Behaviour of high strength concrete beam-column connections

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ABSTRACT
Results from tests on eight high strength concrete external beam-column connection specimens are presented and compared with results from a similar set of normal strength concrete specimens. The number of connection zone column ties was varied from zero up to a total of seven. The technique of internally strain gauging the reinforcement was used to measure strains in the main beam and column reinforcement. Additionally, connection zone column ties were also strain gauged in the eight specimens which had one or three such ties in the connection zone. Results are presented to illustrate the performance of the strain gauged links and to indicate how load is shared between links when several are present. Twelve other specimens were tested. Nine used normal strength concrete augmented with steel fibres whilst the other three used non-standard reinforcement details involving steel plates. Results are presented to compare the performance of these specimens with those using high strength concrete.

INTRODUCTION
Some years ago the lead author was involved with a programme of tests on external beam-column connections whose purpose was to investigate the influence of reinforcement detailing on behaviour within the joint zone (1). The intrinsic mechanisms of joint behaviour were studied by using internally strain gauged reinforcing bars for the main beam and column steel. Further processing of data from the tests (2) led to a revision of the procedures prescribed by British Standard BS8110, Structural Use of Concrete (3) for the design of sections subjected to combined shear and axial compression.

The test specimens all used normal strength concrete (NSC) having a typical compressive cube strength of 50 MPa. However, since these tests were undertaken, there has been an upsurge of interest in the use of high strength concrete (HSC) with compressive cube strengths in excess of 100 MPa being readily obtainable. This prompted initiation of further tests to assess the effects on connection behaviour resulting from this higher strength material.

Many other factors also influence joint strength. One of the most important is the number of column ties within the connection zone, a parameter which the previous tests had not considered. These previous tests all included only one such tie, which was not strain gauged in the manner of the main beam and column reinforcement. Consequently, it was decided that gauged column ties should be included in the high strength tests in order to obtain data pertaining to the behaviour of these influential components. Thus the thrust of the new test programme was to test HSC beam-column connections with the main parameters considered being tension reinforcement detail and number of connection zone column ties. A similar set of NSC specimens would be tested to provide a direct comparison between the two concrete types.

Once the test programme was underway the opportunity arose to conduct tests using normal strength concrete containing steel fibres. It soon became apparent that these specimens were capable of rivalling the performance of those made with HSC and thus provide an alternative method of enhancing joint behaviour. For this reason, details of these tests are included in this paper. Finally three specimens were tested using novel means of anchoring the beam steel and/or stiffening the joint zone. Again this resulted in enhanced joint behaviour approaching that of HSC specimens and for this reason results from these tests are also included as they provide an interesting comparison with the other work. A total of thirty specimens have been tested to date - eight HSC, ten NSC, nine NSC plus fibres and three NSC plus plates - but, as the test programme is still in progress, this paper is essentially an interim report. Further details will be available by the time of the conference.

SPECIMEN DETAILS

Geometry

All specimens, as shown in Fig. 1, had a column 1700 mm high and 150x150 square into which framed, at mid-height, a beam 840 mm long, 210 mm deep and 110 mm wide. All main reinforcement was high yield steel, with four 16 mm diameter rebars being used for the column reinforcement and a pair of 16 mm diameter rebars for the beam tension steel. Ties and stirrups were 6 mm diameter mild steel.

