High strength concrete beams with combination of links and horizontal web steel as alternative shear reinforcement

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ABSTRACT

Equal numbers of high strength and normal strength concrete beams with links and various amounts of horizontal web steel provided at the centre of the cross-section of the members were tested to failure. The rules for estimating the contribution of web reinforcement to the shear resistance have been studied with the help of tests on beams and measurement of stresses in the steel using strain gauges.

Notation

A_{st} is the amount of tension steel (mm^2)
A_{sv} is the area of cross-section of a link (mm^2)
A_b is the area of cross-section of horizontal web steel (mm^2)
b is the width of the cross-section of a beam (mm)
b_n is the net breadth of the beam at level of dowels reinforcement (mm)
d is the effective depth of the cross-section (mm)
d_b is the diameter of horizontal web bar (mm)
f_{cu} is the mean cube strength of concrete (N/mm^2)
f_{yl} is the yield for longitudinal reinforcement (N/mm^2)
f_{yv} is the yield strength for stirrups reinforcement (N/mm^2)
s is the spacing of links along the length of the member (mm)
V_{bu} is the contribution of central bars to Vu (kN)
V_{calc} is the calculated ultimate shear strength (kN)
V_{cu} is the contribution of concrete to Vu (kN)
V_{lu} is the contribution of links to Vu (kN)
V_{test} is the measured ultimate shear strength (kN)
V_u is the ultimate shear resistance of a section (kN)
$\rho = 100\, A_{st}/ bd$
$\rho_b = 100\, A_b/ bd$

Introduction

High strength concrete is now being considered for a wide range of structural applications[1]. The existing recommendations in the British Code of Practice for the shear design of beams[2] are derived from research conducted essentially on Normal Strength Concrete (NSC) with cube strengths up to 50 Mpa, and it was felt that these might not be applicable to High Strength Concrete (HSC).

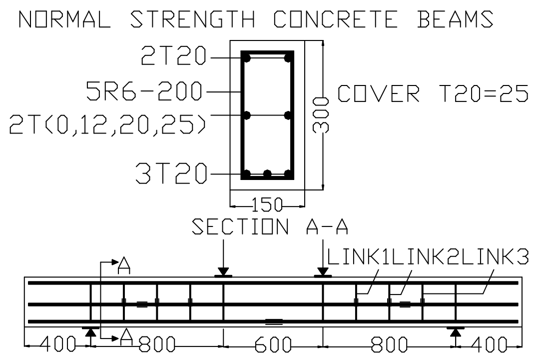

Fig 1. Reinforcement details and position of strain gauges for the Normal Strength Concrete test specimen

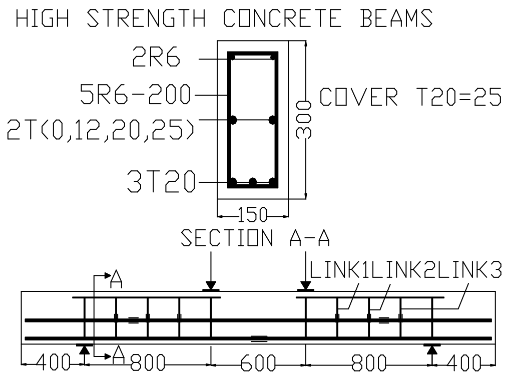

Fig 2. Reinforcement details and position of strain gauges for the High Strength Concrete test specimen

The present and recent[3] tests have shown that significant differences exist in the angle of crack of shear failure of NSC and HSC. In view of this, the current design recommendations of BS8110 for the maximum allowable spacing of shear links should be assessed in relation to HSC beams in shear. Previous investigations[4] have suggested that horizontal web steel can contribute to the overall shear resistance of a member in conjunction with other constituents, concrete, tension and shear steel.

Experimental investigation

Production details

The size and the length of the test specimens were chosen to make the beams fail in shear (a/d=3) and to ensure that the specimens were sufficiently large to simulate real structural elements. Fig 1 &Fig 2 show the details of the eight beams which were 150×300mm in section and had a span of

2.2m. The HSC beams had two 6mm mild steel bars in the top only in the shear spans and the NSC beams had 2T20 in the top throughout.

For all beams the tension steel was 3T20 and shear links were R6 at 200mm centres in the shear spans. Both NSC and HSC beams were tested without and with horizontal web steel of 2T12, 2T20 and 2T25.

Tests were carried out on three specimens representing the steel in the links and the average value f_{yv} was 250 N/mm^2. The reinforcement used for the top, bottom and horizontal web steel was high yield, hot rolled deformed bars with a guaranteed yield value fyl of 460 N/mm^2. The beam notation is explained in Table 1.

