

Developments in high strength concrete beam-column connection design

S.J. Hamil and R.H.Scott
University of Durham, UK

[Abstract](#) - [Introduction](#) - [Specimen Details](#) - [Bar Gauging Technique](#) - [Test Procedure](#)
[Results and Discussion](#) - [Conclusions](#) - [Acknowledgements](#) - [References](#)

ABSTRACT. The presence (or absence) of column 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 high strength concrete and the results compared with those from a similar set of normal strength specimens. Compressive strengths as high as 132 MPa and tensile strengths of up to 5.5 MPa were obtained. Two methods of anchoring the beam steel rebars within the connection zone were used - bending the tension bar down into the column and using a U-bar. The technique of internally strain gauging the reinforcement was used for measuring strains in the main beam and column reinforcement in addition to tie strains within the connection zone. Results are presented to compare the performance of the high strength specimens with those using normal strength concrete.

Keywords: Reinforced concrete, High strength concrete, Beam-column connections, Strain gauged reinforcement

Stephen J Hamil is a Research Student at the University of Durham in the U.K. He is currently working towards his PhD by investigating the monotonic and seismic loading of reinforced concrete beam-column connections. He is in his final year of study.

Dr Richard H Scott is a Reader in Structural Engineering at the University of Durham. He spent ten years in industry, designing a wide range of structures in reinforced concrete, structural steelwork, load bearing brickwork and timber before joining the University of Durham in 1978. His research interests include the behaviour of reinforced concrete structural elements, particularly the measurement of reinforcement strain and bond stress distributions.

INTRODUCTION

Uncertainties exist regarding reinforced concrete beam-column connection behaviour due to the fact that it is governed by a number of variables including shear, bond and confinement which are not fully understood in themselves. Extensive world-wide research has been undertaken to investigate the behaviour of beam-column connections both under monotonic loading and simulated seismic loading. Taylor [1] addressed the problem of monotonic loading in 1974 and carried out 26 tests which resulted in BS8110's recommendations [2] for avoidance of shear cracking and connection zone steel congestion. At around the same time at the Polytechnic of Central London, Ryan [3] conducted a series of 12 tests. Both Taylor and Ryan identified the problem, that unless the beam steel percentage was limited, premature shear failure could occur within the connection zone. Sarsam and Phipps [4] followed up this work and linked it with the results from Meinheit and Jirsa's [5] cyclic work to present refined equations for initial shear cracking and subsequent failure. Scott [6,7] 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) [8]. Recently there have been three PhD's presented within the U.K. proposing strut and tie modelling techniques for beam-column connections. Reys de Ortiz [9], Parker [10] and Vollum [11] have all presented strut-tie models using different approaches.

Although there has been much research world-wide regarding the monotonic loading of reinforced concrete beam-column connections there is, however, an absence of data concerning the strains developed within the column ties. It is also felt that there is an absence of data concerning the performance of high strength concrete under monotonic loading. Consequently, Scott's work [6,7] has been continued by conducting further tests in which the connection zone ties 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 ties have been successfully

manufactured and tested to give a total of around 240 strain gauges in each specimen.

SPECIMEN DETAILS

Sixteen reinforced concrete specimens, having the geometry indicated in Figure 1, have been tested as part of a larger programme of 70 specimens. 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 rebars being used for the column reinforcement and a pair of T16 rebars for the main beam tension steel. Column ties and beam links were all mild steel and 6 mm in diameter. Parameters investigated were concrete strength, beam tension steel detail and presence (or absence) of connection zone ties. 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 [6,7] has been retained for consistency). The high strength concrete had a compressive strength in the range of 120-130 MPa and a tensile strength, by the cylinder splitting (Brazilian) test, in the range of 4-5 MPa. The mix used a moisture/binder ratio of 0.25 and incorporated micro-silica and a superplasticiser. Typical cube strengths for the normal strength specimens were 50-60 MPa.