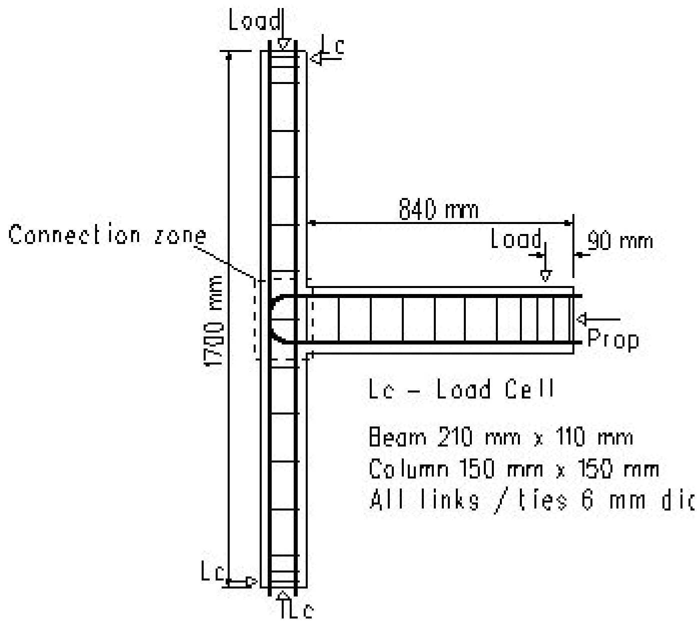

Figure 1 Specimen Geometery

Specimen details are summarised in Table 1. All but three specimens conformed to two arrangements for detailing the beam tension steel. C6L** specimens had U-bars for the main beam steel (Fig. 2a) whilst C4AL** specimens had the main beam steel bent down into the column (Fig. 2b) with a pair of 12 mm diameter rebars being provided in the bottom of the beam. (For consistency, the notation used in previous tests (1) has been retained in this paper.) The three exceptions to the above layouts are described later.

Table 1 Specimen Details

Specimen	Concrete Type	Beam Anchorage	No. of Joint ties	Percentage Fibres
C6LH0	**HSC**	**U-bar**	**0**	**None**
C6LH1 *	**HSC**	**U-bar**	**1**	**None**
C6LH3 *	**HSC**	**U-bar**	**3**	**None**
C6LH5	**HSC**	**U-bar**	**5**	**None**
C4ALH0	**HSC**	**Bent down**	**0**	**None**
C4ALH1 *	**HSC**	**Bent down**	**1**	**None**
C4ALH3 *	**HSC**	**Bent down**	**3**	**None**
C4ALH5	**HSC**	**Bent down**	**5**	**None**
C6LN0	NSC	U-bar	0	None
C6LN1 *	NSC	U-bar	1	None
C6LN3 *	NSC	U-bar	3	None
C6LN5	NSC	U-bar	5	None
C6LN7	NSC	U-bar	7	None
C4ALN0	NSC	Bent down	0	None

C4ALN1 *	NSC	Bent down	1	None
C4ALN3 *	NSC	Bent down	3	None
C4ALN5	NSC	Bent down	5	None
C4ALN7	NSC	Bent down	7	None
C6L_0.4%SF	NSC	U-bar	0	0.4 short
C6L_0.4%LF	NSC	U-bar	0	0.4 long
C4AL_0.4%SF	NSC	Bent down	0	0.4 short
C4AL_1.5%SF	NSC	Bent down	0	1.5 short
C4AL_2.3%SF	NSC	Bent down	0	2.3 short
C4AL_0.4%LF	NSC	Bent down	0	0.4 long
C4AL_1.5%LF	NSC	Bent down	0	1.5 long
C4AL_2.3%LF	NSC	Bent down	0	2.3 long
C4AL_3.8%LF	NSC	Bent down	0	3.8 long
C4PLN0	NSC	Anchor plate	0	None
C6LNP4	NSC	U+shear plate	0	None
C6PLNP4	NSC	Anchor+ shear plate	0	None

* denotes strain gauged reinforcement including joint zone column ties

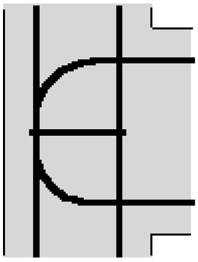

Figure 2a

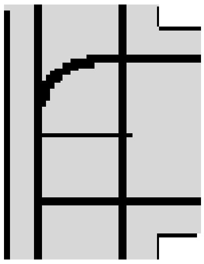

Figure 2b

HSC Specimens
Eight HSC specimens were tested, four each of the two detailing types. Either zero, one, three or five column ties were provided in the connection zone (see Table 1). Specimens with one or three column ties contained strain gauged reinforcement, described below.

Concrete for these specimens used 10 mm aggregate and microsilica (in slurry form) in addition to normal portland cement and sand. The overall moisture/binder ratio was around 0.3 with adequate workability being achieved by the addition of admixtures. Compressive cube strengths at the time of test ranged from 119 to 132 MPa and indirect tensile strengths (measured by splitting cylinders) ranged from 3.7 to 5.5 MPa. Details are given in Table 2.

NSC Specimens
Ten NSC specimens were tested, five each of the two detailing types. The pattern of specimens differed from those using HSC only to the extent that two specimens contained seven column ties in the connection zone (Table 1). Once again, specimens with one or three column ties contained strain gauged reinforcement.

The concrete used 10 mm aggregate with an aggregate/cement ratio of 5.5 and a water/cement ratio of 0.6. Compressive cube strengths at the time of test ranged from 46 to 64 MPa and indirect tensile strengths ranged from 2.7 to 3.5 MPa, as shown in Table 2.

