EXPERIMENTAL STUDY OF INTERIOR BEAM TO COLUMN CONNECTIONS SUBJECTED TO REVERSED CYCLIC LOADING

A. J. Durrani (I)
J. K. Wight (II)

Presenting Author: A. J. Durrani

SUMMARY

The results of an experimental study in which six reinforced concrete interior beam-column subassemblages were tested under quasi-static earthquake type loadings are presented. The variables were the joint shear stress level, the amount of joint reinforcement and the presence of transverse beams and slab. The performance of specimens is evaluated in terms of strength, stiffness and the energy dissipation capacity. The joint shear stress level was found to be more critical for specimens without transverse beams. The increase in the amount of joint reinforcement was observed to be more effective in improving the behavior of specimens with transverse beams and slab.

INTRODUCTION

Reinforced concrete moment resisting frame buildings are expected to undergo inelastic deformations during earthquakes of moderate to strong intensity. The critical building components must therefore be designed and detailed to withstand large cyclic deformations without any significant loss of strength or stiffness. The beam to column connections constitute one of such critical regions which if not properly designed, could lead to brittle failure and possible collapse of the structure.

BACKGROUND

The behavior of beam to column connections under seismic loading has been studied by several researchers over the last two decades. The objective in each case was to explain the joint behavior and to develop procedures for practical and safe design of connections. The current ACI recommendations (Ref. 1) for the design of connections subjected to cyclic loading, is based on the beam shear model. As in the case of beams, the shear strength of the joint is computed as the sum of concrete shear strength and the contribution of steel calculated using truss analogy. In addition, depending on the type of joint, the joint shear stress is limited to a certain maximum value. However, the applicability of this model to connections is not considered very realistic and this design approach has been found to result in excessive amount of steel in the joint. In recent revision (Ref. 2) of these recommendations, the nominal shear strength of the joint is limited to a maximum allowable value and the joint reinforcement is determined entirely from the confinement considerations.

(I) Assistant Professor, Rice University, Houston, Texas.

(II) Associate Professor, The University of Michigan, Ann Arbor, Michigan.

The New Zealand Code (Ref. 3) prescribes a procedure which utilizes the compression strut mechanism to compute the portion of shear resisted by concrete. The horizontal and vertical joint shear reinforcement is determined using the joint truss mechanism to resist the remaining shear.

Zhang and Jirsa (Ref. 4) proposed the use of inclined compression strut to determine the joint shear strength. The transverse reinforcement is primarily assumed to maintain the integrity of the compression strut and its contribution to the shear capacity of the joint is considered rather insignificant.

The recommendations of the Applied Technology Council (Ref. 5) are based on the study by Meinheit and Jirsa (Ref. 6). While these recommendations recognize the compression strut mechanism in the joint, they make no specific requirement for shear reinforcement.

These three design procedures result in considerably different amounts of transverse reinforcement in the joint. As such, the shear resistance mechanism and the design of joints is still open to further research. However, the most important factors which influence the joint behavior have been identified by various researchers (Refs. 7 to 10). These include the joint shear stress level, concrete strength, joint reinforcement, confinement by lateral beams and the column axial load.

EXPERIMENTAL STUDY

This experimental study of interior beam-to-column connections was aimed at investigating the effect of joint shear stress level (650 psi to 850 psi), the amount of joint hoop reinforcement (2% to 3% by volume), and the presence of lateral beams and slab on the behavior of joints subjected to reversed cyclic loading. The test specimens without lateral beams are designated as X-series specimens and the specimens which had lateral beams and slab are called S-series specimens. The design construction and the instrumentation details of these specimens are described in detail in Ref. 10. The parametric details of both series are given in Table I.

TABLE I. Parametric Details of Test Specimens

Specimen	Concrete strength psi	$\frac{\Sigma M \text{ col.}}{\Sigma M \text{ beams}}$	Joint* shear stress psi	Joint** reinforcement %	JPI Ω
X1	4980	1.5	866	2.06	12.1
X2	4880	1.5	866	3.10	10.0
X3	4500	1.4	650	2.06	8.8
S1	6030	1.2	808	2.06	10.0
S2	4480	1.2	820	3.10	7.7
S3	4100	1.2	648	2.06	9.5

* Based on the gross column area.