Details of concrete strengths, f_{cu} and f_{sp} are given in Table 1. In the concrete mix design, Rapid Hardening Portland cement was used in conjunction with 20mm gravel for NSC and 10mm limestone for HSC. fcu was about 44 N/mm^2 for the NSC and 111 N/mm^2 for the HSC. For HSC the water: cement ratio was kept at 0.29 with the addition of admixtures. The beam specimens, the 150 mm (BS) cubes for NSC and 100mm BS cubes for HSC were cured in 28 days. The compressive strength tests were conducted on the same days as the beam tests. The concrete for all the beams was compacted using an immersion mechanical poker vibrator.

Beam test procedures: At each load increment, the vertical deflection at mid-span as well as the strains in the links, horizontal web bars and tensile reinforcing bars, were recorded. The development of cracks was also observed and recorded.

Test results and discussions

A summary of the test specimen details and results is given in table 1. The discussion of this part is presented in four sections: (a) Shear failure loads; (b) Load-deflection behaviour; (c) Crack propagation (d) Load-strain behaviour.

(a) Shear failure loads: The first HSC1failure load of 130 kN (f_{cu} = 109 N/mm^2) appeared low, the second HSC1 failure load of 140 kN (f_{cu} = 101.2 N/mm^2) and third failure load of 160 kN (f_{cu}=106.6 N/mm^2). The average ultimate load carried by these three similar HSC1 beams was 143.3 kN(f_{cu}=105.6 N/mm^2) as compared to ultimate load of beam NSC1 which was 160 kN (f_{cu}=43.2 N/mm^2). The links were similar in the two and neither contained any horizontal web steel. NSC1 did have 1.55% of compression reinforcement which was not present in HSC1.The inclination of the critical shear crack was much steeper in HSC1 at about 50° as compared with approximately 35° in NSC1.

The surprising reduction of shear resistance with increasing concrete strength found for beams NSC1 and HSC1 was reversed when horizontal web steel was provided. With two 25mm web bars in both, the ultimate loads for HSC4 (f_{cu}=112.5 N/mm^2) and NSC4 (f_{cu}=43.3 N/mm^2) were 300 KN and 210 kN respectively.

The major increase of shear strength for the HSC beams occurred between HSC1 (no horizontal web bars) and HSC2 (2T12) with ultimate loads of 130 kN and 265 kN. The rises with increasing horizontal web steel were much more modest - HSC3 (2T20) carried 280 kN and HSC4 (2T25) took 300kN.

With ordinary concrete the influence of horizontal bars was modest; NSC1 (no web bars)-160kN, NSC2 (2T12)-203kN, NSC3 (2T20)- 200kN and NSC4 (2T25)-210kN.

The results for the four high strength concrete beams with horizontal web steel demonstrated that no limit to improvement in shear resistance as the result of increasing the area of horizontal web reinforcement was reached. When the diameter of the web bars was increased from 20 to 25mm a further 7% improvement was recorded.

Beam No	**Top Steel**	**Horizontal web steel**	**Cube Strength (f_{cu}) N/mm²**	**Splitting strength (f_{sp}) N/mm²**	**Ultimate load ($2V_u$) kN**
NSC1	2T20	0	43.2	2.98	160
NSC2	2T20	2T12	41.0	3.01	203
NSC3	2T20	2T20	47.7	3.22	200
NSC4	2T20	2T25	43.3	2.97	210
HSC1	2R6	0	109.0	4.21	140
HSC1-2	2R6	0	101.2	-	143.3
HSC1-3	2R6	0	106.6	-	160.0
HSC2	2R6	2T12	109.3	5.20	265
HSC3	2R6	2T20	112.5	4.34	280
HSC4	2R6	2T25	112.5	4.34	300

TABLE 1. DATA FOR BEAMS

(b) *Load-deflection behaviour.* Mid-span deflections were measured by a single gauge mounted from the laboratory floor and include any settlements of the supports.
The deflection of beam HSC1 was fairly similar to that of NSC1. Both beams were without any horizontal web reinforcement the 1.55% of compression reinforcement, which was present in NSC1, reduced its deflection but the higher strength and elastic modulus of the concrete in HSC1 with no compression steel counter-weighed the compression steel in NSC1 (see Fig 3). The deflection of beam NSC1 was greater than for NSC4 (2T25) at equal loads and NSC1's deflection near failure was the greater (see Fig. 4).

The deflections of HSC2, HSC3 and HSC4 did not change by more than 15% as the area of horizontal web steel was increased in beams of high strength concrete.

(c) *Crack propagation*. At loads of 40 to 60 kN, small flexural cracks appeared, at the bottom surface in the region of constant bending moment. As the load was increased new flexural cracks appeared in the shear spans spreading from the load application sections towards the supports and the flexural cracks in the shear spans tended to become somewhat inclined. This was followed by the sudden occurrence of a wide shear crack in one of the shear spans, which lead to failure.

