than 0.75% led to multiple diagonal cracking and substantial increase in shear strength compared with reinforced concrete beams without stirrup reinforcement. The formula for the calculation of the shear capacity in a steel fibre reinforced beams is given by:

$$V_{fd} = \frac{0.13 \rho f_y}{bd} + 1.2 \left(\frac{V_f}{0.0075} \right)^{\frac{1}{4}} \left(1 - \frac{c}{d} \right), c = 0.1h \quad (15)$$

Ashour *et al.* [16] empirical equations are proposed to predict the shear strength of high strength fibre reinforced concrete beams without shear reinforcement. The researcher purposed two modifications of the ACI building Code equation and of Zutty's equation. The design value for the shear resistance is given by:

$$V_{fd} = (0.7\sqrt{f'_c} + 7F) \frac{d}{a} + 17\rho \frac{d}{a} \quad (16)$$

Yakoub [17] used an expression developed for predicting the contribution of steel fibers to the shear strength of SFRC beams.

$$V_{fd} = \beta\sqrt{f'_c} \left(1 + 0.70 V_f \frac{L_f}{D_f} d_f \right) \quad (17)$$

where $\beta = \frac{0.40}{1+1500\varepsilon_x} \frac{1300}{1000+s_{xe}}$, $\varepsilon_x = \frac{M/d_v + V}{2E_s A_s} S_{xe} = \frac{35s_x}{16 + d_a} \geq 0.85s_x$

M and V are the external failure moment and shear acting on the section, s_x = crack spacing parameter (d_v = flexural lever arm) $d_v = 0.9d$ or $d_v = 0.7h$, ϵ_x longitudinal strain at the mid depth, $d_f = 0.83$ for crimped, 0.89 for duoform, 1.00 for hooked, 0.91 for rounded fibres.

2.4 Effects of certain parameters on the predicted shear strength

The tested beams were typically designed with the following variables; steel fibre volume fraction (V_f) and shear span-to-depth ratio (a_v/d). In this study, the test results for the 67 beams were recorded based on the previously conducted shear failure test on steel fibre reinforced concrete beams and reinforced concrete beams without stirrups. In some tests, failure was caused by flexure or the combined effects of shear and flexure. Table 2 shows the summary beam details and comparison of experimental and predicted shear strength. The model proposed by Sharma, Mansur *et al.* Narayanan and Darwish, Kwak, Swamy *et al.* Dinh *et al.*, Ashour *et al.* Yakoub shows in Table 2 from 1-8. The wide range of (a_v/d) from 1.0 to 6.0 due to the different deep and slender beams. The ratio of shear at ultimate load to calculated shear strength decreases as the (a_v/d) ratio increases. Different types of hooked-end type steel fibres had been used in the experiments. An experimental program was conducted to investigate the effect of steel fibres of reinforced concrete beams subjected to loading.

Dinh *et al.* [1] studied that steel fibre can be used as a shear reinforcement in beams. A total of 25 beams were prepared and tested. All beams had shear span-to-depth ratio of $a_v/d = 3.43$ and 3.50. Dinh *et al.* [1] tested SFRC members containing 0.75%, 1% and 1.5% hooked-end type steel fiber with fibre length (L_f) to fibre diameter (D_f) ratio, $L_f/D_f = 55 - 80$. Using hooked-end type steel fibers in volume fraction $V_f \geq 0.75\%$ led to at least 100% increase in the shear strength of SFRC members compared to that of the similar reinforced concrete members without fibres.

Kwak *et al.* [14] tested 12 reinforced concrete beams up to failure to evaluate the influence of fibre volume fraction, (a_v/d), concrete compressive strength on the beam strength and its ductility. The SFRC members contained different volume fractions of hooked-end type steel fibre with aspect ratio $L_f/D_f = 60 - 75$. These studies showed that using steel fibers between 0.5% to 1.5% volume fraction generally improved the shear strength, but the extent of this improvement was highly related to the other parameters such as (a_v/d).

Ashour *et al.* [16] tested 13 fibre reinforced concrete beams subjected to combined flexure and shear. All beams were singly reinforced and without shear reinforcement. The main variables were the steel fibre content, the longitudinal steel ratio and the shear span-to-depth ratio.

Swamy *et al.* [15] conducted experimental tests on T-beams and rectangular beams. A total of 9 T-beams and 2 rectangular beams were tested in this study. All beams were simply supported over a span of 2.8 m with moment/shear ratio equal to 4.5. Steel fibre generally enhanced the shear strength, decreased crack spacing, increased deformation capacity, and altered brittle failure mode to a ductile one for SFRC members.

Steel fibre increase shear resistance by providing post-cracking diagonal tension resistance across the crack surfaces to the three mechanisms of aggregate interlock, shearing in the compression block and dowel action of the longitudinal reinforcement. This resistance by the steel fibre is called crack-bridging stress. The shear response of SFRC members without stirrups has been studied by several researchers [5,13,11].