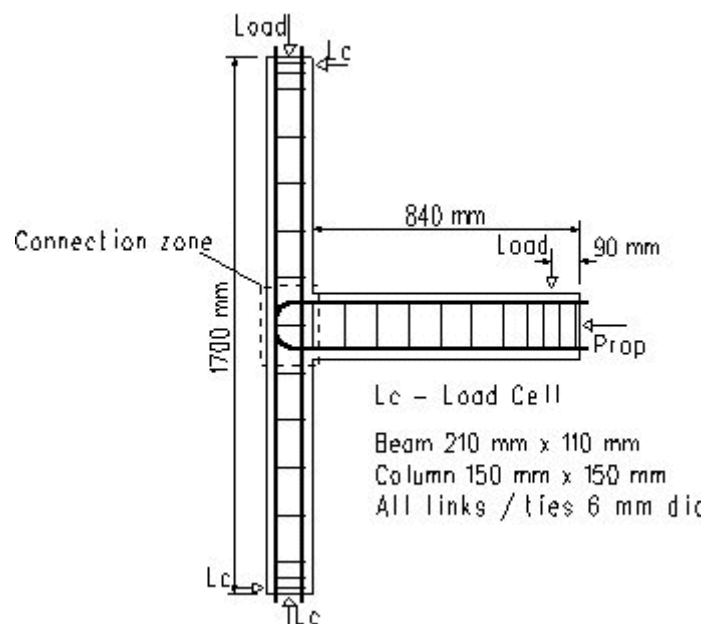


Figure 1 Geometry of a typical specimen

Table 1 Specimen details and behaviour

Specimen	Comp Strength (MPa)	Tensile Strength (MPa)	Beam Steel Detail	No. of Joint Ties	P_{crack} (kN)	P_{fail} (kN)	Failure Type
C6LN0	64	3.4	U	0	19	24	J
C6LN1	64	3.3	U	1*	18	25	J
C6LN3	61	3.2	U	3*	18	29	J
C6LN5	46	2.7	U	5	15	34	J
C6LH0	126	4.7	U	0	25	36	J
C6LH1	127	4.9	U	1*	22	37	J
C6LH3	121	3.7	U	3*	23	41	J
C6LH5	125	5.2	U	5	26	51	B
C4ALN0	53	2.7	D	0	13	27	J
C4ALN1	57	3.4	D	1*	19	34	J

C4ALN3	52	2.9	D	3*	13	35	J
C4ALN5	63	3.2	D	5	14	39	J
C4ALH0	130	4.9	D	0	21	43	J
C4ALH1	119	3.7	D	1*	20	43	B
C4ALH3	132	5.5	D	3*	25	46	B
C4ALH5	123	5.0	D	5	27	49	B

* denotes strain gauged connection zone column ties

U: U-bars, D: Beam reinforcement bent down into column

J: Joint failure, B: Beam failure

BAR GAUGING TECHNIQUE

The technique used to measure the 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. For the purpose of the research programme 12 and 16 mm diameter steel rebars were strain gauged with up to 70 gauges within each bar. It is believed that this is the first test programme to use ties that have been strain gauged in this fashion and specimens containing one and three gauged ties were successfully manufactured and tested. This technique has received extensive development at the University of Durham [12].

TEST PROCEDURE

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. Column loads used were 50 kN for normal strength concrete and 100 kN for high strength concrete specimens, 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 12 000 strain gauge readings.

RESULTS AND DISCUSSION

Specimen Behaviour

All specimens exhibited flexural cracking in the beam and the column regions followed by diagonal cracking in the connection itself, as shown in Figure 2(a). Further shear was then carried by the concrete struts between the cracks assisted by the confinement provided by the connection zone ties.

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, as shown in Figure 2(b). If excessive shear cracking developed in the connection zone, before the beam reached its ultimate moment, then a joint failure occurred, as shown in Figure 2(c). Failure mechanisms are listed in Table 1.

