Connection zone strains in reinforced concrete beam-column connections

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ABSTRACT: The presence (or absence) of column links (ties) within the connection zone of a reinforced concrete beam-column assemblage was investigated using test results from sixteen external connection specimens. Eight specimens were made using normal strength concrete and eight using high strength concrete. The number of connection zone links varied from zero to five. The technique of internally strain gauging the reinforcement was used to measure strains in the main beam and column reinforcement. Additionally, connection zone links were also strain gauged in the eight specimens which had one or three such links 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.

1. INTRODUCTION

The joints between beams and columns are crucial components in reinforced concrete framed structures. The performance of beam-column connections is influenced by many parameters such as column load and the percentage and detailing arrangement of the beam tension steel. In addition the presence (or absence) of column links (ties) within the connection zone itself is also known to have an important effect on joint ductility as increasing the number of links leads to a joint being able to sustain higher loads and increased rotations before failure. This facet of behaviour is used extensively when designing for seismic conditions.

Extensive world-wide research has been undertaken to investigate the behaviour of beam-column connections both under monotonic loading and simulated seismic loading. Taylor (1974) addressed the problem of monotonic loading and carried out 26 tests which resulted in BS8110's recommendations (1985) for avoidance of shear cracking and connection zone steel congestion. Sarsam and Phipps (1985) followed up Taylor's work and linked this with the results from Meinheit and Jirsa's (1977) cyclic work to present refined equations for shear cracking and failure. Scott (1992,1994) continued the study of monotonic specimens by measuring reinforcement strains using internally strain gauged reinforcement. As a result BS8110's equation 6 was split into equations 6(a) and 6(b) (Scott 1994). Parker and Bullman (1995,1997) have recently tested 12 relatively large scale specimens and presented a shear model to represent connection zone behaviour.

There is, however, an absence of data concerning the strains developed in the links over the load history of the specimen, a serious omission in view of their important influence on connection behaviour. Consequently, Scott's work (1992,1994) is being continued by conducting further tests in which the connection zone shear links are strain gauged in addition to the main beam and column reinforcement. Once again, the technique used to measure reinforcement strains is that of internally strain gauging the reinforcement. Specimens containing up to three strain gauged links have been successfully manufactured and tested to give a total of around 240 strain gauges in each specimen. This paper presents preliminary results from this test programme.

2. SPECIMEN DETAILS AND TESTING

Sixteen specimens, having the geometry indicated in Fig 1, have been tested to date from a proposed total of twenty. Each had a column 1700 mm high and 150 x 150 mm 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 T16 bars being used for the column reinforcement and a pair of T16 bars for the main beam tension steel. Links were all mild steel. Parameters investigated were concrete strength, beam tension steel detail and presence (or absence) of connection zone links. These details are summarised in Table 1 which indicates that specimens were divided into four groups determined by reinforcement detail and concrete strength. C6L** specimens had U-bars for the main beam steel and C4AL** specimens had the main beam steel bent down into the column (the notation used in previous tests (Scott 1992,1994) has been retained for consistency). Typical compressive cube strengths were 60MPa and 120MPa for the normal and high strength concretes respectively.

