Shear Studies on High Strength Concrete Slender Beams

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Abstract— In this paper an experimental investigation is carried out on Sixteen HSC beams with constant width (125 mm) and varying depth (130mm and 100mm) by varying (i) shear span to depth ratio, (ii) the longitudinal reinforcement ratio and (iii) the minimum, maximum and no web reinforcement ratio were casted and tested to understand the shear behavior of the beams. The study is also carried for no web reinforcement. The load-deflection behavior and the failure pattern of the beams, ultimate shear strength and reserve shear strength are studied with varying a/d ratio, varying the cross-section and longitudinal reinforcement. The results obtained are compared with the different codal equations. Based on these observations, it can be concluded that, there are many parameters influencing the shear behavior of RC beams such as shear span to depth ratio (a/d ratio>2), depth of the beam and the percentage of the longitudinal reinforcement.

Key words: High Strength Concrete, Shear Span to Depth Ratio. Reserve Strength, Shear Capacity

I. INTRODUCTION

The amount of stirrup has a direct relation on the behavior of reinforced concrete members of general structure, since the structures are possible to fail in brittle manner without any warning sign if the shear stress rides over the shear carrying capacity [1].

One of possible and discovered reason is the size effect of the structure members, which was introduced by Okamura [2]. Another possible problem is the shear carrying capacity's design concept used as the strength of concrete at either failure or shear crack load and sum up with shear resistance of web reinforcement. It is possible that a small amount of web reinforcement cannot maintain the shear strength resist by concrete to be the same up to yielding point of stirrup itself [4]. However, there is still not a simple, albeit analytically derived formula to predict and with accuracy the shear strength of slender beams. In addition many of the factors that influence the determination of the required minimum amount of shear reinforcement are not yet known [3]. Unlike flexural failures, reinforced concrete shear failures are relatively brittle and particularly for members without stirrups can occur without warning because of this, the prime objective of shear design is to identify where shear reinforcement is required to prevent such a failure and then in a less-critical decision how much is required. Shear reinforcement, usually called stirrups links together the flexure tension and flexure compression sides of a member and ensures that the two sides act as a unit.

Shear failures involve the breakdown of this linkage and for members without stirrups, typically involves the opening of major diagonal crack .With advent of higher concrete compressive strengths and the corresponding increase in concrete tensile strengths there is concern that traditional amounts of minimum shear reinforcement may not be sufficient in high strength concrete beams. Minimum shear reinforcement must prevent sudden shear failure on the formation of first diagonal tension cracks at service loads levels.

To prevent brittle failures adequate reserve strength must be provided by the shear reinforcement after diagonal cracking of reinforced concrete beams. To control crack widths at service loads levels not only a minimum amount of shear reinforcement be provided but the maximum stirrup spacing must also be limited. Due to the higher tensile strength of high[4]-strength concrete a higher cracking shear is expected and hence would require a larger amount of minimum shear reinforcement stresses, the redistribution of shear stresses between flexural cracks and the local weakening of a cross section by transverse reinforcement which causes a regular pattern of discontinuities along a beam.

In the early stages of reinforced concrete design diagonal cracking was considered to be undesirable. However, it is now recognized that diagonal cracking under service load conditions is acceptable, provided crack widths remain within the same limits accepted for flexure.

Shear strength in steel reinforced concrete beams has been the subject of many controversies and debates since the beginning of 20th century. The shear strength of reinforced concrete beams has been extensively studied over the last five decades [6]. A large number of experimental and analytical works have been carried out for the case of slender beams (having a shear span to depth ratio a/d > 2.5) with and without shear reinforcement under two-point loading. Transversely loaded reinforced concrete beams may fail in shear before attaining their full flexural strengths if they are not adequately designed for shear. Unlike flexural failures, shear failures are very sudden and unexpected, and sometimes violent and catastrophic. A thorough knowledge of the different modes of shear failures and the mechanisms involved is necessary to prevent them.

II. LITERATURE REVIEW

Existing codes and specifications of different countries for reinforced concrete design with regard to shear differ considerably in important aspects. This only reflects the fact that we know very little about the behavior and strength of reinforced concrete subjected to shearing force in spite of the considerable number of tests and theoretical investigations made during more than half a century. Despite the great research efforts, however, there is still not a simple, albeit analytically derived formula to predict quickly and accurately the shear strength of slender beams. In addition, many of the factors that influence the determination of the required minimum amount of shear reinforcement are not yet known [15].