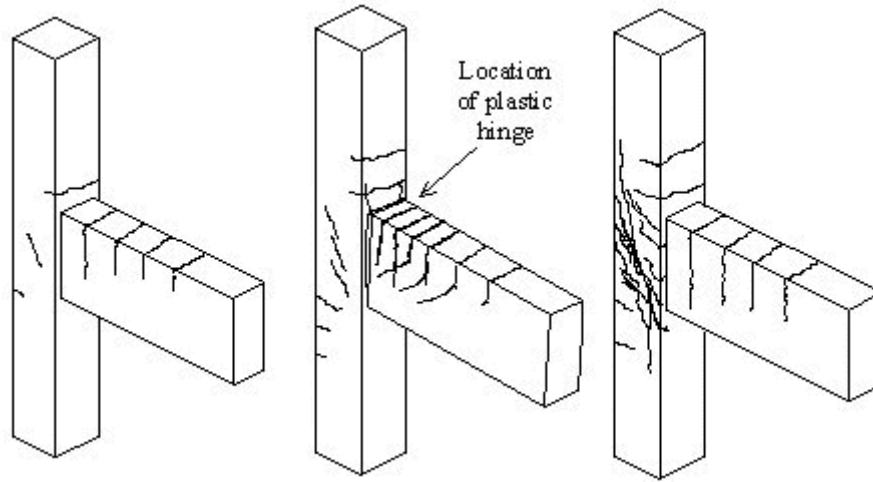


Figure 2(a)
Initial joint cracking

Figure 2(b)
Beam plastic hinge

Figure 2(c)
Extensive joint cracking

Initial Joint Cracking

The load for initial joint cracking (P_{crack} in Table 1) was dependent mainly on concrete strength although the C6 specimens appeared to perform better than the C4A specimens (possibly because the U-bar reinforcement detail had a greater volume of 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 unaffected by the number of ties within the connection zone.

Specimen Failure

Figure 3 shows a comparison of performance between similar specimens made from normal and high strength concrete. The enhancement, given as a percentage, is indicated on the chart.

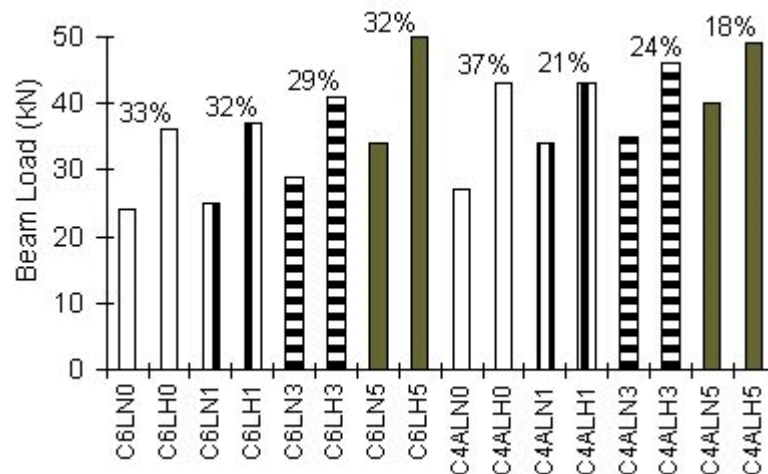


Figure 3 Comparisons of performance between normal and high strength concrete

All the U-bar specimens achieved an increased performance of around 30% when using high strength concrete. The bent-down specimens all achieved a similar increased performance or changed a joint failure into a beam failure.

Throughout the testing it was clear that specimens with the main beam steel bent down into the column performed better than the specimens using the U-bar detail. It was believed that the bent down steel detail had better anchorage due its anchor leg transmitting the beam's force into the column away from the joint. All of the beam's force within the U-bar specimen was transmitted into the column within the connection zone, accelerating joint failure.

Tie Strains

High strength concrete's ability to sustain a greater shear force within the joint before failure

was the reason why the high strength concrete specimens performed so much better than the normal strength specimens. Figure 4 shows the individual strain values within the connection zone ties for specimens C6LN1 and Figure 5 shows the equivalent strains for C6LH1.

Figures 4 and 5 show that strain in the connection zone tie for both normal strength and high strength specimens was very low until joint cracking first occurred. After joint cracking the strain in the ties steadily increased until the connection zone failed due to shear cracking. Whereas the tie within the normal strength specimen, C6LN1, displayed strains of approximately 1500 microstrain at failure the tie within the high strength specimen, C6LH1, yielded and strains of over 16000 microstrain were recorded.