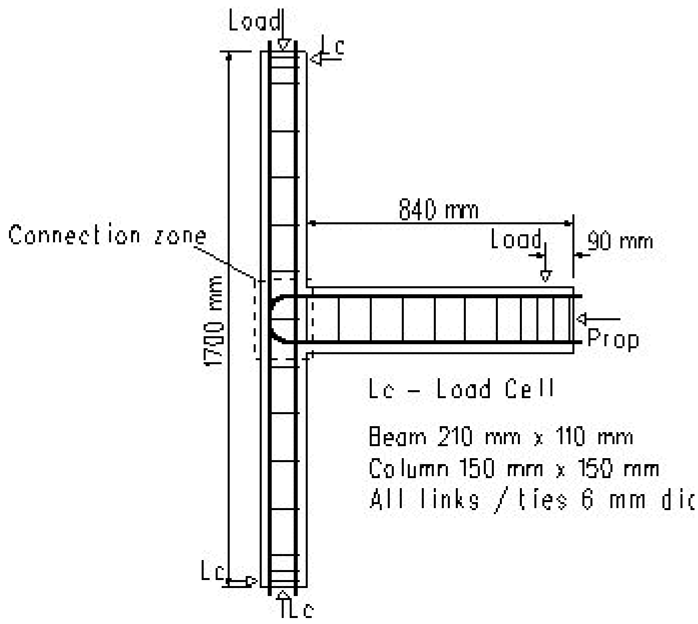

Figure 1. Geometry of a typical specimen

The technique used to measure reinforcement strains 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 received extensive development at Durham University (Scott 1992,1994) It is believed that this is the first test programme to use links that have been strain gauged in this fashion and specimens containing up to three gauged links have been successfully manufactured and tested.

Table 1. Specimen details

Specimen	Concrete Type	Beam Steel Detail	No. of Joint Zone Links
C6LN0	NSC	U	0
C6LN1	NSC	U	1*
C6LN3	NSC	U	3*
C6LN5	NSC	U	5
C6LH0	NSC	U	0
C6LH1	NSC	U	1*
C6LH3	NSC	U	3*
C6LH5	NSC	U	5
C4ALN0	HSC	D	0
C4ALN1	HSC	D	1*
C4ALN3	HSC	D	3*
C4ALN5	HSC	D	5
C4ALH0	HSC	D	0
C4ALH1	HSC	D	1*
C4ALH3	HSC	D	3*
C4ALH5	HSC	D	5

* denotes strain gauged connection zone links.
NSC: Normal Strength Concrete
HSC: High Strength Concrete
U: U-bar
D: Beam reinforcement bent down into column

The test procedure was to load the column first and then maintain the column load while the beam was loaded incrementally, vertically downwards, at a point 90 mm in from its free end. The column loads used were 50 kN for normal strength concrete and 100 kN for high strength concrete, both of which produced strains of around 100 microstrain in the column bars.

Beam loading caused a diagonal crack to form in the connection zone of each specimen, followed, in due course, by failure of the specimen itself either by a plastic hinge forming in the beam at the face of the column or by further extensive cracking of the connection zone. A full set of strain gauge readings were recorded at each load increment and typically there were around 50 load increments per test generating a total of around 12000 strain gauge readings.

3. SPECIMEN BEHAVIOUR

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.

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, before the beam reached its ultimate moment, then a joint shear failure occurred. Failure mechanisms are listed in Table 2.

Table 2. Specimen behaviour

Specimen	P_{crack} (kN)	P_{fail} (kN)	Failure Type	Specimen	P_{crack} (kN)	P_{fail} (kN)	Failure Type
C6LN0	19	24	J	C4ALN0	13	27	J
C6LN1	18	25	J	C4ALN1	19	34	J
C6LN3	18	29	J	C4ALN3	13	35	J
C6LN5	15	34	J	C4ALN5	14	39	J
C6LH0	25	36	J	C4ALH0	21	43	J
C6LH1	22	37	J	C4ALH1	20	43	B
C6LH3	23	41	J	C4ALH3	25	46	B
C6LH5	26	51	B	C4ALH5	27	49	B

J: Joint shear failure
B: Beam failure

The load for initial joint cracking (Pcrack in Table 2) was dependent mainly on concrete strength, although the C6 specimens appeared to perform better than the C4A specimens (possibly beacause the U-bar reinforcement detail had more volumetric steel in the connection zone and also provided a more rigid core than was achieved by bending the steel down into the column). Initial joint cracking was largely unaffected by the number of shear links in the connection zone.

