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Investigation into shear properties of medium strength reinforced concrete beams

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Abstract. The shear contribution of transverse steel in reinforced concrete beams is generally assumed as independent of the concrete strength by most of the building codes. The shear strength of RC beams with web reinforcement is worked out by adding the individual contributions of concrete and stirrups. In this research 70 beams of medium strength concrete in the range of 52-54 MPa, compressive strength were tested in two sets of 35 beams each. In one set of 35 beams no web reinforcement was used, whereas in second set of 35 beams web reinforcement was used to check the contribution of stirrups. The values have also been compared with the provisions of ACI, Eurocode and Japanese Code building codes. The results of two sets of beams, when compared mutually and provisions of the building codes, showed that the shear strength of beams has been increased with the addition of stirrups for all the beams, but the increase is non uniform and irregular. The comparison of observed values with the provisions of selected codes has shown that EC-02 is relatively less conservative for low values of longitudinal steel, whereas ACI-318 overestimates the shear strength of RC beams at higher values of longitudinal steel. The Japanese code of JSCE has given relatively good results for the beams studied.

Keywords: shear; transverse; building codes; stirrups.

1. Shear strength of reinforced concrete beams

The shear strength of Reinforced Concrete (RC) beams has been researched for more than five decades; however the research in this area is still underway. In one of the pioneering works on shear, Kani (1966) explained some of the basic facts leading to diagonal failure of RC beams. The main difficulty in perceiving the shear failure of RC beams is the fact that a large number of parameters influence the shear strength of RC beams. Kani (1966) identified parameters like compressive strength of concrete, strength of steel, longitudinal steel ratio, geometry of beams, width of beam web, effective depth of steel, shear arm, web reinforcement and type of loading etc.

The shear behavior of RC beams with and without transverse steel is different.

The research on shear strength of concrete has shown that reinforced concrete beams without transverse reinforcement can resist the shear and flexure by means of beam and arch actions, also sometimes called concrete mechanisms (Russo *et al*. 2004).

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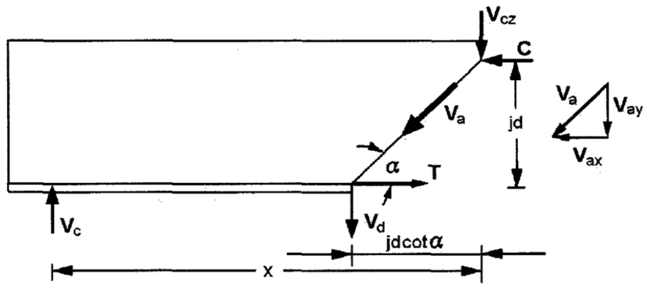


Fig. 1 Forces acting in a beam element within the shear span and internal arches in a RC beam (Russo *et al.* 2004)

The forces acting on the beam element in its shear span are shown in Fig. 1.

It was assumed that the resultant of the aggregates interlocking at the crack interface can be replaced by *Va* as shown in the Fig. 1 above, whose direction passes through the point of application of the internal compression force *C*. The shear contribution due to dowel *Vd* is negligible at the rotation equilibrium. The resultant bending moment is given by

$$Mc = V_c \cdot x = T \cdot jd \tag{1}$$

Where *Vc* is the shear force due to concrete resisting contribution, *T* is tensile force in the longitudinal reinforcement and *x* is the distance between the support and the point where crack has been appeared.

The shear force is the derivative of the bending moment $Vc = dMc/dx$

$$Vc = jd\left[\frac{d}{dx}T\right] + T \cdot \frac{d}{dx}jd \tag{2}$$

The first term in Eq. (2), is the resistance to shear as contribution of the beam action, whereas the second part is called arch action. The distinction of beam action and arch action leading to the failure of RC beams in shear was also given by Kani in his earliest work in 1964.

The joint committee ASCE-ACI-426 (1973) and later in 1998 reported the following five mechanisms for the shear in reinforced concrete sections;

i. Shear in the Un-cracked Concrete Zone:
ii. Residual Tensile Stresses in the cracked section
iii. Interface shear transfer or aggregates interlocking.
iv. Dowel Action of the longitudinal bars.

The ASCE-ACI Committee 426 (1973), has reported the following equation for the shear strength of RC beams without web reinforcement.

$$v_c = (0.80 + 100\rho)(\sqrt{fc'}/12) \le 0.192(\sqrt{fc'}/12) \text{ [MPa]} \tag{3}$$

For beams with transverse reinforcement, the basic model to explain the mechanism for carrying the shear was proposed by Ritter (1899). The load was assumed to flow down the concrete diagonal struts and then lifted to the compression chord by transverse tension ties on its way to support as

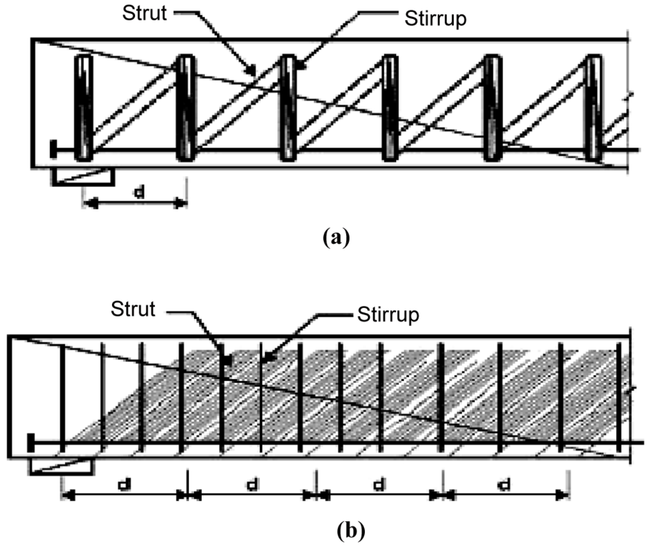


Fig. 2 (a) Parallel chord truss model of beam, (b) The struts are intercepted by the stirrups at spacing of *d* (NCHRP report 2005)