Flexural crack width is approximately proportional to the strain of tension reinforcement, the crack width at failure becomes smaller as the longitudinal steel ratio is increased. Also, with increasing *a*/*d*, the strain of tension reinforcement at failure is increased. Meanwhile, it is naturally expected that the interlocking force will be increased when the strength of concrete is high [7].

The compressive strength of concrete and the area of tension reinforcement [8].

Possibly from point 2. Shortly before reaching the critical failure point at 2 the more inclined lower crack 3 will open back, at least to the steel level and usually cracks marked 4 will develop [15].

III. EXPERIMENTAL INVESTIGATIONS

A. Preliminary Investigation

To produce HSC (M70),various tests on ingredients i.e. cement, sand and coarse aggregate and trial mixes cubes were cast using a low water binder ratio (< 0.3), carried out and chemical admixture suitably and using ACI 211 code. Required cube and cylindrical strength after achieving has been tabulated. Variable and constant parameters in the experiment has been detailed and described.

B. Material Property

The constituent materials used in the present experimental shown in table 1 work as follows Cement used is 53 grade OPC as per IS 12269-1970

Coarse aggregate	Crushed granite, passing through 12.5 mm and retained on 4.75mm sieve specific gravity =2.59, as per IS 383-1978
Fine aggregate	River sand, zone II, specific gravity =2.63 as per IS 383-1978
Water	Potable water as per IS 456-2000
Super plasticizer	CONPLAST SP 430 as per IS 9103-1999

Table 1: Material Properties

C. Mix Proportion

With above aim a Mix design for M 70 grade of concrete was done using ACI – 211 the final proportions for M70 concrete is given in the Table 2

Mix	**Cement**	**Fine agg.**	**Coarse agg.**	**Water**	**Super plasticizer**
M_{70}	1	1.25	1.95	0.28	0.18

Table 2: Showing mix proportion for M70

Parameter	**3 days in N/mm²**	**7 days N/mm²**	**28 days N/mm²**
Cube	42	56	80
Cylinder	36	48	72

Table 3: Compressive strength of high strength concrete (HSC)

D. Test Specimens

In the present investigation, 32 HSC slender beams were cast and tested. The beams were with and without web reinforcement, in accordance to IS, BS 8110, ACI 318, EC 2 with concrete strength of 70Mpa(M70). The cross sectional dimension of the beams are in two series 1) 125mm x 130mm, with 100 mm effective depth of 16 nos, 2) 125mm x 160 mm with 130mm effective depth, for both series the effective length was 0.9m and 1.5m. For the listed series a/d ratio, effective length, effective depth, longitudinal reinforcement, shear reinforcement spacing were varying and keeping compressive strength of concrete constant, in such a way that the beam fails in shear before attaining the flexural capacity, shear governs design. Details of specimens are shown in the table.4**.**

Series	**Beam**	**A/D shear span to depth**	**ρ (%) longitudinal steel ratio**	**S(mm) Spacing of shear steel**
A	B1	2.5	1.8	150
	B2	2.5	1.8	225
	B3	2.5	1.8	300
	B4	2.5	1.8	-
B	B5	2.5	3.2	150
	B6	2.5	3.2	225
	B7	2.5	3.2	300
	B8	2.5	3.2	-
C	B9	3.25	1.8	150
	B10	3.25	1.8	225
	B11	3.25	1.8	300
	B12	3.25	1.8	-
D	B13	3.25	3.2	150
	B14	3.25	3.2	225
	B15	3.25	3.2	300
	B16	3.25	3.2	-

Table 4: Details of beam specimens

The beams for series A,B,C and D are 125mmX130mm and series E,F,G and H are 125mmX100mm. The beams were simply supported having an effective span 0.9m,1.5m respectively.

The load was applied from a 500KN hydraullic jack through steel sections and steel plates supported on 2 steel rollers which were placed exactly on the loading points that were marked.

Beam. he load at which the first visible crack appeared was taken as " cracking load (P_{cr})" and the load at which the beam failed completely was taken as "ultimate load (P_u)".