The ultimate moment of resistance of the beam was reached at a beam load of around 48 kN. For a specimen to fail in this mode it had to be strong enough to withstand the shear force that this beam load transferred into the connection zone. 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. It is clear from Table 2 that the high strength concrete gave increased shear strength in the joint. The bent-down beam reinforcement detail was seen to perform better than the U-bar detail due to the anchor leg of the bent-down bar transferring the beam load into the column below the connection zone. A higher number of connection zone shear links also increased the ultimate shear strength due to the joint core being more confined.

4. LINK BEHAVIOUR

The strain gauged links performed well. With the test programme still in progress it is premature to present detailed results, but a number of trends can be identified to illustrate their effectiveness.

4.1 Single link

Strain distributions for the single links in specimens C6LN1 and C6LH1 are shown in Fig 2. These were both U-bar specimens which differed only in the concrete type used (strain values are the average of the ten strain gauges within each link).

Joint cracking, at a link 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 link (1510 microstrain). C6LH1 withstood a greater joint shear force and thus a significantly higher link strain was recorded when failure occurred (7290 microstrain).

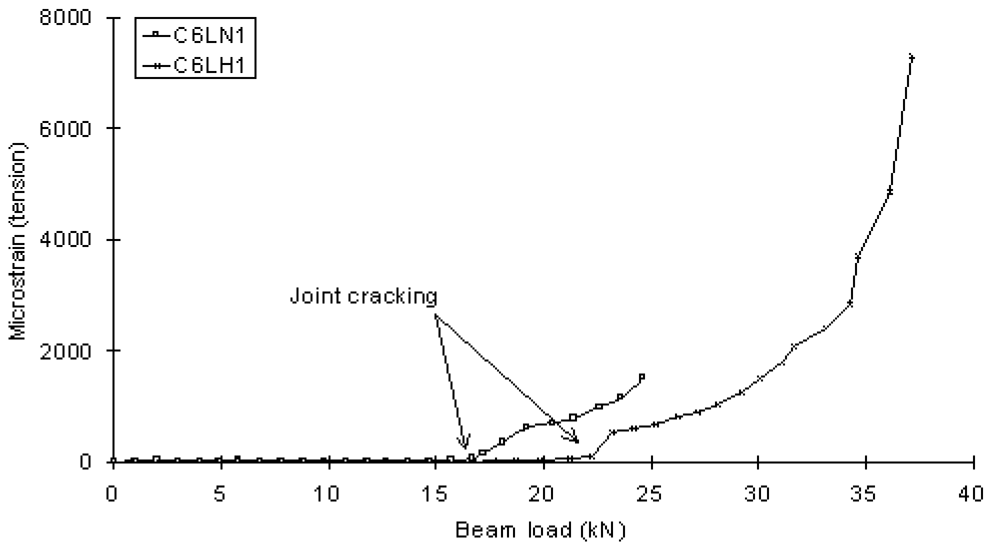

Fig 2. Strain distributions for the single links in specimens C6LN1 and C6LH1

Fig 3 shows differences in behaviour between the two beam steel details. Similar 'U-bar' and 'bent-down bar' specimens are plotted (C6LH1 and C4ALH1 respectively) and again average link 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.

After joint cracking there was a clear change in the rate at which tensile strains increased in the two links. 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 link strain of only 2020 microstrain in spite of its failure load being nearly 7kN 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.