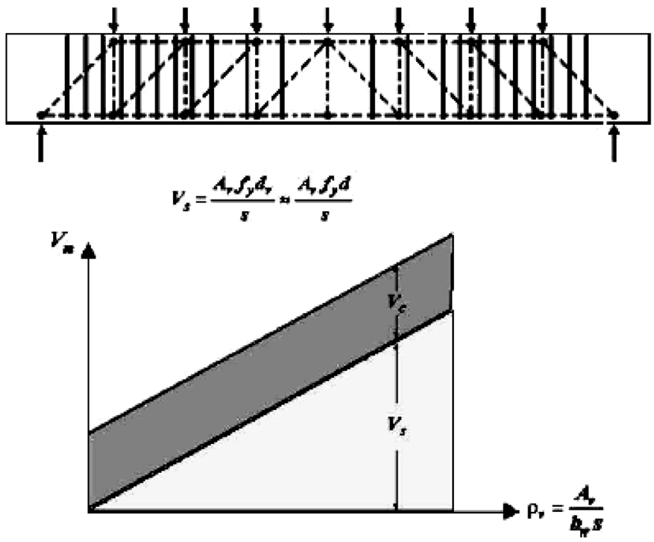


Fig. 3 Shear strength of RC beams with shear reinforcement (NCHRP report 2005)

shown in Fig. 2.

The shear strength of RC beams with transverse reinforcement is generally determined by summing the individual contributions of concrete and steel as shown in Fig. 3. i.e.

$$V_n = V_c + V_s \tag{4}$$

Kani (1969) explained that the structural function of web steel is to produce supports to the internal arches, rather than to resist the shear force. This was in sharp contrast to the earlier knowledge about the shear failure of RC beams.

Kotsovos (1983, 1986) proposed the concept of Compressive Force Path to explain the causes and mechanisms of shear failure of RC beams. He also used Finite Element Analysis (FEA) for the analysis of RC beams with web reinforcement and showed that the shear failure is associated with the development of tensile stresses in the compression zone, which leads to the failure of bond between concrete and tension reinforcement.

According to European Code EC2-2003, the shear strength for RC members without shear reinforcement is given as

$$V_c = [\tau_R \kappa (1.2 + 40\rho)] b_w d \tag{6}$$

Where $\tau_R = 0.0525 fc'$

$\kappa = 1.6 - d \geq 1.0$

$\rho = As / b_w d \leq 0.02$

For beams with shear reinforcement, the shear capacity of beams is given as

$$V_s = 0.9 \rho_v f_{yv} b_w d \tag{7}$$

According to Japanese Society of Civil Engineers (JSCE) Code (1986) the shear capacity of reinforced members is given as

$$V_{cd} = \beta_d \beta_p \beta_n f_{vcd} b_w d / \gamma_b$$

where $\gamma = 1.3$ for $a/d \geq 2$

$$f_{vcd} = 0.2 f'_{cd}{}^{1/3} \leq 0.72 \tag{8}$$

$\beta_d = (1000/d)^{1/4} \leq 1.5$

$\beta_p = [0.75 + 1.4(a/d)]$

$\beta_d = (100\rho_w)^{1/3}$

$\beta_n = [0.75 + 1.4(a/d)]$

For simply supported beams $\beta_n = 1$

Since we are comparing the test values directly with the code values therefore $\gamma = 1.0$, no reduction factor shall be applied.

$$V_s = \rho_v f_{yv} b_w d \tag{9}$$

The research on the contribution of web reinforcement in shear strength of RC beams is still an active area and various empirical equations have been proposed by researchers, on the basis of experimental results.

Zarari (2003) proposed the following equation for the contribution of transverse reinforcement.

$$V_s = \left(0.50 + 0.25\frac{a}{d}\right)\rho_v f_{yv} b d \tag{10}$$

Arsalan (2007) developed the following equation for predicting the diagonal cracking shear strength of beams without stuirrups.

$$v_{cr} = 0.15(f_c)^{0.50} + 0.02(f_c)^{0.65} \tag{11}$$

For Normal Strength Concrete and

$$v_{cr} = 0.12(f_c)^{0.65} \tag{12}$$

For High Strength Concrete

Arsalan (2008), further proposed the following expressions for the shear strength of RC beams with stirrups

$$v_n = 0.15(f_c)^{0.50} + 0.02(f_c)^{0.65} + \rho_w f_w \tag{13}$$

For normal Strength concrtete beams

$$v_n = 0.12(f_c)^{0.50} + 0.02(f_c)^{0.65} + \rho_w f_w \tag{14}$$

For High Strength Concrete beams

Tompos and Frosch (2002) studied the effect of various parameters like beam size, longitudinal steel abd stirrups and reported that the current shear design provisons of ACI are based on database of the beams sizes, not commonly used sizes in actual practice. They further reported that for longitudinal steel of 1% or low, the shear strength of beams has been reduced for all sizes of beam.