** Volumetric percentage of joint reinforcement.

Note: $1000\sqrt{f_c'(\text{psi})} = 83\sqrt{f_c'(\text{MPa})}$

Test Specimens

All specimens had an overall dimension of 8.0 ft. x 14.0 ft. and the beam and column sizes were 11 in. x 16.5 in. and 14.5 in. x 14.25 in., respectively (Note: 1 ft. = 0.3048m, 1 in. = 25.4mm). The width of slab in S-series specimens was 39½ inches. On average the column axial load was approximately twenty percent of the balanced load. Fig. 1 illustrates the testing arrangement. In calculating the flexural capacity of beams, the full width of slab was considered effective. The joint shear stress is calculated by

$$V_j = 1.25 f_y (A_{st} + A_{sb}) - V_{col}$$

where f_y is the yield stress of hoop steel, A_{st} and A_{sb} are the areas of top and bottom steel in beams and V_{col} is the shear in column when beams reach their ultimate capacities. In S-series specimens, the torsion in lateral beam introduces additional shear in the joint. The exact amount and mechanism of this indirect loading of joint is not clearly understood yet. For the purpose of this study, reinforcement in half of the width of slab was assumed to contribute toward the joint shear.

Loading History

The beam-column subassemblages were subjected to slow reversed cyclic displacements. The first cycle represented the yielding of beam main reinforcement. The subsequent cycles were controlled in terms of the yield cycle displacements. In S-series specimens, it was difficult to ascertain yield cycle displacement due to the progressive yielding of slab reinforcement. The X-series specimens yielded at displacement levels approximately three-fourths of the S-series specimens. Consequently, for the same amount of maximum displacement, the X-series specimens appeared to have more ductility. A typical displacement routine is shown in Fig. 2. On average each specimen was subjected to seven cycles of load reversals which amounted to an average cumulative ductility of 30 for X-series specimens and 25 for S-series specimens. The cumulative ductility is defined as $2(\mu_i \times n_i)$ where n_i is the number of cycles at ductility μ_i. In each case, the maximum displacement attained during the test corresponded to a relative column end displacement of six percent.

Test Results

The behavior of each specimen is evaluated in terms of its ability to maintain strength, stiffness and the energy dissipation capacity during the loading cycles. A typical load-displacement response of specimens X2 and S2 is shown in Fig. 3a and 3b, respectively. Both of these specimens had approximately the same level of joint shear stress and the same amount of joint reinforcement. However, the specimen S2 showed relatively superior load resistance and dissipated more energy. The strength envelopes of test specimens are shown in Fig. 4. Except for specimen X1, there is no significant loss in strength of specimens at displacements corresponding to a story drift of three percent. However, at the end of the seventh cycle, the X-series specimens exhibited a drop in strength of approximately twenty percent. For the given joint shear stress level and the amount of transverse reinforcement, the S-series specimens did not experience any significant loss of strength.

The stiffness degradation of specimens is shown in Fig. 5. All specimens appear to have suffered a continuous loss of stiffness, although the S-series specimens maintained higher stiffness than the X-series specimens. The higher stiffness of the specimen S1 compared to other specimens is attributed to the higher strength of concrete. The energy dissipated by each specimen during the loading cycles is given in Fig. 6. Specimens S2 and X3 dissipated more energy than the other specimens. The larger energy dissipation by specimen X3 can be attributed to the lower level of joint shear stress. On the other hand, specimen S2 dissipated more energy because of the joint core being well confined by the joint hoop reinforcement and the lateral beams.

The slippage of beam and column bars was relatively higher among X-series specimens than the S-series specimens. Except for the specimen X1, which showed the slippage of beam bars during the second cycle, the slippage of reinforcement through the joint core for the remaining specimens could be considered negligible. However, the column bars showed more slippage than the beam bars.

All S-series specimens attained a ductility level of 3 without losing strength by more than ten percent. With the exception of specimen X1, the X-series specimens were able to reach an average ductility level of 3.5 with a reduction in strength of not more than twenty percent. In the absence of any specific performance criteria required by the current recommendations, the behavior of these specimens could be regarded as satisfactory. Because the stiffness is also an important consideration in the satisfactory performance of connections, the relative performance of specimens may be evaluated in terms of their energy dissipation capabilities. Knowing the relative effect of joint shear stress, joint reinforcement, concrete strength, and the lateral beams on the behavior of connections, a comparative performance index could be developed. Based on the performance of these specimens, an empirical joint performance index for specimens without lateral beams can be calculated by

$$\Omega = \frac{v^{1.3}}{2\sqrt{p_v f_c'}}$$

and for specimens which had transverse beams and slab, it is given by

$$\Omega = \frac{v^{1.1}}{p_v\sqrt{f_c'}}$$

where Ω is the performance index, p_v is the volumetric percentage of joint steel ratio, f_c' is the concrete strength (psi units) and v is the joint shear stress (psi units). The joint shear stress is calculated by

$$v = \frac{1.25 f_y(A_{st}+A_{sb}) - V_{col}}{b_c h_c}$$

However, V_{col} is usually about 25% of $f_y(A_{st}+A_{sb})$. Hence, the joint shear stress is approximately given by

$$v = \frac{f_y(A_{st}+A_{sb})}{b_c h_c}$$

where b_c and h_c are the width and the height of column.