Fibre Reinforced Specimens
Nine specimens were tested which used NSC enhanced with steel fibres. Two types of fibre were used, short and long, both made from carbon steel. Short fibres were 25 mm long, crimped and

made by the slit sheet method. Long fibres were 50 mm long cold drawn wire with end hooks. Manufacturer's specified tensile strengths were 410-830 MPa and 1100-1400 MPa respectively. Fibre quantities, by volume, were 0.4, 1.5, 2.3 and 3.8% (approximately equivalent to 30 to 300 kg/m3). The NSC was mixed in the normal way with the fibres being sprinkled over the mix at the end and then mixed in. 3.8% fibres was the upper limit to this mixing procedure. All but two of the specimens used the bent down detail for the beam tension steel (see Table 1).

Non-Standard Specimens
Three additional specimens were made and tested to investigate the possibility of enhancing the performance of the NSC specimens by novel reinforcement layouts in the connection zone. These layouts are illustrated in Figs 2c, 2d and 2e.

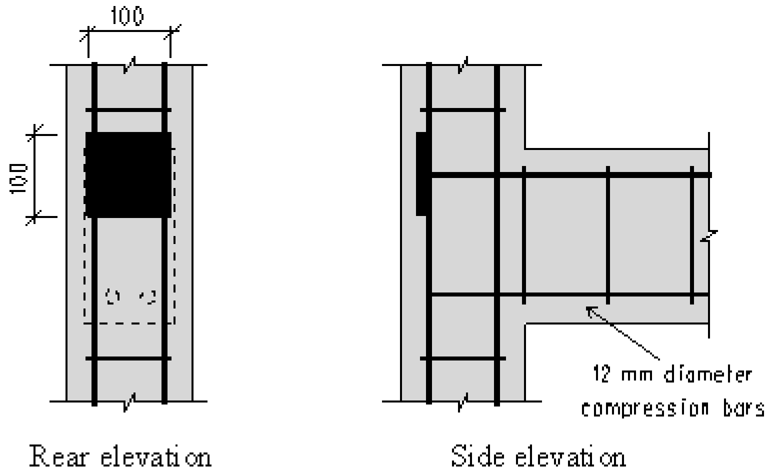

Figure 2c

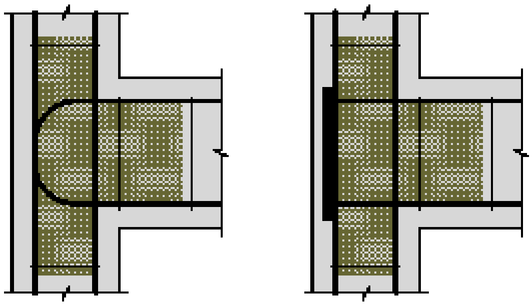

Figure 2d **Figure 2e**

Specimen C4PLN0 (Fig. 2c) was based on the C4AL** type specimens. However, instead of bending the beam tension steel down into the column, straight bars were used which were secured by being welded to anchor plates located in the cover to the main column bars. The purpose of this detail was to reduce the slip experienced with the bent down detail.

Specimen C6LNP4 was a U-bar specimen with the connection zone stiffened with a shear plate which extended 150 mm into the beam (Fig 2d). The plate was made from 4 mm thick mild steel with Expamet steel fabric spot welded to both sides to improve bond with the concrete. Increased joint stiffness was expected.

Specimen C6PLNP4 was a combination of the previous two as it used both anchor plates for the bars (top and bottom in this case) and a shear plate (Fig. 2e). A very large increase in joint stiffness was expected.

None of these three specimens contained connection zone column ties. The concrete used was the same as that for the standard NSC specimens.

Strain Gauged Reinforcement
As indicated earlier, eight specimens (see Table 1) contained strain gauged reinforcing bars to achieve very detailed measurements of reinforcement strain and bond stress distributions.

The technique used was that of internally strain gauging the reinforcement whereby electric resistance strain gauges were mounted in a central duct running longitudinally through the centre of the reinforcing bars, thus avoiding disruption of the bond between the bars and the surrounding concrete. This technique has undergone considerable development at Durham University where it has also received extensive use. In the gauged connection specimens, strain gauged reinforcement was used for two of the column bars, the U-bar (C6L** specimens), the top and bottom beam bars (C4AL** specimens) and the connection zone column ties. This involved producing bars in a range of shapes in addition to bars which were simply straight. In particular, the gauging of column ties is believed not to have been attempted previously. Each gauged specimen contained up to 240 strain gauges.