Cladera and Mari (2004), developed an Artificial Neural Network (ANN) to predict the shear strength of RC beams, using a large database of experimental results and made the following important conclusions:

1. The influence of the amount of web reinforcement is not linearly proportional to the amount of web reinforcement. i.e., the shear strength due to increase in shear reinforcement is not increasing in the same ratio. The effectiveness of stirrups decreases with their increase.
2. Due to increase in size at low shear reinforcement, the shear strength has been reduced by 25% when the size of beam has been increased from 250 to 750 mm.
3. The influence of compressive strength of concrete also changes with the amount of web reinforcement.
4. AASHTO LRFD design equation gives relatively good results as compared with the ACI-318 and Eurocode-2.

Cladera and Mari (2005) worked on the HSC beams failing in shear and reported a very brittle failure of the HSRC beams without shear reinforcement. The failure was observed as more sudden with further increase in the strength of concrete. They also concluded that the limitation of 2% longitudinal steel for HSC beams with web reinforcement is also not justified.

Sarkar *et al.* (1999) studied the contribution of the compression zone concrete υcz, aggregate interlocking υa and dowel action of the longitudinal steel υd to the shear capacity for high strength reinforced concrete (HSRC) beams without transverse reinforcement. The research was carried out on beams with compressive strength ranging from 40 MPa to 110 MPa. The following inferences were made;

i. The role of aggregate interlocking mechanism at higher concrete strengths is slightly enhanced. In addition, this mechanism had a predominant influence on the ultimate load carried by the beam. In other words, the contribution of this mechanism to the total shear strength carried by the beam was around 42% for higher concrete strength beams with compressive strength of 110 MPa as compared to 34% for NSC of 40 Mpa. However this increase in the shear contribution due to aggregates interlocking is much less than the increase in the compressive strength of concrete.

ii. The contribution of the compression zone concrete remained fairly constant at higher compressive strength with very little increase from 13% to 17% with the increase of compressive strength from 40 Mpa to 110 Mpa.

iii. The contribution of dowel action remained the main part in the absence of the aggregate interlocking but it decreased from 53% to 43% with the increase of concrete strength.

Londhe (2009) proposed a simplified expression for the shear strength of RC without web reinforcement incorporating variables such as compressive strength of concrete, percentage of longitudinal and vertical steel, shear span to depth ratio etc and based on the comparison with the provisions of IS:456-2000, BS-8110-1997, ACI318-02, it was proposed to revise the procedures for the shear design given by these codes as these codes are less conservative for shear design of RC members.

In this research, seventy beams have been tested in this research in two sets of 35 beams each. In one set of 35 beams no web reinforcement was used, whereas in 35 beams of other set, web reinforcement corresponding to minimum value proposed by ACI has been used. The test results and beam failure mechanism for both of beams has been observed. The contribution of stirrups has also been investigated and is compared with the provision of ACI-318, Eurocde-02, and Japanese Code.

2. Research objectives and significance

The research objectives can be summarized as follows:

i. To check the failure modes of RC beams.
ii. To check the effect of stirrups on the failure modes of beams.
iii. To check the increase in the shear strength of concrete with the addition of transverse steel.
iv. To compare the results obtained with the provisions of ACI-318, EC-02.
v. To check the whether the assumption of summing the individual concrete and stirrups contribution for the combined shear capacity of reinforced concrete beams with transverse reinforcement is experimentally proved or not?

3. Material

Deformed steel of 60 grade (410 MPa) have been used as the main reinforcement and for transverse steel, 40 grade (276 MPa) were used.

All beams were cast from the same mix design of concrete. The coarse aggregates of lime stone source were used with ¾ in (19 mm) and below sizes. The fine aggregates with modulus of fineness as 2.67 were used in the concrete. The High Range water Reducers Conforming to ASTM C-494 type F standards was used at 1.70% by weight of cement to control the water cement ratio. The design strength of 7500 psi (51.71 MPa) was used, however the average concrete strength was

Table 1 Mix proportioning/designing of high strength concrete

Constituent	Proportion
Type-I Cement	1058 lbs/yd^3 (628 kg/m^3)
Fine aggregates	815 lbs/yd^3 (484 kg/m^3)
Coarse aggregates	1900 lbs/yd^3 (1128 kg/m^3)
HRWR @ 1.7% by weight of cement	18 lbs/yd^3 (10.70 kg/m^3)
Water @ 0.25 *w/c* ratio	164.5 lbs/yd^3 (97.65 kg/m^3)
Design Compressive strength (28 days)	7500 psi (51.71 MPa)
Actual Average strength (28 days)	8200 psi (56.54 MPa)

achieved as 8200 psi (56.54 MPa). The indidividal strength variation of specimen remained within the prescribed ACI limits of 500 psi (3.5 MPa). For each *a/d* ratio, the pouring of concrete was done at the same time in all five beams with ρ =0.333%, 0.73%, 1%, 1.5% and 2.0%, to have better control over compressive strength. Thus the pouring was completed in seven sets. The details of Mix design of Concrete are given in Table 1.

4. Experimental investigations

Two series of beams comprising 35 beams each of size 23 cm × 30 cm were selected such that in series-I, no web reinforcement was used, whereas in series-II minimum web reinforcement as per ACI-318 was used to ensure shear failure of the beams.