Table 2. Summary of beam details and comparison of experimental and predicted shear strength

Researcher	Beam ID	Fibre Type	V_f (%)	a_v/d	v_{exp} (MPa)	v_{exp}/v_{pred}							
						(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Dihn et al.[1]	B 18-0a	Hooked	0	3.43	1.1	0.88	0.84	0.79	0.73	1.10	1.21	1.23	0.84
	B 18-0b		0	3.43	1.1	0.79	0.82	0.75	0.98	1.23	1.43	1.14	0.83
	B 18-1a		0.75	3.43	2.9	0.86	0.89	0.73	0.98	0.98	1.12	0.86	0.82
	B 18-1b		0.75	3.43	2.8	0.87	0.89	0.72	0.87	1.32	1.25	1.33	0.82
	B 18-2a		1	3.5	3	0.88	0.86	0.81	0.87	1.23	0.98	1.30	0.82
	B 18-2b		1	3.5	3.1	0.91	0.93	0.82	0.89	1.45	1.12	1.22	0.86
	B 18-2c		1	3.5	3.5	0.99	0.92	0.83	0.91	0.88	0.43	0.76	0.85
	B 18-2d		1	3.5	2.6	0.97	0.98	0.93	0.90	1.23	0.76	1.12	0.85
	B 18-3a		1.5	3.43	2.6	0.92	0.97	0.99	0.90	1.12	1.97	1.32	0.86
	B 18-3b		1.5	3.43	3.4	0.82	0.95	0.86	0.89	1.32	1.54	1.23	0.86
	B 18-3c		1.5	3.43	3.3	0.99	0.97	0.88	0.89	1.03	1.21	0.89	0.87
	B 18-3d		1.5	3.43	3.3	0.82	0.94	0.90	1.11	1.12	1.32	0.93	0.87
	B 18-5a		1	3.43	3	0.83	0.94	0.90	1.11	1.01	0.99	0.89	0.87
	B 18-5b		1	3.43	3.8	0.83	0.93	0.85	1.07	0.71	1.12	0.89	0.81
	B 18-7a		0.75	3.43	3.3	0.83	0.93	0.85	1.71	1.68	1.32	1.32	0.81
	B 18-7b		0.75	3.43	3.3	0.83	0.94	0.87	0.91	1.74	0.96	1.51	0.81
	B 27-1a		0.75	3.5	2.9	0.83	0.94	0.87	0.91	1.77	1.12	1.52	0.81
	B 27-1b		0.75	3.5	2.7	0.91	1.10	0.94	0.92	1.25	1.02	1.54	0.81
	B 27-2a		0.75	3.5	2.8	0.92	1.10	0.94	0.75	1.27	1.12	1.22	0.81
	B 27-2b		0.75	3.5	2.8	0.92	0.89	0.88	0.98	1.37	1.15	1.21	0.80
B 27-3a	0.75	3.5	2.7	0.92	0.89	0.88	0.98	1.32	1.21	1.24	0.80		
B 27-3b	0.75	3.5	2.8	0.92	0.94	0.91	1.12	1.32	1.39	1.13	0.78		
B 27-4a	0.75	3.5	2.1	0.92	0.94	0.91	1.12	1.46	1.40	1.48	0.78		
B 27-4b	0.75	3.5	1.8	0.92	1.15	0.92	0.95	0.93	0.81	0.97	0.72		
Ashour et al.[16]	B-1-0.5-A	Hooked	0.5	1.0	9.09	0.73	0.93	0.81	1.12	2.12	1.75	1.13	0.97
	B-2-0.5-A		0.5	2.0	4.82	1.15	1.21	0.86	1.32	0.89	1.16	0.98	1.12
	B-4-0.5-A		0.5	4.0	2.27	0.89	0.94	0.78	0.85	1.02	1.43	1.07	0.92
	B-6-0.5-A		0.5	6.0	1.95	0.72	0.93	0.79	0.71	1.51	1.08	1.10	0.86
	B-1-1.0-A		1.0	1.0	12.74	1.12	0.97	0.89	1.47	0.96	0.99	1.02	1.12
	B-2-1.0-A		1.0	2.0	6.06	1.23	1.13	0.92	1.41	1.51	2.32	1.19	1.13
	B-4-1.0-A		1.0	4.0	3.17	0.97	1.18	0.89	0.98	1.22	1.63	1.02	0.99
	B-1-1.5-A		1.5	1.0	13.95	1.05	0.88	0.95	1.33	1.13	1.58	0.99	1.03
	B-2-1.5-A		1.5	2.0	7.21	1.29	1.15	1.09	1.44	0.71	1.05	1.12	1.12
	B-4-1.5-A		1.5	4.0	3.51	0.90	1.02	0.81	0.89	0.78	1.08	1.05	0.99
B-2-1.0-M	1.0	2.0	6.73	1.13	1.17	0.99	1.45	1.21	1.49	1.04	1.12		
B-4-1.0-M	1.0	4.0	3.88	1.04	1.14	0.97	1.12	1.05	1.51	0.98	1.20		
B-6-1.0-M	1.0	6.0	2.93	0.90	1.05	0.82	0.87	1.05	1.54	1.03	0.91		
Kwak et al.[14]	FHB1-2	Hooked	0	2	3.02	1.23	1.32	0.85	1.35	1.05	0.72	0.76	1.12
	FHB1-3		0	3	2.53	1.71	1.67	1.22	1.34	0.96	1.10	1.01	1.42
	FHB1-4		0	4	1.98	1.52	1.41	1.28	1.06	0.83	1.22	1.08	1.12
	FHB2-2		0.5	2	5.09	1.81	1.62	1.32	1.87	1.12	1.35	0.96	1.23
	FHB2-2		0.5	2	4.04	1.81	1.46	1.48	2.004	1.12	1.32	0.96	1.33
	FHB3-2		0.75	2	5.44	1.72	1.45	1.25	1.52	1.24	1.45	1.23	1.25
	FHB2-3		0.5	3	3.09	1.31	1.65	1.23	1.11	1.12	1.64	1.17	1.32
	FHB2-3		0.5	3	2.55	1.51	1.65	1.49	1.52	1.08	1.24	0.93	1.43
	FHB3-3		0.75	3	3.40	1.37	1.56	1.25	1.23	1.24	1.65	1.23	1.31
	FHB2-4		0.5	4	2.41	1.23	1.34	1.16	1.07	1.14	1.52	1.10	1.10
	FHB2-4		0.5	4	2.00	1.38	1.34	1.31	1.23	1.25	1.43	1.15	1.12
FHB3-4	0.75	4	2.74	1.11	1.34	1.17	1.12	1.14	1.25	0.89	1.09		
1TL-1	1TL-1		0	2	3.37	0.98	1.34	0.93	1.95	1.65	1.86	1.43	1.42
	2TL-1		0	2	2.49	0.96	1.15	0.85	1.46	1.24	1.32	1.23	1.17
	3TL-1		0	2	2.15	1.14	1.24	0.82	1.24	0.95	1.12	0.97	0.94