SPECIMEN TESTING

The test procedure was to load the column first and then maintain this load while the beam was loaded incrementally, vertically downwards. The load points and load cell arrangement are shown in Fig. 1. Column loads were 100 kN for the HSC specimens and 50 kN for all the others, the intention being to induce a strain of 100 microstrain in the column bars prior to beam loading.

Beam loading caused a diagonal crack to form in the connection zone of each specimen followed, in due course, by specimen failure due to further extensive joint cracking or due to a plastic hinge forming in the beam at the face of the column. Typically there were around 50 load increments per test.

SPECIMEN BEHAVIOUR

Failure Modes

Specimen behaviour was typical in that all specimens exhibited flexural cracking in the beam and column regions followed by diagonal cracking in the connection zone itself. Further shear was then carried by the concrete struts between the cracks assisted by the confinement of the joint provided by the connection zone links, where these were present.

Failure occurred when a specimen could not withstand further increase in load. There were two different failure mechanisms. If the ultimate moment of resistance of the beam was reached then a plastic hinge formed in the beam at the face of the column. If excessive shear cracking developed in the connection zone or anchorage failure of the beam steel occurred, before the beam reached its ultimate moment, then a joint shear failure occurred. Cracking loads (Pcrack), failure loads (Pfail) and failure mechanisms (Beam or Joint) are listed in Table 2.

Table 2 Cracking and Failure Loads

Specimen	Cube Strength (MPa)	Tensile Strength (MPa)	Pcrack (kN)	Pfail (kN)	Failure (B- beam J - joint)
C6LH0	126	4.7	25	36	J
C6LH1 *	127	4.9	22	37	J
C6LH3 *	121	3.7	23	41	J
C6LH5	125	5.2	26	51	**B**
C4ALH0	130	4.9	21	43	**B**
C4ALH1 *	119	3.7	20	43	**B**
C4ALH3 *	132	5.5	25	46	**B**
C4ALH5	123	5.0	28	49	**B**
C6LN0	64	3.4	19	24	J
C6LN1 *	64	3.3	18	25	J
C6LN3 *	61	3.2	18	29	J
C6LN5	46	2.7	15	34	J

C6LN7	59	3.3	19	40	J
C4ALN0	53	2.7	13	27	J
C4ALN1 *	57	3.4	19	34	J
C4ALN3 *	52	2.9	13	35	J
C4ALN5	63	3.2	14	40	J
C4ALN7	63	3.5	19	45	B
C6L_0.4%SF	54	3.2	17	24	J
C6L_0.4%LF	42	4.5	18	23	J
C4AL_0.4%SF	45	4.5	18	31	J
C4AL_1.5%SF	47	3.8	18	33	J
C4AL_2.3%SF	55	4.7	24	42	B
C4AL_0.4%LF	42	4.2	16	35	J
C4AL_1.5%LF	50	4.6	23	36	J
C4AL_2.3%LF	56	5.2	22	43	B
C4AL_3.8%LF	52	4.7	25	45	B
C4PLN0	54	3.1	19	32	J
C6LNP4	58	3.0	20	38	J
C6PLNP4	51	3.2	18	48	J

Joint Cracking Loads

Joint cracking loads are displayed as a histogram in Fig. 3. Results are scattered (due partly to the difficulty in identifying precisely when a crack actually formed) but some trends may be discerned. With HSC specimens an increased number of joint zone column ties raised the cracking load, but having no ties at all was remarkably effective in the case of the U-bar specimen (C6LH0). Values for the NSC specimens are even more scattered with no convincing trend being discernible. It would seem reasonable to conclude that initial joint cracking was largely unaffected by the number of column ties in the connection zone.