In each series of 35 beams, five values of longitudinal steel ratio was used as ρ = 0.33%, 0.75%, 1%, 1.5% and 2%. The beams with no web reinforcement are designed as B1, B2, B3, B4 and B5, whereas beams with web reinforcement have been denoted as Bs1, Bs2, Bs3, Bs4 and Bs5, respectively. The detail of beams and reinforcement is given in Table 2

Table 2 Reinforcement details of beams

Beam Title	Main Steel		Beam Title	Transverse steel for beam with web reinforcement	
	Bars	ρ%		Stirrups	ρv (%)
B1	1#10+1#13 (1#3+1#4)	0.33	Bs1	#6@15 cm (#2@6 in)	0.16
B2	2#10+2#13 (2#4+2#3)	0.73	Bs2	#6@15 cm (#2@6 in)	0.16
B3	2#19 (2#6)	1.00	Bs3	#6@15 cm (#2@6 in)	0.16
B4	3#19 (3#6)	1.50	Bs4	#6@15 cm (#2@6 in)	0.16
B5	2#22 (2#7)	2.00	Bs5	#6@15 cm (#3@6 in)	0.16

(SI units of bar # given in Metric sizes and corresponding US sizes given in parenthesis)
$\rho = A_s/bd$, Where As = Area of steel bars in the cross section of the beam, b is width of the beam = 23 cm, d is effective depth of the beam = 30 cm.

Table 3 Shear span to depth ratio and corresponding clear span of seven beams in each set of longitudinal reinforcement

a/d	3.0	3.5	4.0	4.5	5.0	5.5	6.0
Clear span of beams (cm)	150	178	203	229	254	280	305

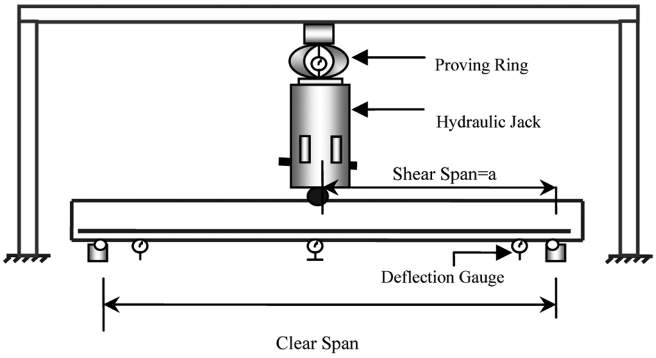


Fig. 4 Typical loading arrangement for testing of beams

Testing set up

The beams were tested by applying monotonic concentrated load at the mid span.

For each set of 5 longitudinal reinforcement values, seven values of shear span to effective depth (*a/d*) ratio of 3.0, 3.5, 4.0, 4.5, 5.0, 5.5 and 6.0 were selected, mainly to study the behavior of slender beams (*a/d* $\geq$ 3), where typical shear failure can be anticipated. The corresponding clear span for particular value of *a/d i*s given in Table 3.

The loads were gradually applied through hydraulic system and the reading from the gauge of proving ring was taken manually. The corresponding equivalent load in kN was later determined from the conversion table provided by the manufacturer of the proving ring.

The schematic diagram of loading arrangement is shown in Fig. 4.

5. Observations

Loads were applied at uniform rate of about 5 kN per second, so that the application of load is gradual and monotonic. The readings of the calibrated proving rings were taken after every increment of 5 kN, the beams were continuously observed from both sides. The deflection of the gauges placed under the mid span and critical sections for shear were taken for each 5 kN increment of load. The cracks appearing in the beams were carefully observed and the corresponding load applied was recorded at the point, where the crack initiated. The crack path and the respective loading were also marked on both sides of every beam. The application of loading was continued till the failure of the beams.

The nominal shear strength of beams is taken as half of the total load carried by the beams at the failure point, as the beams are simply supported. The total load taken by the beams includes the external load applied and self weight of the beams. The shear carried by 35 beams without web reinforcement and failure angles are given in Table 4. For beams with web reinforcement, the shear strength and failure angles have been given in Table 5.

Table 4 Shear strength and failure angles of beams, without web reinforcement

Beam Title	*fc'* (MPa)	Steel ratio (ρ)	*a/d*	Shear taken by the beam at the failure $V_{test} = P_u$(test)/2 (kN)	Approx. Failure angle (degrees)
$B_{0.33,3}$	55.25	0.33	3.0	35.24	60-70
$B_{0.33.3.5}$	57.23		3.5	30.27	
$B_{0.33,4}$	56.13		4.0	25.11	
$B_{0.33,4.5}$	55.19		4.5	23.92	
$B_{0.33,5}$	56.78		5.0	21.06	
$B_{0.33,5.5}$	54.21		5.5	18.88	75-80
$B_{0.33,6}$	56.81		6.0	16.04	
$B_{0.73,3}$	55.25	0.73	3.0	61.44	55-65
$B_{0.73.3.5}$	57.23		3.5	56.72	
$B_{0.73,4}$	56.13		4.0	51.74	
$B_{0.73,4.5}$	55.19		4.5	46.78	
$B_{0.73,5}$	56.78		5.0	42.01	
$B_{0.33,5.5}$	54.21		5.5	36.97	75-80
$B_{0.73,6}$	56.81		6.0	26.74	
$B_{1,3}$	55.25	1.0	3.0	79.02	40-55
$B_{1,3.5}$	57.23		3.5	67.96	
$B_{1,4}$	56.13		4.0	60.36	45-65
$B_{1,4.5}$	55.19		4.5	57.36	
$B_{1,5}$	56.78		5.0	50.69	
$B_{1,5.5}$	54.21		5.5	49.76	
$B_{1,6}$	56.81		6.0	38.46	
$B_{1.5,3}$	55.25	1.50	3.0	115.69	35-50
$B_{1.5.3.5}$	57.23		3.5	103.31	
$B_{1.5,4}$	56.13		4.0	89.58	
$B_{1.5,4.5}$	55.19		4.5	79.58	40-60
$B_{1.5,5}$	56.78		5.0	69.53	
$B_{1.5,5.5}$	54.21		5.5	62.52	
$B_{1.5,6}$	56.81		6.0	55.13	
$B_{2,3}$	55.25	2.0	3.0	147.69	30-50
$B_{2,.3.5}$	57.23		3.5	123.98	
$B_{2,4}$	56.13		4.0	101.61	
$B_{2,4.5}$	55.19		4.5	95.75	35-60
$B_{2,5}$	56.78		5.0	85.68	
$B_{2,5.5}$	54.21		5.5	76.81	
$B_{2,6.0}$	56.81		6.0	69.64	