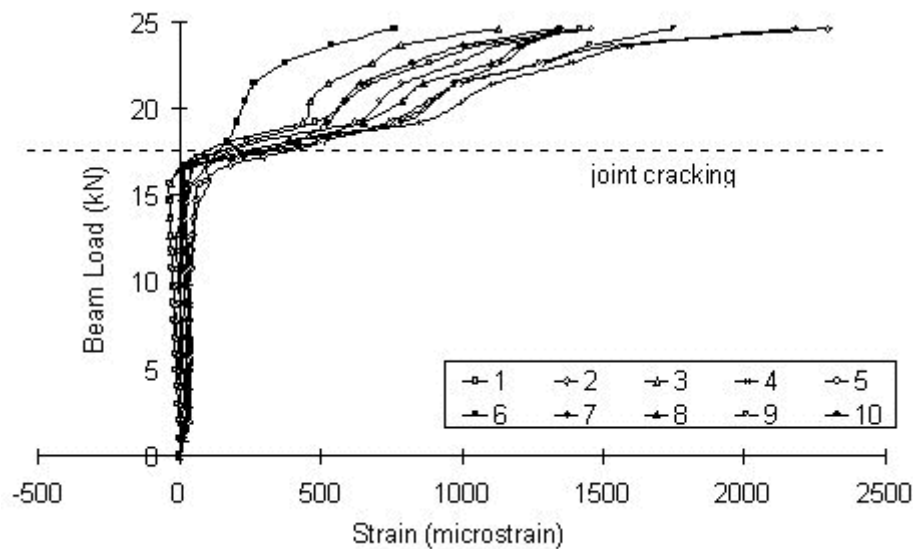


Figure 4 Tie strains within the specimen C6LN1

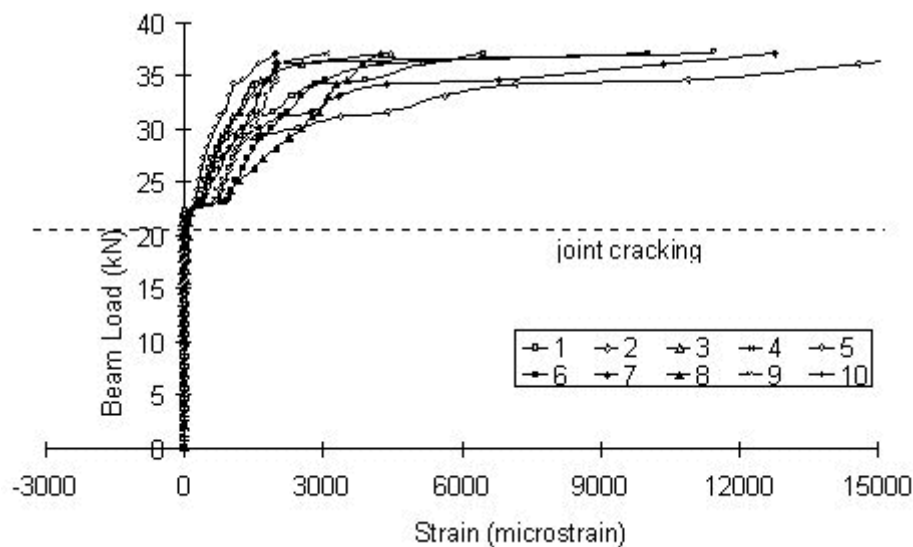


Figure 5 Tie strains within the specimen C6LH1

Figure 6 shows average strains for the three connection zone ties in specimen C6LH3, the high strength U-bar specimen. For comparison, the bold line shows the average strains for the single tie in the similar specimen C6LH1.