The experimental test results are shown in table 5, the load v/s deflection for different beam specimen are shown in graph 1 to 3 and the variation of shear strength and shear behaviour of different specimen varying different parameters like a/d, logitudinal steel and spacing of shear steel are shown in the form of bar charts 1 to 5.

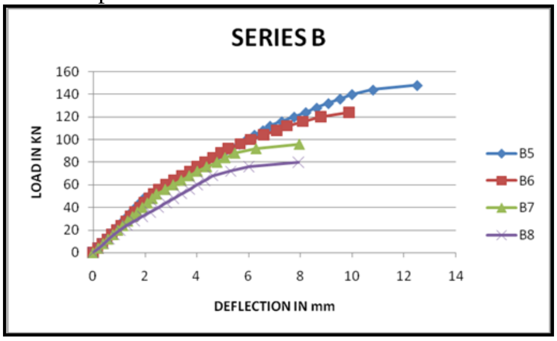
E. Experimental Test Results

Sr.	**Beam**	**P_{cr} (KN)**	**$\tau_{CR} = P_{cr}/2bd$**	**P_u (KN)**	**$\tau_u = P_u/2bd$**	**Mode of Failure**
A	B1	32	0.98	128	3.9	Shear
	B2	28	0.86	112	3.44	Shear
	B3	22	0.68	84	2.58	Shear
	B4	20	0.61	76	2.3	Shear
B	B5	42	1.29	148	4.55	Shear
	B6	33	1.02	124	3.9	Shear
	B7	24	0.75	96	2.95	Shear
	B8	20	0.62	80	2.46	Shear
C	B9	30	1.23	108	4.32	Shear
	B10	24	0.96	96	3.84	Shear
	B11	21	0.86	88	3.52	Shear
	B12	19	0.76	76	3.04	Shear
D	B13	30	1.21	120	4.8	Shear

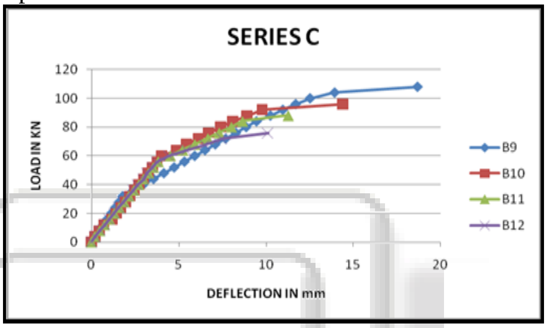
	B14	26	1.04	104	4.16	Shear
	B15	23	0.93	92	3.68	Shear
	B16	19	0.76	64	3.04	Shear

Table 5: Details of Test Results

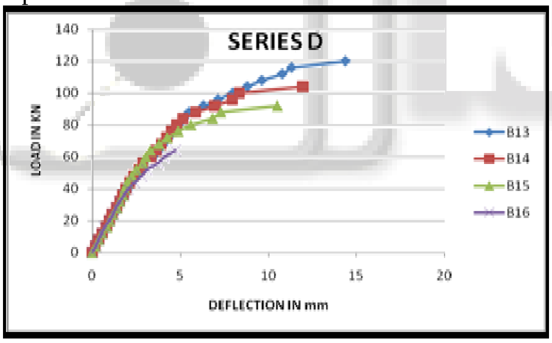
Results obtained by ACI CODE 318 were quite satisfactory as compared to other codes. So the values of ultimate shear strength from the experimental investigation has been compared with ACI code.



Graph 1: Load V/s Deflection for HSC series A beams



Graph 2: Load V/s Deflection for HSC series B beams



Graph 3: Load V/s Deflection for HSC series C beams

Fig. 5: Failure Patterns of Hsc Beams

For all the tested beams, the variation of the central deflection with the load were plotted to obatin the graphs as shown in load v/s deflection graphs. From graphs 1 to 4 .

The first series of graph are for single beams and the other series are for combination of different beams. From the graphs it was observed that lower reinforced beams (1.8 % longitudinal steel ratio) beams deflected more and also has lower load carrying capacity than higher reinforced beams (3.2% longitudinal steel ratio) keeping all other parameters constant , the beams with larger depth (130mm) deflected less and carried more load as compared to beams with smaller depths keeping all other parameters constant,but in case of beams without shear reinforcement, the longitudinal steel ratio and the depth of the beam was very insignificant and did not increase the load carrying capacity of the beam i.e there was no significant increase in shear capacity of the beam. It can be observed that as the effective length of the beam increased from 0.9m to 1.5m and keeping all the parameters of the beams constant the load carrying capacity decreased and bending moment in the beam increased, the deflection also increased significantly . The load deflection diagram indicates that the variation of load and deflection was linear up to cracking and it was non linear behaviour beyond cracking .