	1TL-2	0	3.4	1.30	0.89	0.85	0.76	0.91	1.35	0.97	1.01	1.04
	2TL-2	0	3.4	1.22	0.99	0.90	0.84	0.93	1.23	0.95	1.00	1.01
	3TL-2	0	3.4	1.02	1.02	0.99	0.85	0.81	1.17	0.76	0.89	0.98
	1TL-3	0	4.9	1.19	0.97	0.95	0.74	0.93	1.35	0.85	0.97	0.92
Swamy et al.[15]	2TL-3	0	4.9	1.06	0.92	0.91	0.74	0.86	0.87	0.73	0.85	0.91
	3TL-3	0	4.9	1.03	1.16	1.12	0.96	0.86	0.87	0.75	0.84	0.91
	1TLF-1	1	2	5.48	1.02	0.98	1.18	1.91	1.13	1.32	1.12	1.12
	2TLF-1	1	2	4.91	1.13	0.95	1.02	1.51	1.58	1.51	1.01	0.96
	3TLF-1	1	2	4.63	1.25	1.12	1.15	1.45	1.48	1.42	0.96	0.97
	1TLF-2	1	3.4	4.03	1.21	1.28	1.25	1.54	1.53	1.52	1.23	1.23
	2TLF-2	1	3.4	3.11	1.07	1.11	1.11	1.36	1.17	1.24	1.12	1.11
	3TLF-2	1	3.4	2.83	1.05	1.25	1.05	1.21	1.10	1.13	0.97	1.01
	1TLF-3	1	4.9	2.90	0.92	1.02	0.96	1.27	1.11	1.13	1.05	1.00
	2TLF-3	1	4.9	2.92	1.09	1.14	1.04	1.24	1.04	1.12	1.08	1.07
	3TLF-3	1	4.9	2.01	0.76	1.05	0.91	0.96	0.89	0.89	0.81	0.87

3. Conclusion

Existing empirical equations for shear strength of concrete presented in the literature and RILEM Model were used and then compared to that value obtained from the test by previous researchers. The proposed model was used to predict the shear strength of steel fibre reinforced concrete beams. The safety margins obtained as V_{test}/V_{pred} (the shear test value divided by the theoretical shear value) were used as a reference to compare the results obtained from the different beams. Comparison between the test results and theoretical shear capacity shows that all equations conservatively estimate the occurrence of shear failure with the value range from 0.59 to 1.96. The model proposed by Sharma provided the most conservative results. The average safety factor was 1.05.

Steel fibres used in addition to conventional shear reinforcement increases the shear strength significantly. Steel fibre reinforced concrete is characterized by its enhanced toughness due to the bridging effects provided by steel fibre. Steel fibre provides substantial post-peak resistance and ductility. Shear strength behavior depended on the fibre type, volume fraction and shear span-to-depth ratio. The shear capacity of the steel fibre reinforced concrete beams increased with an increase in the tensile-reinforcement ratio and the ultimate compressive strength.

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