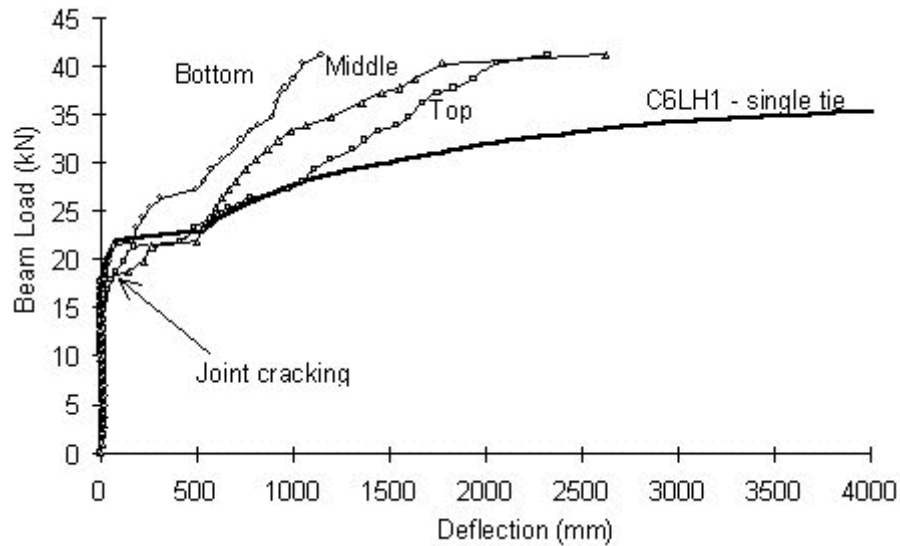


Figure 6 Strain distribution for the ties in specimens C6LH1 and C6LH3

It can be seen that load was not shared equally between the three ties within C6LH3. After joint cracking strains in the top and middle are significantly higher than those in the lower tie, probably due to the beam tension steel causing the top of the joint to be more strained than the bottom. Specimen C6LH1, with only a single tie, went through joint cracking at approximately the same load as the three tie specimen. After joint cracking the single tie in specimen C6LH1 is seen to develop strain at a higher rate than the three ties in C6LH3. As stated previously, the single tie within C6LH1 yielded and strains as large as 16 000 microstrain were observed.

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 were positioned between the mid height of the joint and the level of the main beam tension steel. It is recommended that if, in the design of reinforced concrete connections, shear resistance is of greatest concern then joint ties should be positioned around the mid-height level of the connection zone. If an enhancement of the beam steel anchorage is sought then the ties should be positioned around the level of the main beam steel.

Average Bond Stresses

The beam rebars contained four strain gauges within their top bend, one at the start, one at the end and two equally spaced gauges within the bend. The strain gauges within the top bend allowed detailed values of bond stress to be calculated. A comparison of the average bond stresses around this top bend is given in Figure 7. The average bond stress values were calculated by taking the average of the stress gradients between the four strain gauges within the top bend.

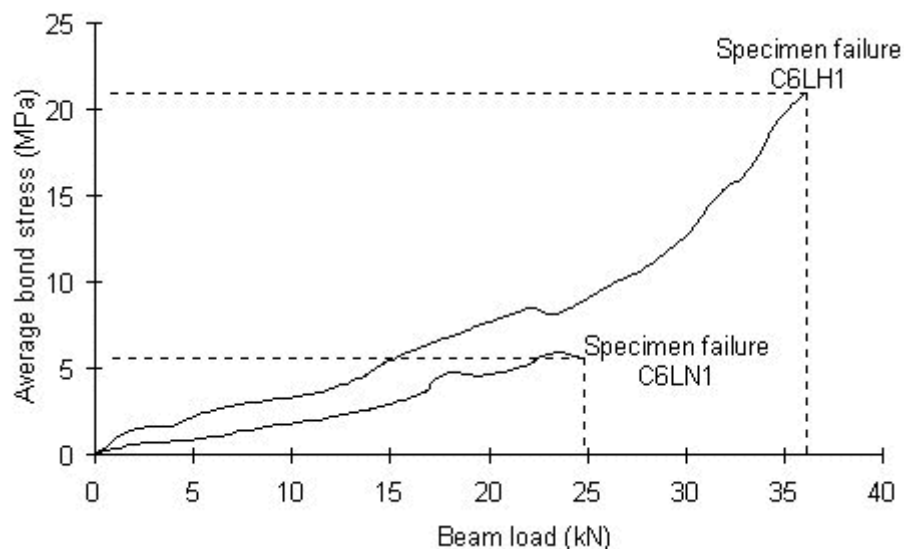


Figure 7 Average bond stress values for high and normal strength specimens

Both specimens C6LN1 and C6LH1 failed due to excessive joint cracking, the high strength specimen achieved a failure load over 30 % greater than the normal strength specimen. The ability of the high strength specimen to attain a greater bond stress around the top bend allowed the joint to resist a greater shear force before failure.

Load-Deflection Response

Figure 8 shows the load-deflection response of four specimens over their initial loading. All four specimens failed at a beam load of approximately 40 kN thus a direct comparison of stiffness over the initial load history is made.