$B_{0.33,3}$ stands for beam without web reinforcement, having longitudinal steel of 0.33% and *a/d* = 3

Table 5 Shear strength and failure angles of 35 beam with web reinforcement

Beam Title	fc' (MPa)	Steel ratio (ρ)	a/d	Shear taken by the beam at the failure $V_{test} = P_u$(test)/2 (kN)	App. Failure angle. (degrees)
$Bs_{0.33,3}$	55.25	0.33	3.0	40.18	60-55
$Bs_{0.33.3.5}$	57.23		3.5	36.99	
$B_{0.33,4}$	56.13		4.0	31.90	
$Bs_{0.33,4.5}$	55.19		4.5	34.42	
$B_{0.33,5}$	56.78		5.0	31.58	
$Bs_{0.33,5.5}$	54.21		5.5	24.47	70-80
$Bs_{0.33,6}$	56.81		6.0	21.79	
$Bs_{0.73,3}$	55.25	0.73	3.0	81.77	50-65
$Bs_{0.73.3.5}$	57.23		3.5	77.70	
$B_{0.73,4}$	56.13		4.0	67.27	
$Bs_{0.73,4.5}$	55.19		4.5	62.60	
$Bs_{0.73,5}$	56.78		5.0	57.68	
$Bs_{0.33,5.5}$	54.21		5.5	53.01	70-80
$Bs_{0.73,6}$	56.81		6.0	48.11	
$Bs_{1,3}$	55.25	1.0	3.0	95.69	30-50
$Bs_{1,3.5}$	57.23		3.5	84.81	
$Bs_{1,4}$	56.13		4.0	78.64	40-65
$Bs_{1,4.5}$	55.19		4.5	77.53	
$Bs_{1,5}$	56.78		5.0	72.92	
$Bs_{1,5.5}$	54.21		5.5	64.76	
$Bs_{1,6}$	56.81		6.0	52.91	
$Bs_{1.5,3}$	55.25	1.50	3.0	125.18	30-45
$Bs_{1.5.3.5}$	57.23		3.5	116.10	
$Bs_{1.5,4}$	56.13		4.0	95.92	
$B_{1.5,4.5}$	55.19		4.5	82.21	35-60
$Bs_{1.5,5}$	56.78		5.0	71.84	
$Bs_{1.5,5.5}$	54.21		5.5	65.93	
$Bs_{1.5,6}$	56.81		6.0	58.76	
$Bs_{2,3}$	55.25	2.0	3.0	160.54	30-50
$Bs_{2,.3.5}$	57.23		3.5	135.31	
$Bs_{2,4}$	56.13		4.0	115.98	
$Bs_{2,4.5}$	55.19		4.5	112.66	
$Bs_{2,5}$	56.78		5.0	99.37	
$Bs_{2,5.5}$	54.21		5.5	95.03	
$Bs_{2,6}$	56.81		6.0	77.77	

Table 6 Increase in shear strength with the addition of transverse reinforcement

Steel ratio (ρ)	*a/d*	Shear Strength (kN)		Increase in shear strength
		Beams with No web steel	Beams with web steel	
0.33	3.0	35.24	40.18	4.94
	3.5	30.27	36.99	6.72
	4.0	25.11	31.90	6.79
	4.5	23.92	34.42	10.50
	5.0	21.06	31.58	10.52
	5.5	18.88	24.47	5.59
	6.0	16.04	21.79	5.75
			Average	7.30
0.73	3.0	61.44	81.77	20.33
	3.5	56.72	77.70	20.98
	4.0	51.74	67.27	15.53
	4.5	46.78	62.60	15.82
	5.0	42.01	57.68	15.67
	5.5	36.97	53.01	16.04
	6.0	26.74	48.11	21.37
			Average	17.96
1.0	3.0	79.02	95.69	16.67
	3.5	67.96	84.81	16.85
	4.0	60.36	78.64	18.28
	4.5	57.36	77.53	20.17
	5.0	50.69	72.92	22.23
	5.5	49.76	64.76	15.00
	6.0	38.46	52.91	14.45
			Average	17.66
1.50	3.0	115.69	125.18	9.49
	3.5	103.31	116.10	12.79
	4.0	89.58	95.92	6.34
	4.5	79.58	82.21	2.63
	5.0	69.53	71.84	2.31
	5.5	62.52	65.93	3.41
	6.0	55.13	58.76	3.63
			Average	5.80
2.0	3.0	147.69	160.54	12.85
	3.5	123.98	135.31	11.33
	4.0	101.61	115.98	14.37
	4.5	95.75	112.66	16.91
	5.0	85.68	99.37	13.69
	5.5	76.81	95.03	18.22
	6.0	69.64	77.77	8.13
			Average	13.64