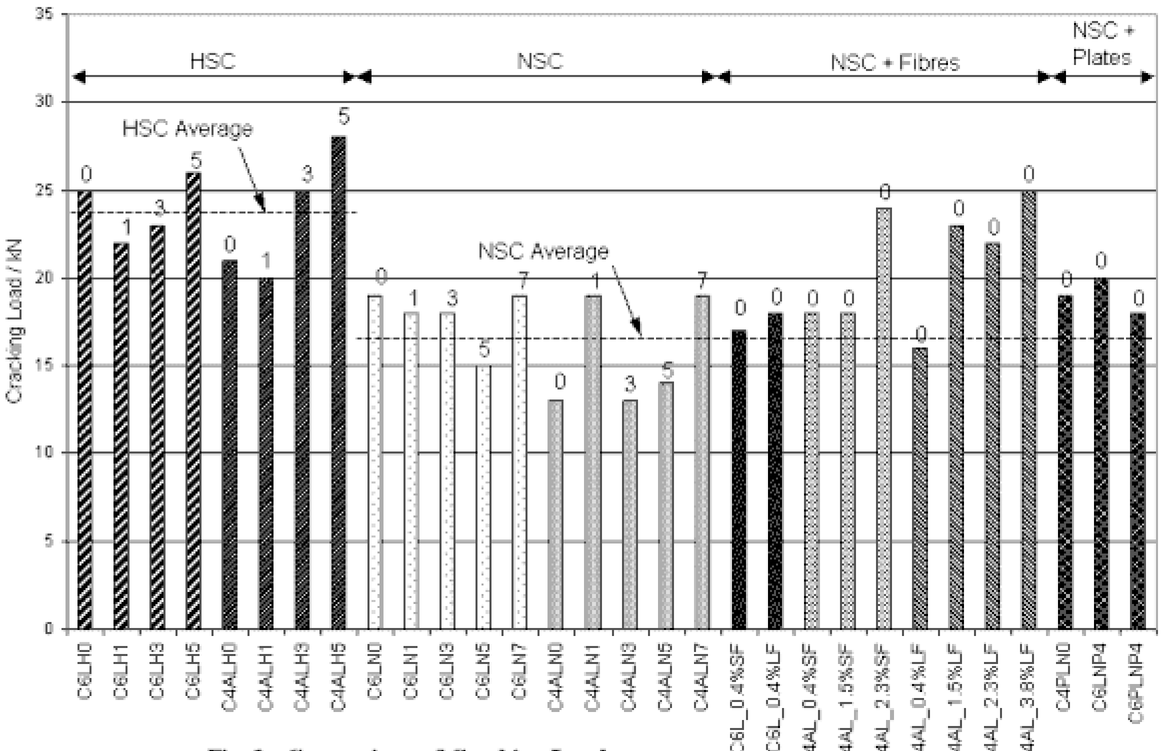

Fig. 3 : Comparison of Cracking Loads

Figure 3 Joint cracking loads

Average joint cracking loads were 23.8 and 16.7 kN for the HSC and NSC specimens respectively, as indicated in Fig. 3, an increase of 42% in the former over the latter. However, the lowest value for an HSC specimen was 20 kN (C4ALH1), an increase of only 1 kN over the best NSC value of 19 kN (C6LN0, C6LN7 and C4ALN7).

Adding fibres gave more consistency. 0.4% fibres, long or short, gave cracking loads close to the NSC average for both reinforcement details, as did 1.5% short fibres for the bent down detail (C4AL_1.5%SF). 2.3% short fibres, U-bar detail, and 1.5% and above long fibres, bent down detail, all gave markedly bigger cracking loads although the range of 22-25 kN was small. The average for these four specimens was 23.5 kN, almost identical with the HSC average.

There was little difference in behaviour between the three plate specimens which had an average cracking load of 19 kN, slightly above the NSC average.

Failure Loads

The load for specimen failure (Pfail in Table 2) was significantly affected by the number of links in the connection zone, as well as by concrete strength and beam steel detail. Joint failure loads are listed in Table 2 and displayed as a histogram in Fig. 4.