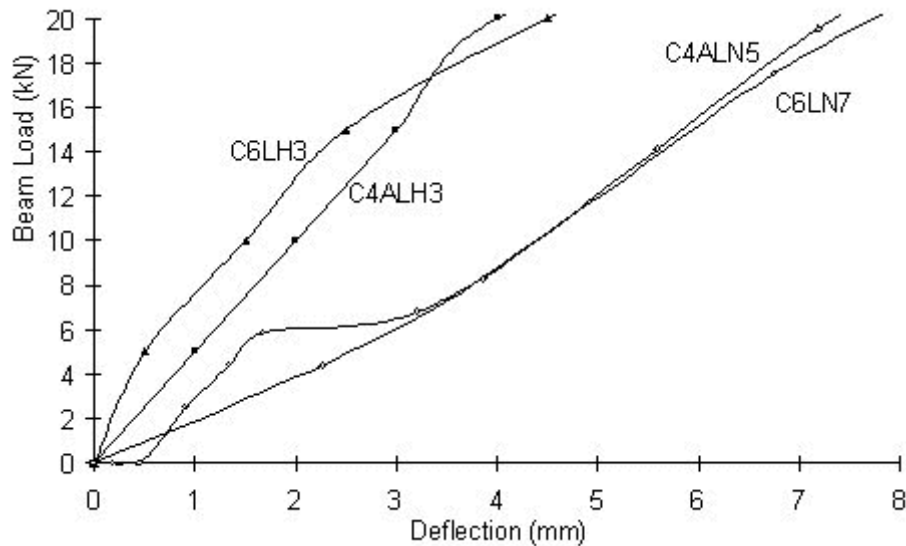


Figure 8 Load-deflection curves for normal and high strength specimens

Although all of the specimens failed at a similar load it can be seen that there is a clear difference in behaviour between those made with high and normal strength concrete. Both of the high strength specimens were around twice as stiff as the normal strength specimens.

It is suggested that a larger safety factor might be more appropriate for high strength structural design as a shear failure would occur with less warning.

CONCLUSIONS

1. Initial joint cracking was strongly influenced by the tensile strength of the concrete within the connection zone. Specimens made with high strength concrete had a greater resistance to initial joint cracking due to their greater tensile strength.
2. The number of connection zone ties did not effect the performance of the joint until after joint cracking had occurred.
3. Use of high strength concrete increased the ultimate shear strength of the joint and allowed higher strains to be developed within the joint zone 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 within the connection zone increased the ultimate shear strength of the joint, due to the enhanced confinement of the joint core.
6. The load-deflection response of specimens made using high strength concrete was seen to be twice as stiff compared with specimens made using normal strength concrete.

ACKNOWLEDGEMENTS

The financial support of the Engineering and Physical Sciences Research Council is gratefully acknowledged as are the contributions made by Tarmac Topmix Ltd, Mr Paul Brayshaw and the technical staff of the School of Engineering.

REFERENCES

1. TAYLOR, H.P.J. : 'The behaviour of in situ beam-column joints', Technical Report 42.492, Cement & Concrete Association, May 1974
2. BS 8110 STRUCTURAL USE OF CONCRETE : Part 1 : Code of practice for design and construction, London, British Standard Institution, 1985
3. RYAN, J. : 'Reinforced concrete beam-column connections', Concrete Magazine, pages 37-39, March 1977
4. SARSAM K.F. AND PHIPPS M.E. : 'The shear design of in situ reinforced concrete beam-column joints subjected to monotonic loading', Magazine of Concrete Research, Vol 37, No 130, March 1985
5. MEINHEIT D.F. AND JIRSA J.O. : 'The shear strength of reinforced concrete beam-column joints', Austin, University of Texas at Austin, Dept. of Civ Eng., 1977, Report 77-1
6. SCOTT R.H. : 'The effects of detailing on RC beam/column connection behaviour', The Structural Engineer, Vol 70, No 18, 15 Sep 1992, pages 318-324
7. SCOTT R.H. ET AL : 'Reinforced concrete beam-column connections and BS 8110', The Structural Engineer, Vol 72, No 4, 15 Feb 1994, pages 55-60
8. BS 8110 STRUCTURAL USE OF CONCRETE : Part 1 : Code of practice for design and construction, London, British Standard Institution, 1993
9. REYS DE ORTIZ I. : 'Strut and tie modelling of reinforced concrete short beams and beam-column joints', PhD thesis, University of Westminster, 1993
10. PARKER D.E. : 'Shear strength within reinforced concrete beam-column joints', PhD thesis, Bolton Institute of Higher Education, 1997
11. VOLLUM R.L. : 'Design and analysis of reinforced concrete beam-column joints', PhD thesis, Imperial College, London, 1998
12. SCOTT R.H. AND GILL P.A.T. : 'Short-term distributions of strain and bond stress along tension reinforcement', The Structural Engineer, 65B, No. 2, June 1987, pages 39-43, 48