Although such an index is not explicitly based on specific performance criteria, it reflects the relative significance of each parameter considered important for the satisfactory performance of connections. The performance index of each specimen calculated as above is given in Table I. The relative performance of these specimens as given by their performance index is inversely proportional to their energy dissipation capabilities as shown in Fig. 6. The specimens which have a performance index of less than ten could be expected to behave satisfactorily. It should be noted that this criterion is intended for comparison purposes only. However, it may be used for determining the joint reinforcement for a given joint shear stress level and concrete strength. In such case, a joint reinforcement ratio of 3 percent and concrete strength of 6000 psi should be considered as upper limits.

Conclusions

(1) The behavior of connections without transverse beams was found to be significantly more sensitive to the level of joint shear stress compared to the specimens with transverse beams and slab.

(2) An increase in the amount of joint reinforcement improved the performance of specimens with transverse beams and slab proportionately more than it did for specimens without transverse beams and slab.

(3) Specimens which had a lower joint shear stress dissipated relatively more energy than the specimens which had a higher joint shear stress and a larger amount of confining reinforcement.

(4) Higher strength of concrete helped reduce the stiffness degradation of joints under cyclic loading.

(5) Column bars showed more slippage through the joint core than the beam bars.

Acknowledgment

The research reported in this paper was made possible by a grant from the National Science Foundation (No. NSF PFR 78-24556). The experiments were carried out in the G. G. Brown Laboratory of the University of Michigan, Ann Arbor.