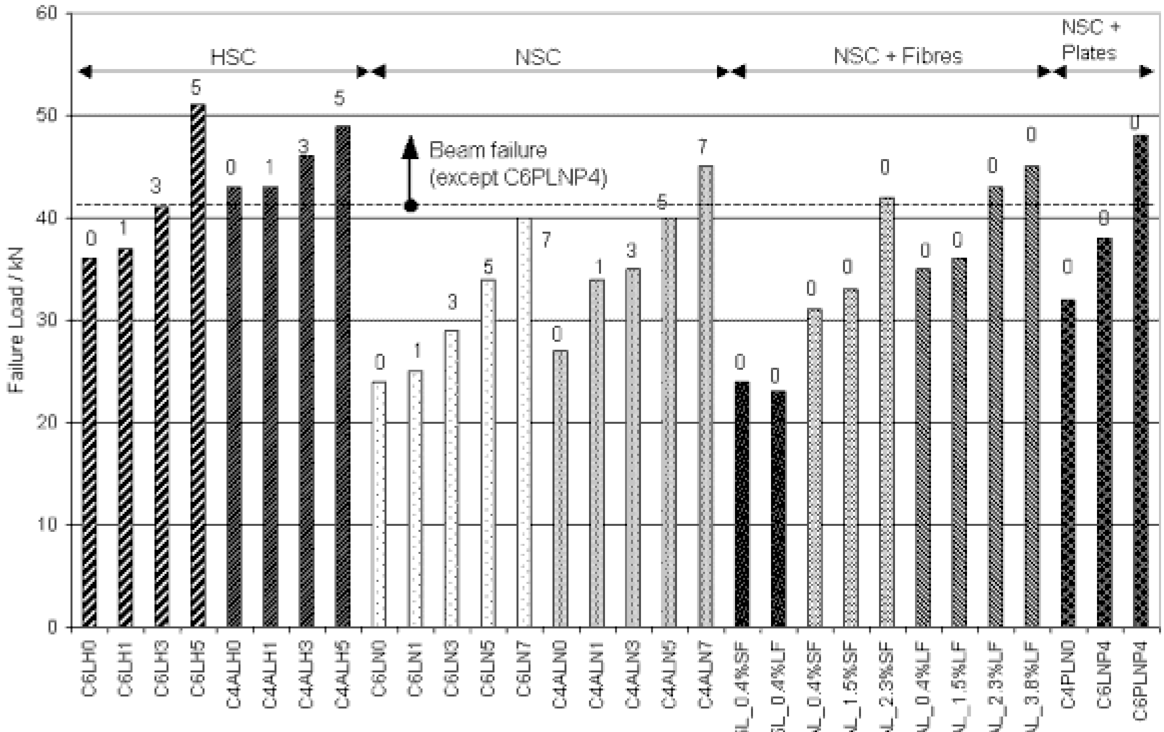

Fig. 4 : Comparison of Failure Loads

The bent down beam reinforcement detail performed better than the U-bar detail for all concrete types. This was because the anchor leg of the bent down bar transferred most of the beam moment into the column below the connection zone whereas, with the U-bar detail, all moment transfer occurred within the joint zone itself. This is discussed in more detail elsewhere (1).

The ultimate moment of resistance of the beam was reached at beam loads of around 45 and 42 kN for the HSC and NSC specimens respectively, although there was some variation between specimens due to differing cube strengths and, maybe, variations in steel yield strengths. The boundary between beam and joint failure shown in Fig. 4 should thus be regarded as a separator rather than referring to a specific failure load. For a specimen to fail in bending the joint had to be strong enough to withstand the shear force that this beam load transferred into the connection zone. It is clear from Fig. 4 that HSC gave increased shear strength in the joint. Failure loads for HSC specimens which failed in shear (all U-bar detail) were 40-50% higher than their NSC equivalents. The joint zone shear strength of the HSC bent down specimens was sufficiently enhanced for them all to exhibit beam failures even when no joint zone column ties were present.

The addition of column ties consistently improved the shear strength of the joint zone as is particularly evident with the NSC connections. Provision of seven such ties gave performance comparable with that using HSC and Fig. 4 enables a direct comparison to be made. The addition of steel fibres made little difference to the two U-bar specimens tested but significantly improved the performance of the bent down detail, particularly when it is noted that none of these specimens contained joint zone column ties.

The three non-standard specimens, C4PLN0, C6LNP4 and C6PLNP4 all failed in shear although the failure load of C6PLNP4 was comparable with the best of the more conventionally detailed specimens (see Fig. 4). The hope with these specimens was that the joint zone shear strength would be sufficiently enhanced for failure to occur in the beam rather than the joint. C6PLNP4, having both anchor plates and a shear plate came nearest to achieving this aim although the test indicated that the shear plate extended too far into the beam to allow beam failure to occur. This would be easy to remedy in future tests.

COLUMN TIE BEHAVIOUR
The strain gauged column ties performed well. With the test programme still in progress it is

premature to present detailed results, but a number of trends will be presented to illustrate the effectiveness of the technique.

Single Column Tie

Strain distributions for the single links in specimens C6LN1 and C6LH1 are shown in Fig. 5. These were both U-bar specimens which differed only in the concrete type used. All strain values quoted are the average of the ten strain gauges within each tie.