F. Bar Chart for Load v/s variation of Shear Reinforcement

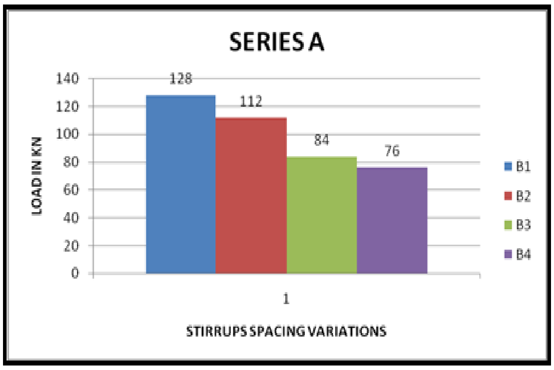


Chart 1: Load V/s Shear reinforcement spacing variation for HSC beam series A

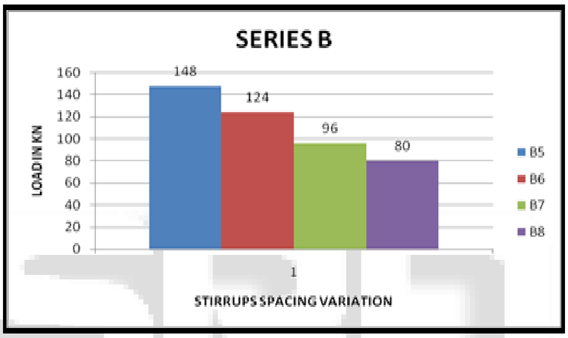


Chart 2: Load V/s Shear reinforcement spacing variation for HSC beam series B

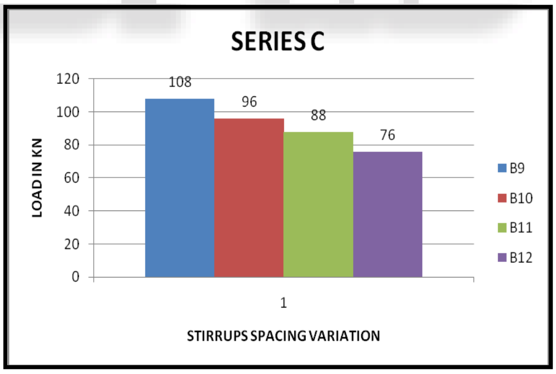


Chart 3: Load V/s Shear reinforcement spacing variation for HSC beam series C

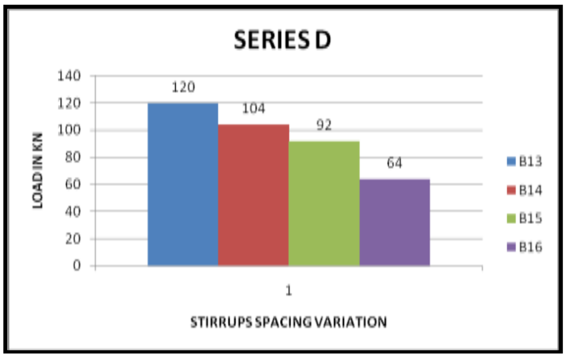


Chart 4: Load V/s Shear reinforcement spacing variation for HSC beam series D

Fig. 6: HSC beam series

The load v/s % shear reinforcement ratio i.e spacing of shear reinforcement has been plotted in the form of bar charts (5.41 – 5.48) to show the variation, it can be observed that as the spacing of shear reinforcement decreases the load carrying capacity increases.