Table 7 Comparison of Vtest/VCode values for beams without web reinforcement

ρ (%)	*a/d*	ACI	EC-02	JSCE
1	3	1.29	1.07	1.27
1	3.5	1.12	0.92	1.09
1	4	1	0.82	0.97
1	4.5	0.96	0.77	0.92
1	5	0.85	0.68	0.81
1	5.5	0.83	0.67	0.80
1	6	0.65	0.52	0.62
1.5	3	1.85	1.39	1.63
1.5	3.5	1.67	1.24	1.45
1.5	4	1.46	1.08	1.26
1.5	4.5	1.3	0.96	1.12
1.5	5	1.15	0.83	0.98
1.5	5.5	1.03	0.75	0.88
1.5	6	0.84	0.66	0.77
2	3	2.13	1.6	1.89
2	3.5	1.81	1.34	1.59
2	4	1.5	1.1	1.30
2	4.5	1.42	1.03	1.22
2	5	1.28	0.93	1.10
2	5.5	1.15	0.83	0.98
2	6	1.05	0.75	0.89
Mean		1.25	0.95	1.12
CoV		0.13	0.07	0.09

Table 8 Comparison of Vtest/VCode values for beams with web reinforcement

ρ (%)	*a/d*	ACI	Euro code	JSCE
1	3	1.33	1.10	1.29
1	3.5	1.15	0.98	1.10
1	4	1.01	0.91	1.00
1	4.5	0.96	0.90	0.99
1	5	0.87	0.84	0.97
1	5.5	0.84	0.81	0.91
1	6	0.63	0.61	0.80
1.5	3	1.95	1.51	1.71
1.5	3.5	1.73	1.33	1.50
1.5	4	1.51	1.16	1.31
1.5	4.5	1.35	1.02	1.15
1.5	5	1.18	0.89	1.09
1.5	5.5	1.06	0.81	1.02
1.5	6	0.84	0.72	1.01
2	3	2.26	1.58	2.10
2	3.5	1.89	1.30	1.61
2	4	1.56	1.13	1.41
2	4.5	1.47	1.04	1.26
2	5	1.31	0.91	1.15
2	5.5	1.18	0.79	1.10
2	6	1.07	0.72	1.01
Mean		1.29	1.00	1.21
CoV		0.16	0.06	0.09

The increase in the shear strength with the addition of web reinforcement has been given in Table 6.

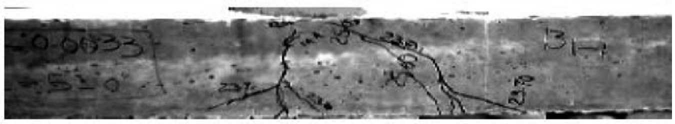
The shear failure of beams is more prominent for longitudinal steel ratio of 1% or more. The comparison of observed values with the values proposed by provisions of ACI-318, EC-02 and Japanese Codes for beams with and without web reinforcement is given in Table 7 and Table 8 respectively.

5. Discussion of results

5.1 Cracking pattern of beams

The cracking pattern and failure mode of the beams was closely observed. When loads were applied to beams without web reinforcement vertical cracks appeared in the mid span region. Initially the cracks were of small width and concentrated in the mid span region with angles being more or less vertical. However with further increase of load, the depth and width of cracks increased. The angles of cracks became shallower and turned diagonal. The change in the angle of cracks can be attributed to the cantilever action of the cracked concrete restrained by the longitudinal reinforcement in the tension zone. When load was further increased, depth of some of the diagonal cracks further enhanced and crossed into the compression zone of the beams, which ultimately caused the failure of the beams as the cracks extended further towards the point of application of loads. The typical shear failure of beams has been shown in Fig. 5. This kind of failure is also called "diagonal tension" failure, which was observed in the beams having a/d = 3 and more. For beams having $a/d > 5$, the failure has been observed predominantly due to flexural cracks, which are also called the shear flexure failure as shown in Fig. 6. Here the flexural cracks are dominant in the middle third region and the angle of failure is large. These represent the values of a/d, where the beams are about to achieve the flexural strength before shear failure on the upper boundary of famous "Kani's shear valley". The theoretical flexural values and shear strength of such beams are falling very closer to each other in this region.

The failure of beams without web reinforcement and longitudinal steel of 1% or more has been observed due to shear failure as shown in Fig. 7. However the pattern of shear crack has been changed with the increase of longitudinal steel and a/d ratio. At lower values of a/d (3, 3.5 and 4.0), the failure is more like a pure shear crack, in the form of arch action compression failure. The cracks originate closer to the supports and gradually extend towards the mid span at relatively shallower angle, in the range of 40 to 50 degrees. When the crack further extends to the mid span and a clearer shear crack observed starting from the region near the supports and reaching at the mid span to crack the beam, across a well defined path. This kind of failure is more typical for high strength concrete beams, particularly, where the longitudinal steel is more. This brittle failure phenomenon of the HSC beam must be surely a point of concern in the contemporary research.



($B_{0.33,3,}$ ρ = 0.33 %, a/d = 3.00 , span = 152 cm)

($B_{073,3:}$ ρ = 0.73 % , a/d = 3 span = 152 cm)

($B_{1.0,3,}$ ρ = 1 %, a/d = 3 span = 152 cm)

Fig. 5 Typical shear failure of beams without web reinforcement due to diagonal shear failure

($B_{0.33,5.5}$, ρ = 0.33 % , a/d = 5.5 span = 279.4 cm)

($B_{0.73,5.5}$ ρ = 0.73 % , a/d = 5.5 span = 279.4 cm)

($B_{0.73,6.0}$ ρ = 0.73% , a/d = 6.0 span = 279.4 cm)

Fig. 6 Flexural shear failures of beams without web reinforcement for *a/d* > 5

($B_{1.0,5}$, ρ = 1% , a/d = 5 span = 254 cm)

($B_{1.5,5.5}$, ρ = 1% , a/d = 5.5 span = 280 cm)

($B_{2,6}$$\rho$ = 2% , a/d = 6 span = 305 cm)

Fig. 7 Typical shear failures of beams without web reinforcement. The failure is more brittle and sudden amongst all

For beams with web reinforcement, the cracking pattern has been considerably affected by the addition of stirrups in beams. The number of cracks has been increased but their widths have been decreased. The failure angles have also been reduced.