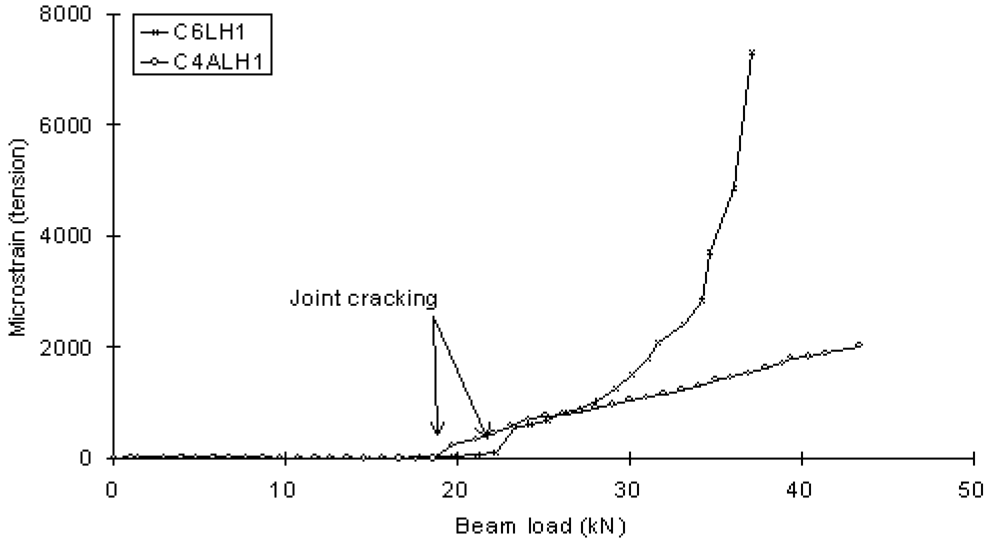

Figure 5 C6LN1 / C6LH1 strain comparisons

Joint cracking, at a strain of around 150 microstrain, is denoted by a marked change in slope in the strain distributions and is observed to have occurred at a higher load in the high strength specimen (C6LH1). Once joint cracking occurred strains increased at a similar rate in both specimens. Specimen C6LN1 failed when the shear strength of the joint was reached and this corresponded to a relatively low strain value in the tie (1510 microstrain). C6LH1 withstood a greater joint shear force and thus a significantly higher tie strain was recorded when failure occurred (7290 microstrain).

Fig. 6 shows differences in behaviour between the two beam steel details. Similar U-bar and bent-down specimens are plotted (C6LH1 and C4ALH1 respectively) and again average tie strain is shown for the single link in each specimen. Joint cracking occurred earlier in the test for the bent down specimen (C4ALH1) than for the U-bar specimen (C6LH1). It is thought that the U-bar main beam steel arrangement was more resistant to initial shear cracking due to there being more steel in the connection zone and the joint being more rigid.

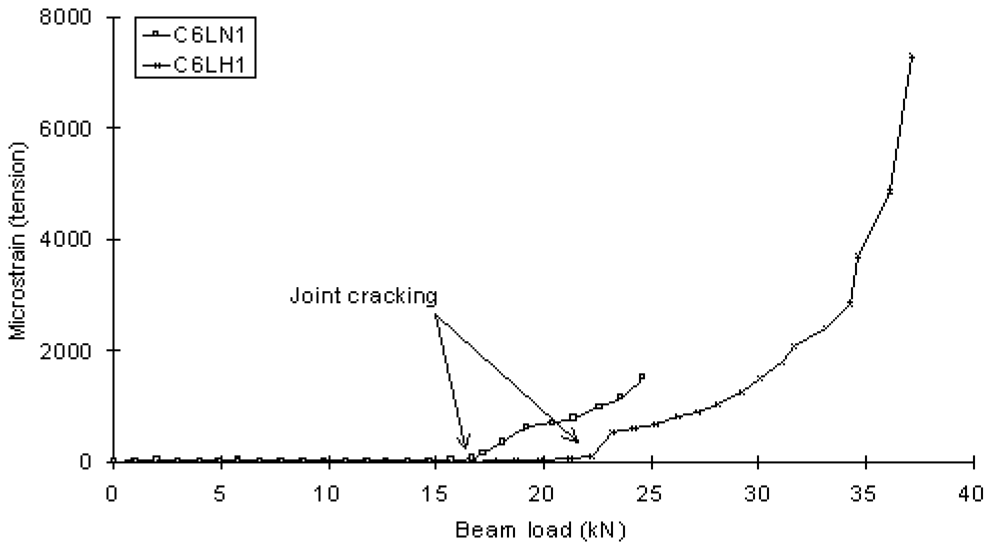

Figure 6 C6LH1 / C4ALH1 strain comparisons

After joint cracking there was a clear change in the rate at which tensile strains increased in the two ties. The U-bar specimen's joint cracked later than the bent-down specimen's but strains in the former then increased at a greater rate. C6LH1 failed by joint cracking with a maximum link strain of 7290 microstrain. Specimen C4ALH1 failed at a tie strain of only 2020 microstrain in spite of its failure load being nearly 7 kN more than that for C6LH1. This was because the plastic hinge which developed in the beam of C4ALH1 limited the rotation, and hence the strains, in its joint zone.