G. Load V/S Percentage of Longitudinal Reinforcement

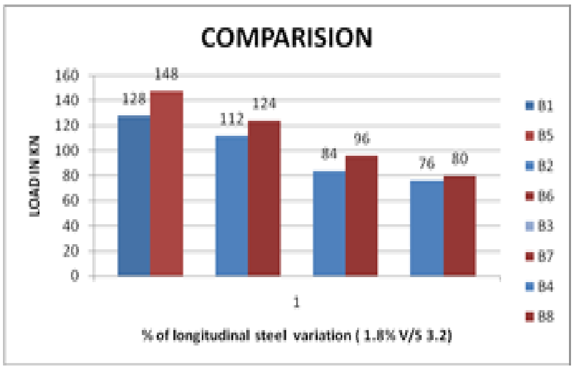


Chart 5: Load V/s Percentage of longitudinal reinforcement for HSC beam

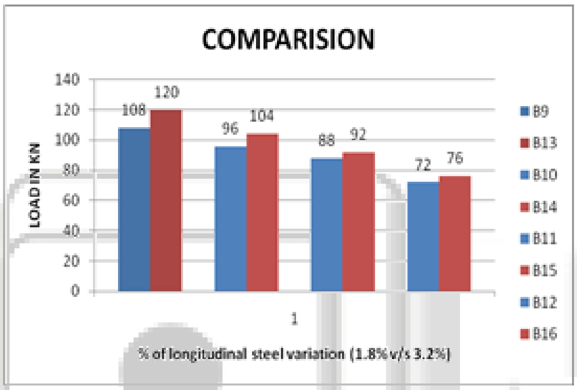


Chart 6 : Load V/s Percentage of longitudinal reinforcement for HSC beam

Fig. 7: Load V/S Percentage of Longitudinal Reinforcement

From the bar charts(5.49-5.52) it can be seen that the load carrying capacity increases as the % of longitudinal steel reinforcement increases, it is more significant for closer spacing of shear reinforcement (150mm, 225mm) and less significant for larger spacing of shear reinforcement (300mm) and the % of longitudinal steel reinforcement is very insignificant for beams without shear reinforcement.

From the graph 5.53 load carrying capacity increased as the shear span to depth ratio decreased and there was considerable variation in failure pattern of the beam with the variation of shear span to depth ratio. It is the most pivotal parameter in shear strength of the beams, smaller shear span to depth ratio (2.5, 3.25), higher the shear strength and larger the a/d ratio (4.25, 5.5) lower the shear strength.

H. Comparison of Experimental Load and Theoretical Loads (ACI 318)

Beam Series	**Experimental Load V(Exp) in KN**	**Theoretical Load V (The) in KN**	**Ratio (Exp/ The) V(Exp)/V(The)**
B1	64	35.65	1.79
B2	56	31.61	1.77
B3	42	29.59	1.41
B4	38	23.52	1.61
B5	74	37.27	1.98
B6	62	33.19	1.86
B7	48	31.17	1.53
B8	40	25.11	1.59
B9	54	27	2.01
B10	48	24	2.00
B11	44	22	2.00
B12	38	17.73	2.13
B13	60	28	2.14
B14	52	25	2.08
B15	26	23.35	1.11
B16	32	18.68	1.71

Table 5.1: Comparison of Experimental Load and Theoretical Loads (ACI 318)

The values tabulated in the last column indicate the ratio of experimental load to the theoretical load. Codal shear equations are conservative.

IV. CONCLUSION

The following conclusions were made from the experimental investigation

1) Mix proportion for HSC M70 was obtained using chemical admixture and without using mineral admixture and the average 28 day strength was found to be 77 Mpa.
2) As the percentage of longitudinal reinforcement increased from 1.8% to 3.2%, the shear strength of HSC beams also increased (dowel shear contribution increased), it was significant for small spacing of shear steel (150,225mm), but this was not significant for beams without web reinforcement and large spacing of stirrups (300mm) Failure becomes more sudden and explosive as the longitudinal steel ratio increases from 1.8% to 3.2%.
3) As the depth of the HSC beams increased, the deflection decreased and the load carrying capacity (shear capacity) increased but load carrying capacity was insignificant for beams without shear reinforcement.
4) As the length of the beam increased, keeping all other parameters constant than the load carrying capacity decreased and bending moment in the beam increased.
5) The shear span to depth ratio was very significant in the shear capacity and shear behavior of HSC slender beams, keeping all other parameters constant and as the a/d ratio decreased the load carrying capacity increased and vice-versa. Lower a/d ratio showed shear tension or diagonal tension mode of failure and higher a/d ratio showed shear flexure mode of failure. For the beams with a/d = 2.5 shows a shear compression failure for less stirrup spacing and for large stirrup spacing.

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