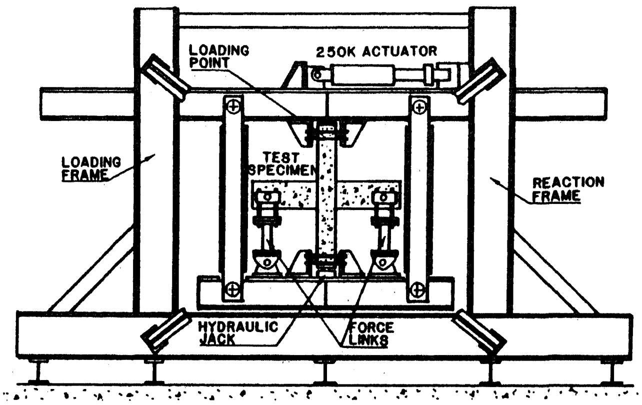

Fig. 1 Test setup

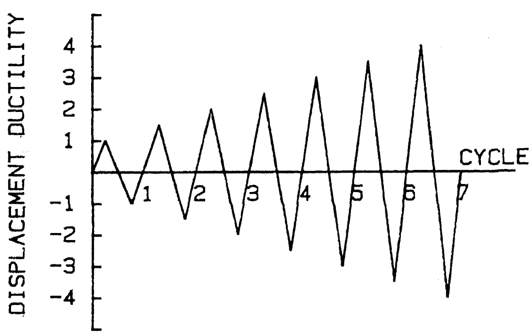

Fig. 2 Cyclic loading routine

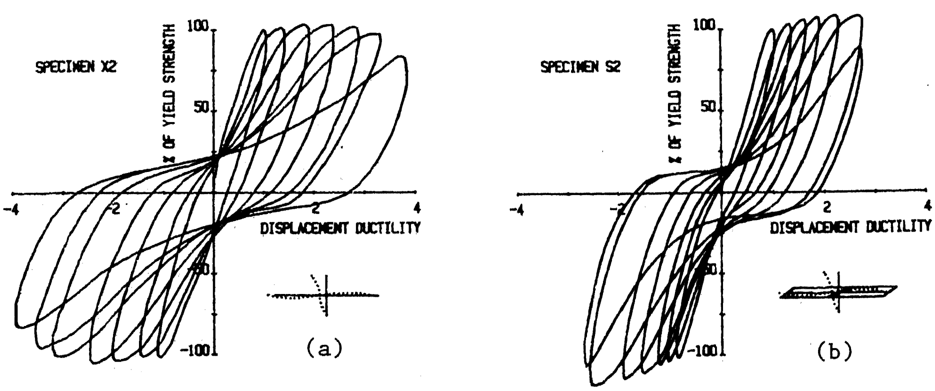

Fig. 3 Column load point vs displacement ductility curves

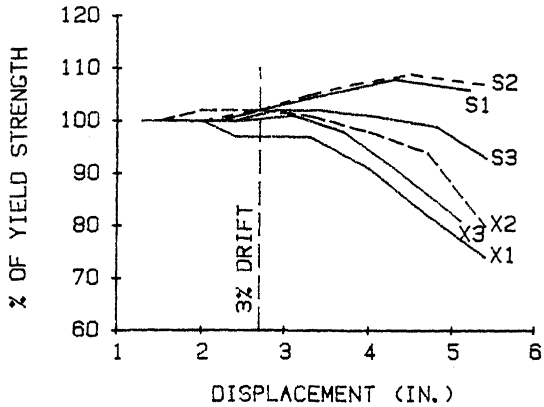

Fig. 4 Strength envelopes of subassemblages

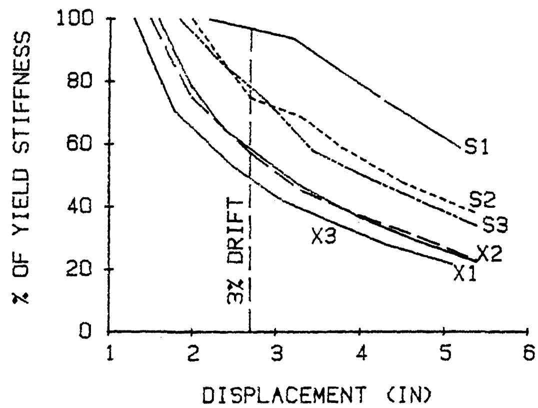

Fig. 5 Stiffness degradation of specimens

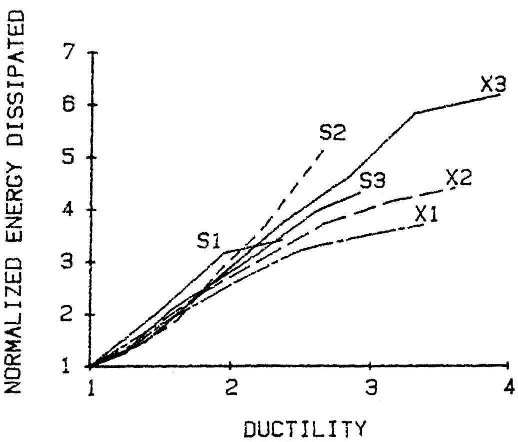

Fig. 6 Relative energy dissipated by each specimen

REFERENCES

1. ACI-ASCE Committee 352, 1976, "Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures," ACI Journal (July 1976), pp. 375-393.

2. ACI-ASCE Committee 352, Revised Recommendations with Commentary, June 1982. (unpublished draft)

3. New Zealand Standard Code of Practice for the Design of Concrete Structures, NZS 3106 Parts 1 & 2:1982, Standards Association of New Zealand, Wellington, 1982.

4. Zhang, L. and Jirsa, J. O., "A Study of Shear Behavior of Reinforced Concrete Beam-Column Joints," PMFSEL Report No. 82-1, The University of Texas, Austin, February 1982.

5. Applied Technology Council,"Tentative Provisions for the Development of Seismic Regulations for Buildings," National Bureau of Standards, Special Publication 510, June 1978.

6. Meinheit, D. F. and Jirsa, J. O., "The Shear Strength of Reinforced Concrete Beam-Column Joints," CESRL Report No. 77.1, The University of Texas, Austin, January 1977.

7. Hanson, N. W. and Conner, H. W., "Seismic Resistance of Reinforced Concrete Beam-Column Joints," Journal of the Structural Division, ASCE, No. ST5, October 1967, pp. 533-560.

8. Uzumeri, S. M. and Seckin, M., "Behavior of Reinforced Concrete Beam-Column Joints Subjected to Slow Load Reversals," Report No. 74-05, Department of Civil Engineering, University of Toronto, March 1974.

9. Scribner, C. F. and Wight, J. K., "Delaying Shear Strength Decay in Reinforced Concrete Flexural Members Under Large Load Reversals," Report No. UMEE 78R2, Department of Civil Engineering, University of Michigan, May 1978.

10. Durrani, A. J. and Wight, J. K., "Experimental and Analytical Study of Internal Beam to Column Connections Subjected to Reversed Cyclic Loading," Report No. UMEE 82R3, Department of Civil Engineering, University of Michigan, July 1982.