5.2 Effect of longitudinal steel on the shear strength of beams

When the longitudinal steel ratio has been increased, the shear strength of beams in both sets has been increased. However this increase in more in case of beams with web reinforcement. The

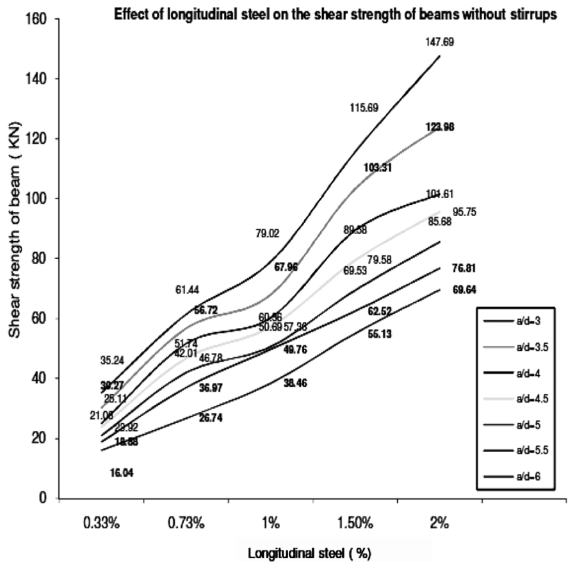


Fig. 8 Effect of longitudinal Steel ratio on the shear strength of concrete beams without stirrups for same value of *a/d*

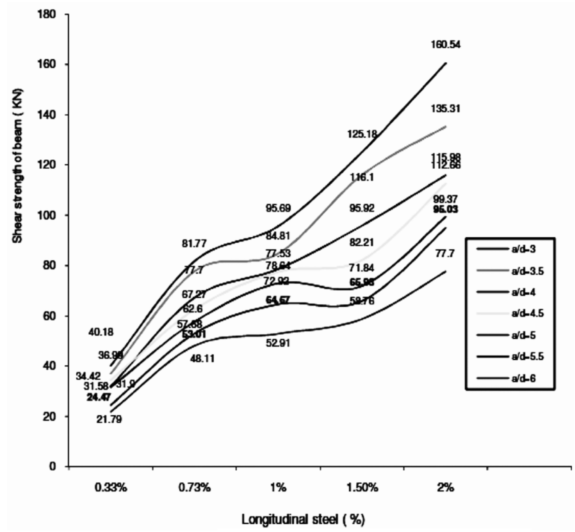


Fig. 9 Effect of longitudinal Steel ratio on the shear strength of concrete beams with stirrups for same value of *a/d*

increase in shear strength of beams with the increase of longitudinal steel is also referred as "dowel action". For a constant *a/d* ratio, when the longitudinal reinforcement was increased, the number of cracks, their widths and failure angle reduced. This verifies the concept of bond between concrete and longitudinal steel given in "Kani tooth model". Due to increase in the longitudinal steel, the bond force between the cracked concrete at the cantilever end also increased, thereby applying more action at the free end of cracks and reducing the failure angle. The phenomena is well illustrated by Modified Compression Filed Theory (MCFT) of Vecchio and Collins (1986), where the steel provided on the tension face of beams plays a significant role in restraining the cracks and improve the shear strength of beams. The effect of longitudinal steel on shear strength has been shown in Figs. 8 and 9 for both types of beams.

5.3 Effect of shear span to depth ratio on shear strength of concrete

The shear span to depth *a/d* ratio has a strong influence on the shear strength of RC beams. The shear strength decreases with the increase of *a/d* values for the same longitudinal steel. However the decrease is relatively more in case of beams without web reinforcement. The increase in shear span increases the number of cracks formed and as result more cantilever force applied at the cracked concrete, reducing the shear strength of concrete to greater extent. The effect of *a/d* values on the shear strength of beams has been shown in Figs. 10 and 11.

The increase in shear span for a constant section of beam leads to increase in the shear span to depth ratio. When the shear span increases, the deflection under external loads also increases and flexural cracks are formed at relatively lower values of external loads. The crack widths also increases which leads to reduction in interface shear transfer and larger cracks are formed. These cracks also reduce the depth of compression zone responsible for resisting the tensile stresses in the

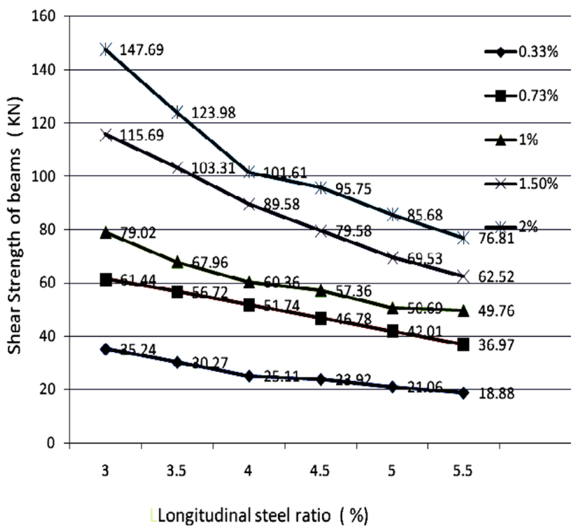


Fig. 10 Effect of *a/d* ratio on the shear strength of concrete beams without stirrups for same value longitudinal steel ratio