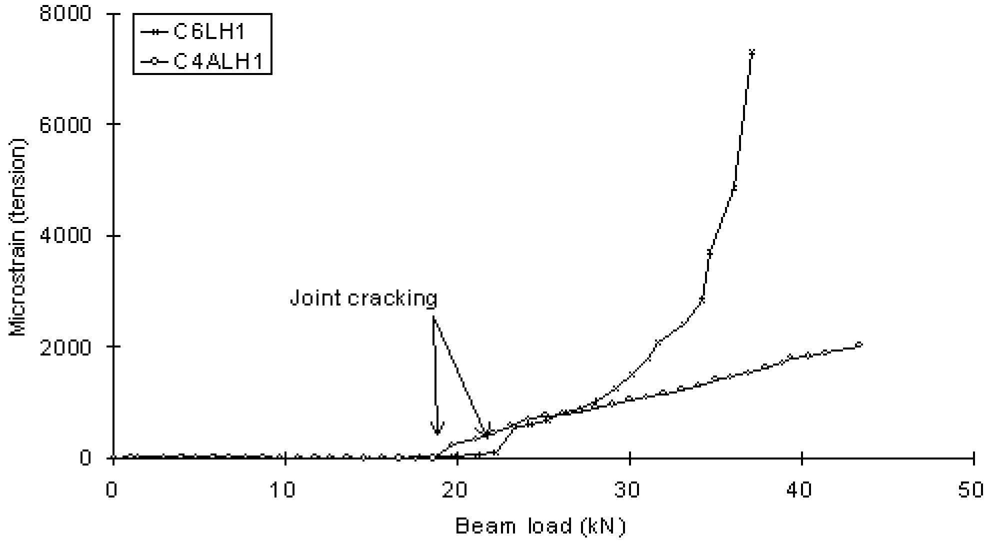

Fig 3. Strain distributions for the single links in specimens C6LH1 and C4ALH1

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.

4.2 Load sharing between links

Fig 4 shows strains for the three connection zone links in specimen C6LN3, the normal strength U-bar specimen. For comparison, the bold line shows strains for the single link in the similar specimen C6LN1.

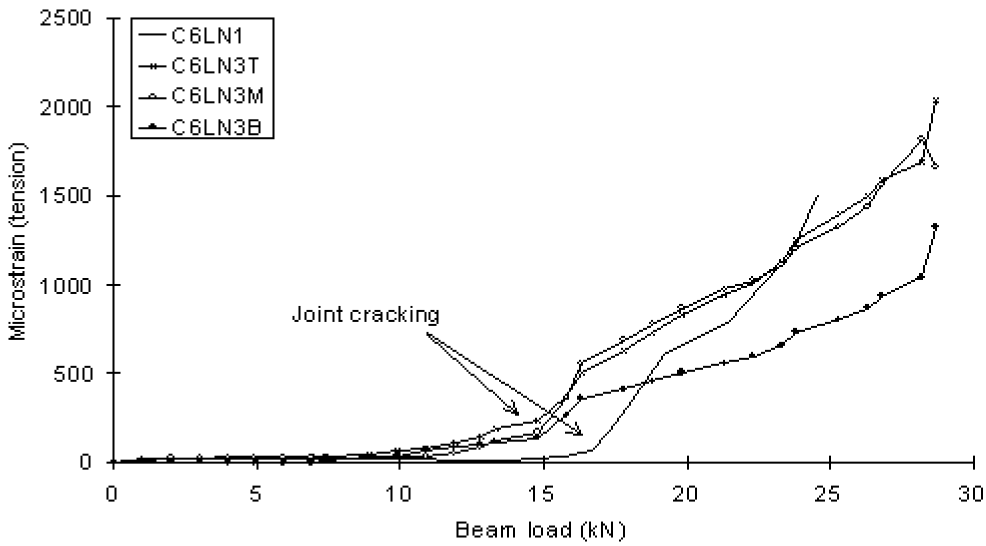

Fig 4. Strain distributions for the link in specimens C6LN1 and C6LN3

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

Results from the three link specimens suggest that when only a single link 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 link would enhance joint shear capacity significantly, perhaps towards that for a three link specimen. Further work is currently in progress at Durham to address this issue.

5. CONCLUSIONS

1. Sixteen external reinforced concrete beam-column connections specimens were tested. Eight specimens had internally strain gauged reinforcement which included strain gauged links in the connection zone.

2. Connection zone links 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 links.

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 links in the connection zone increased the ultimate shear strength of the joint, due to the enhanced confinement of the joint core.

6. REFERENCES

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