A crack angle was defined as the angle between a tangent to the crack at the centre of the depth of the beam and its x-axis.

The angle of the failure crack for the high strength concrete beam HSC1 was about 50° compared to the 35° for the normal strength concrete beam NSC1.

Beams HSC2, HSC3 and HSC4 had respective angles of cracks of about 43°, 45° and 42° compared to beams NSC2, NSC3 and NSC4 with angles of cracks 28°, 27° and 27°.

HSC1 and NSC2 had dowel cracks at the level of the bottom steel. These cracks were formed at 120kN (92% V_u) and 140kN(64% V_u).

NSC3 and HSC4 may possibly have had dowel cracks in mid-web formed at 190kN (86% V_u) and 230kN (77% V_u). HSC3 and NSC4 developed web dowel cracks at 210kN (75% V_u) and 200kN (95% V_u).

(d) Load-strain behaviour. A comparison can be made between strains in links for the beams HSC4 and NSC4. Both beams had 2T25 horizontal web reinforcement

In the beam NSC4 links 1,2 and 3 yielded at 200 kN. Whereas, in HSC4 links 2 and 3 yielded at 200 kN and link 1 yielded at about 230kN. This shows that the difference between HSC and NSC is relatively small at the stage of stirrup yielding compared to the greater difference in failure load. Fig.5**.**

Beam HSC4 continued to sustain load for an increment of 100 kN after links 2&3 yielded and an increment of 70 kN after link 1 yielded. The horizontal web reinforcement (2T25) of HSC4 yielded at 270 kN, Fig 6**.**

One possible explanation is that the horizontal web reinforcement in beam HSC4 was stabilising arching. This resulted in yielding of the links and increased the forces in the main steel near supports. This tie effect of the tension steel continued until the tension reinforcement reached 90% of its yield strain at 300 kN when the beam failed. Fig 7**.**

The difference between high and normal strength concrete beams is partly in terms of the loads at which stirrups yielded. As Fig 5 shows this difference could amount to a maximum load difference of 70 kN.

In beam HSC1 as Figure 8 shows link 2 yielded at about 100 kN and link 3 reached 80% of its yield at 110kN. Shear failure occurred with a crack positioned between links 2 and 3. When failure occurred link 1 had not yet reached 40% of its yield, Fig 9, and the strain at mid-span of the tension steel had reached only 40% of its yield. Fig 10.

Proposal of an alternative design rule

The shear resistance of rectangular reinforced concrete beams with vertical stirrups can be assessed by the BS8110 equation, which with safety factors eliminated, becomes;

$$V_{cu} = 0.27 f_{cu}^{\frac{1}{3}}.\rho^{\frac{1}{3}}.\left(\frac{d}{400}\right)^{\frac{1}{4}}.bd + A_{sv}.d\frac{f_{yv}}{s} \quad \leq f_{cu}^{\frac{1}{2}}.bd$$

In the code upper limits of $\rho < 3\%$ and $f_{cu} < 40$ N/mm^2 are imposed. One way of assessing the total shear resistance of a member with a single layer of horizontal web steel is to add it's dowel resistance to the above V_{cu}.

Using Baumann's[5] dowel cracking expression:

$$D_{cr1} = K.b_n.d_b.f_{cu}^{\frac{1}{3}}$$

Baumann's equation is based on the idea that;

$$D_{cr} = Tensile\,strength\,of\,the\,concrete \times Net\,breadth\,of\,beam \times \Pr imary\,bearing\,length$$

The bearing length is proportional to: $\sqrt[4]{\left(\frac{flexural\,stiffness\,of\,dowel}{\mathrm{mod}\,ulus\,of\,\mathrm{sup}\,port}\right)}$

When there are n dowel bars then
Flexural stiffness of total dowel = n × Stiffness of one bar.
The modulus of support ought to be practically independent of the number of bars. This suggests a change of Baumann's equation from

$$D_{cr1} = K.b_n.d_b.f_{cu}^{\frac{1}{3}} \qquad \text{to} \qquad D_{cr1} = K.b_n.d_b.\sqrt[4]{n}.f_{cu}^{\frac{1}{3}}$$

To check if the movements of cracks should be sufficient for the mobilisation of D_{cr}, reference was made to published measurements of vertical movements at flexural cracks that developed into shear cracks. It was clear that the movements are large enough for dowel resistance to be fully achieved as it is limited by the tensile strength of the concrete, and a movement of about 0.1 mm can adequately mobilise it.