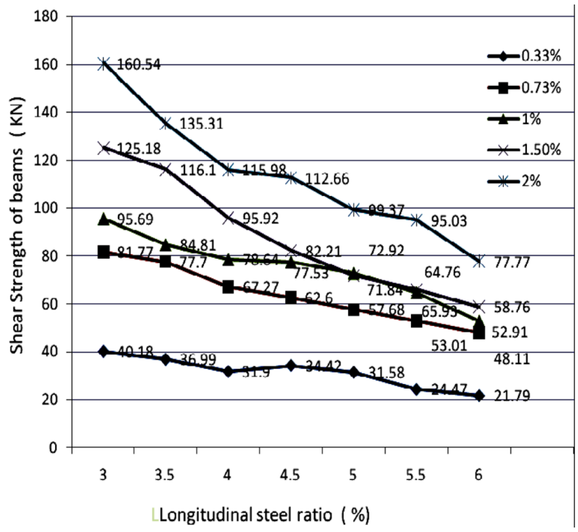


Fig. 11 Effect of *a/d* ratio on the shear strength of concrete beams with stirrups for same value longitudinal steel ratio

un-cracked part of concrete web. The phenomena can also be explained in terms of the cracked concrete, when the depth of the concrete cracks increases due to more deflection of the beams, the lever arm of the cracked concrete cantilever also increases, leading to more diagonal force on the un-cracked part of the concrete web, forcing it to fail at relatively lower value of applied load. Hence the "Tooth model of Kani (1966)" and "Failure of Compression zone" of Kotsovos (1986) can explain this phenomenon.

5.4 Effect of web reinforcement

The addition of web steel has increased the failure loads of the beams, thereby reducing the gap between the flexural moment (M_f) and ultimate cracking moment (M_u) on the famous "Kani's valley of shear failure (1966)". One of the arguments put forward by Kani (1966), for this increase is due to providing supports to the part of compression arches, which are otherwise unsupported and transfer their reactions to the supports, thereby avoiding their failure at lower loads. This explanation is no doubt in contrast to the famous "Parallel chord truss model of Ritter (1899)", where the role of stirrups was assumed to lift the load to the compression zone. The later argument seems more relevant to explain the increase in shear strength with stirrups. The same rationale was also supported by Kotsovos (1983) in his later work.

The increase in the shear strength is however not uniform for same values of web reinforcement. This has supported some of the research by Shehata *et al.* (2000), where the shear strength provided by the addition of shear reinforcement has been described as a complex phenomena and merely addition of the equivalent shear strength of stirrups with the concrete shear strength would not predict the total shear strength of RC beams. The results also support the fact that the role of web reinforcement in increasing the shear strength of beams is still not fully understood and there are contrasting explanations to this phenomenon. Hence further research is required in this area.

The existing building and bridges Codes in most of the cases use a uniform value of shear for certain level of web reinforcement, which is often taken as independent of the compressive strength of concrete, longitudinal steel ratio and shear span to depth ratio *a/d*. The test results have not supported this basic consideration of the codes, as the increase in shear strength is random for uniform value of web reinforcements. This observation deserves further research.

The average increase in the shear strength of beams with addition of web steel is relatively more at low level of longitudinal steel ratio. In beams without web reinforcement and lower values of longitudinal steel, flexural cracks are developed at early stage and the depth of cracks and concrete teeth increase with further increase of loads. The resultant cantilever action also increases and the diagonal cracking due to diagonal tension failure of beams happens. However, when the stirrups are added to such beams, the propagation of cracks is avoided at lower values of loads due to resistance of web steel in the compression zone, thereby increasing the resistance to the diagonal cracking, leading to increase in the shear strength of beams.

5.5 Comparison of the observed values with the provisions of ACI-318, EC-02 and JSCE

The comparison of observed values and cited codes, shows that the provisions of ACI, and JSCE codes are fairly reasonable for low values of longitudinal steel but the provisions EC-02 are less conservative comparatively for beams without web reinforcement.

The JSCE code has given relatively best and safe prediction amongst the three codes compared.

6. Conclusions

i. The failure in most of the beams has been caused due to diagonal tension cracking; however it was more dominant failure mode for beams without web reinforcement and having $\rho \geq 1\%$. For beams with $\rho < 1\%$, flexural shear failure was obvious failure mode.

ii. For beams without web reinforcement and having large values of longitudinal steel ($\rho = 1\%$ and 1.5%), the shear failure is more brittle and sudden, giving no sufficient warning.

iii. In beams with web reinforcement, the failure has been caused mainly by diagonal tension cracking even for small longitudinal steel ratio.

iv. The shear strength of all the beams having same longitudinal steel ratio and shear span to depth ratio has been increased with the addition of stirrups, but in a non-uniform manner. Hence uniform increase in shear strength of beams as given in most of the Codes was not observed. This increase is more prominent at lower steel ratios.

v. The addition of web reinforcement has avoided the brittle failure of the beam at higher values of longitudinal steel and the ductility of beams has increased.

vi. The shear strength of the beams has been increased with the increases of longitudinal steel in both the cases without and with web reinforcement. This increase is relatively more in case of beams with web reinforcement.

vii. For both types of beams, the shear strength of HSC beams has been decreased with the increase of shear span to depth (*a/d*) ratio. However this decrease is relatively more in the beams without web reinforcement.

viii. The provisions of EC-02 are less conservative for low values of longitudinal steel ratio. ACI-308 overestimates the shear strength of RC beams at higher values of longitudinal steel ratio. The Japanese Code, however reasonably estimated the shear strength of beams studied.

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