The reason for the different failure modes in these two specimens is thought to be due to the anchorage effects. With the bent-down specimen, C4ALH1, the force from the beam is transmitted away from the connection zone and into the lower column, whereas in the U-bar specimen, C6LH1, all force transfer has to take place within the connection zone. This puts the connection zone under more stress and precipitates a joint failure.

Load Sharing Between Ties

Fig 7 shows strains for the three connection zone ties in specimen C6LN3, the normal strength U-bar specimen. For comparison, the bold line shows strains for the single tie in the similar specimen C6LN1.

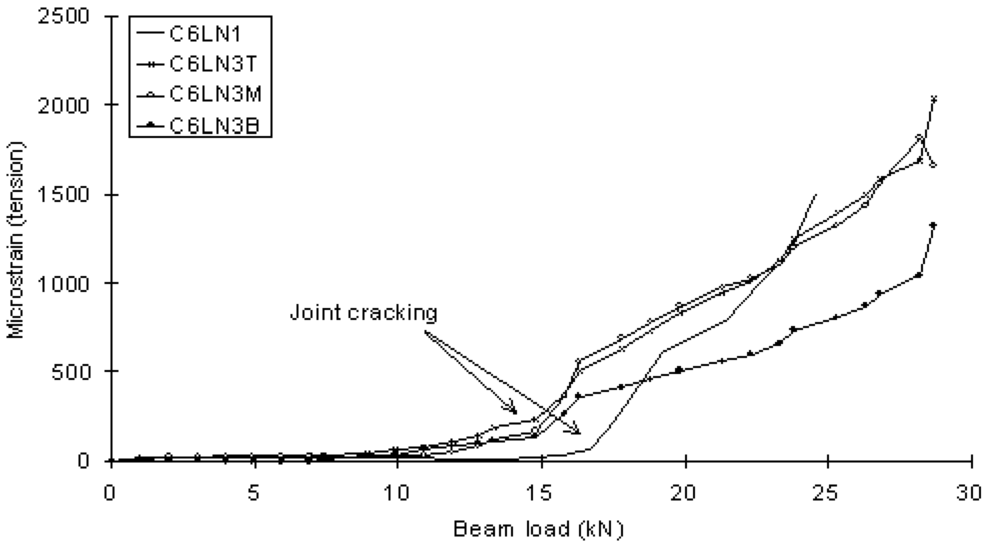

Figure 7 C6LN3 / C6LN1 comparisons

It can be seen that load is not shared equally between the three ties in C6LN3. After joint cracking strains in the top and middle tie are significantly higher than those in the lower tie, probably due to the top beam steel being more highly stressed than that in the bottom. Specimen C6LN1, with only a single tie, goes through joint cracking slightly after the three tie specimen. After joint cracking the single tie in specimen C6LN1 is seen to develop strain at a higher rate than the three ties in C6LN3. The single tie specimen also failed at a lower ultimate load than the three tie specimen.

Results from the three tie specimens suggest that when only a single tie is provided in the connection zone it would perform more effectively if it was positioned nearer to the level of the main beam tension steel rather than at mid-height. It is suspected that raising the tie would enhance joint shear capacity significantly, perhaps towards that for a three tie specimen. Further work is currently in progress at Durham to address this issue.

CONCLUSIONS

(1) Thirty external reinforced concrete beam-column connections specimens have been tested to date. Eight specimens had internally strain gauged reinforcement which included strain gauged ties in the connection zone.

(2) Connection zone ties appeared to have little effect on the performance of a joint until initial joint cracking first occurred.

(3 Use of high strength concrete increased the ultimate shear strength of the connection zone and caused higher strains to be developed in the joint zone column ties.
(4) Bending the main beam steel down into the column rather than using a U-bar increased the ultimate load at which a specimen failed.
(5) Increasing the number of shear ties in the connection zone increased the ultimate shear strength of the joint, due to the enhanced confinement of the joint core.
(6) Adding steel fibres to normal strength concrete considerably increased joint performance even when no joint zone column ties were present.
(7) Promising results were obtained from the three specimens using plate anchorages.

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