Hence if D_{cr} is adequately mobilised, the suggested formulation for the shear strength of the beam with stirrups and horizontal web reinforcement is;

$$V_{cu} = 0.27 f_{cu}^{\frac{1}{3}}.\rho^{\frac{1}{3}}.\left(\frac{d}{400}\right)^{\frac{1}{4}}.bd + A_{sv}.d\frac{f_{yv}}{s} + b_n.d_b.\sqrt[4]{n}.f_{cu}^{\frac{1}{3}}$$

The other proposal by Desai is;

$$V_{cu} = 0.27 f_{cu}^{\frac{1}{3}} . \rho^{\frac{1}{3}} . \left(\frac{d}{400} \right)^{\frac{1}{4}} .bd \left(1 + 40 \rho_b \right) + A_{sv} .d \frac{f_{yv}}{s} \leq V_{\max}$$

$$V_{cu} = 1.4 \times 0.27 f_{cu}^{\frac{1}{3}} . \rho^{\frac{1}{3}} . \left(\frac{d}{400} \right)^{\frac{1}{4}} .bd + A_{sv} .d \frac{f_{yv}}{s}$$

It is difficult to follow the reason why the ratio of main reinforcement should affect the contribution of the web bars. The upper limit is also hard to understand.

The test results from the present experimental work were compared with predictions from the proposal expression and Desai's equation.

For all the beams 6 mm diameter single links at 200 mm centres were used.

Therefore $V_{lu} = A_{sv} .d \frac{f_{yv}}{s}$

where A_{sv} = 56.6 mm^2 , f_{yv} = 250 N/mm^2 , d= 270 mm & s = 200 mm

Hence V_{lu} = 19.1 kN

100 A_s/bd = 2.33, d=270, b=150, ξ=1.1033,

$v_c = 0.395 f^{\frac{1}{3}}$, $V_{cu} = 15.994 f^{\frac{1}{3}}$

From the modified Baumann equation

$$V_{bu} = 1.64\, b_n\, d_b\, \sqrt[4]{n} . f^{\frac{1}{3}}$$

$$= 1.95 b_n d_b f^{\frac{1}{3}} \quad \text{(where n=2)}$$

Conclusion

- The use of strain gauges, a Demec enabled the cracking and deformation of slender reinforced high strength and normal strength concrete beams with stirrups, with and without horizontal web steel to be investigated at loads up to peak load.

- Design rules proposed as the result of previous research by S. B. Desai hold fair for the beams tested here. His rules produce reasonable estimates of ultimate shear resistance.

- Design rules proposed by BS8110 for normal strength concrete beams, with stirrups, and without horizontal web reinforcement are not valid if extrapolated to high strength concrete beams.

- Research by Desai, and the present tests on normal strength concrete beams with stirrups shows that for normal strength concrete, there is a limit to the maximum contribution of a central bar for beams with or without links.
- In general the tests on high strength concrete beams proved that horizontal web reinforcement located towards the centre of the beam improves the shear resistance significantly.
- The results for beams HSC1 compared with HSC2, HSC4 and NSC4 showed an enhancement of shear resistance of about 130% when horizontal web steel is provided.
- Research by Desai shows that the horizontal bars can provide, for design purposes, when considering fire exposure, their location protected by the surrounding concrete would be of some advantage.
- Further research will be required to find more realistic design rules for the enhancement of the shear resistance of high strength reinforced concrete members when horizontal web reinforcement is provided at the centre of the cross section.

Beam No	**NSC1**	**NSC2**	**NSC3**	**NSC4**	**HSC1**	**HSC2**	**HSC3**	**HSC4**
f_{cu} (N/mm^2)	43.2	41.0	47.7	43.3	109.0	109.3	112.5	112.5
V_{cu} (kN)	56.1	55.2	58.0	56.1	76.4	76.5	77.2	77.2
V_{lu} (kN)	19.1	19.1	19.1	19.1	19.1	19.1	19.1	19.1
Web Steel	-	2T12	2T20	2T25	-	2T12	2T20	2T25
V_{bu} (kN)	-	10.2	15.6	17.1	-	14.1	20.7	23.5
V_{calc} (kN)	91	100	108	108	111	125	132	135
V_{test} (kN)	80	101.5	100	105	65	132.5	140	150
$\frac{V_{test}}{V_{calc}}$	0.88	1.02	0.93	0.97	0.59	1.06	1.06	1.11
$100\rho_b$	0	1.06	1.50	2.44	0	1.06	1.50	2.44
V_c (1+0.4) (kN)	56.1	67.6	81.2	78.7	76.4	93.6	108.1	108.1
V_{cu} (kN) (Desai)	91	102	116	113	111	128	143	143
$\frac{V_{test}}{V_{cu}\ (Desai)}$	0.88	1.00	0.86	0.93	0.59	1.04	0.98	1.05

Table 2: Experimental values of ultimate shear resistance compared to values predicted from the proposed and Desai's formulae for beams with horizontal web bars
N.B: BS 8110's limit on f_{cu} has been ignored

References

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