PERFORMANCE OF LARGE REINFORCED CONCRETE BEAM-COLUMN JOINT UNITS UNDER CYCLIC LOADING

R W G Blakeley[I], F D Edmonds[II], L M Megget[III], M J N Priestley[IV]

SYNOPSIS

The design and static cyclic (reversed) load testing of three full scale reinforced concrete beam-column assemblies is described. Member details were based on a frame building designed to current New Zealand practice. Extensive instrumentation enabled a detailed assessment of the performance in the joint region and indicated very satisfactory behaviour at deformations expected under severe seismic loading. Stable behaviour was obtained up to maximum beam displacement ductility factors from 6 to 10 for the three units. Results are presented and assessed, and recommendations are made for revisions to common procedures for joint reinforcement and beam hinge detailing.

INTRODUCTION

A programme of cyclic load testing of large reinforced concrete beam-column assemblies was undertaken following the poor joint performance reported in many previous tests[1], particularly bond failure of the beam bars and joint concrete disintegration. Size effects, such as the ratio between bar diameter and member size, and the absence of intermediate column bars may well have contributed to joint failures in smaller test specimens. This test series also differed from most others[2] by applying more critical joint conditions through lower column loads and higher imposed beam rotations.

JOINT SHEAR RESISTING MECHANISMS

The forces acting within joints are shown in Fig. 1. Shear transfer across the panel zone may be idealised as due, in varying proportions, to three mechanisms: arch action, truss action and aggregate interlock. Concrete compression forces tend to be transferred by direct arch action. Those forces induced in the panel zone through bond to the reinforcing bars tend to be transferred by a truss mechanism comprising a number of diagonal compression struts in the concrete and tension ties in the steel. In a conventional joint there are horizontal ties, but the vertical strut components must be resisted by intermediate column bars. Joint details which promote arch action, such as use of prestressing, are desirable.

TEST UNIT DETAILS

The test unit members were equal in size to those of a prototype structure; a 3 by 3 bay, 9 m span, 4-storey reinforced concrete frame designed to the new New Zealand loadings code[3]. The three units comprised two internal joints (Units 1 and 3) and one external joint (Unit 2). Fig. 2 shows the member details. The 1.8 MPa prestress in Unit 3 was designed to balance the prototype structure's floor dead load only. In the design of each unit the concrete was considered to make no contribution to the beam shear strength in the pastic hinge zones. In Units 1 and 2 the tie set spacing of 150 mm in these regions complied with the code[4] maximum of $^{d}/4$

I Design Eng., Ministry of Works and Development, New Zealand.
II Eng., Ministry of Works and Development, New Zealand.
III Sen Eng., Ministry of Works and Development, New Zealand.
IV Sen Lecturer, University of Canterbury, New Zealand.

and with common buckling limitations of 6 bar diameters. In Unit 3 the spacing was reduced to 100 mm. The column reinforcement was derived from a capacity design approach aimed to confine plastic hinging to the beams with the columns remaining elastic. An axial column load of 620 kN (5% axial load capacity) was applied to the internal joints to represent the minimum likely value in the prototype structure. No axial load was applied to the external column.

The joint shear steel was designed to carry the shear force, from yielding of beams in one plane, across a corner to corner diagonal tension crack. An allowance for 25% overstrength on beam bar yield stress and a capacity reduction factor for shear of 0.85 were applied. The contribution of the two short legs in each tie set was neglected. In Units 1 and 2 the concrete was considered to make no contribution to joint shear resistance, and a factor of 2/3 was applied to account for variability in effectiveness of the ties. In Unit 3, because of prestress, the joint concrete was assumed to resist shear in accordance with the code[4], and all tie sets were assumed to be fully effective.

INSTRUMENTATION AND TESTING

Fig. 3 shows the test set-up for the internal joint units. Demec gauges measured joint shear strains. Reinforcing strains for beam and column main steel, and joint ties, were measured by approximately 170 gauges per test unit. Strains were converted to stress using a Bauschinger Analysis. The units were tested under displacement controlled loading with two complete reversed cycles at displacement ductility factors (DF) of 0.75, 2, 4, 6, etc. till failure. Complete test details are given elsewhere[5].

RESULTS

Moment-Deflection. All three units exhibited very satisfactory behaviour. Moment-deflection hysteresis loops are shown in Fig. 4. Theoretical ultimate moment capacities based on 0.3% and 0.4% concrete compression strains and measured material properties are indicated by MU.003 and MU.004. Note that only minor load and stiffness degradation occurred at displacement ductilities below DF = 8, 10 and 12 for the three units respectively. In each case failure resulted from horizontal buckling of main beam reinforcing after loss of cover concrete, and in Units 1 and 2 from excessive sliding shear movements along vertical cracks in the beam. The latter effect was largely a result of the unequal top and bottom steel areas preventing crack closure for much of each cycle. Axial beam prestress and closer tie set spacing markedly improved the beam behaviour of Unit 3.

Steel Stresses. Beam steel stress distributions through the joint at DF = 6 are plotted in Fig. 5(a). Stress distributions within the joint are close to linear, and yield progressed a maximum distance of approximately 150 mm within the joint. Bond and confinement conditions were adequate to allow very high beam steel stress gradients to develop without resulting in the steel slipping through the joint. Column steel stress distributions through the joint at DF = 6 (Fig. 5(b)) show large tensile peaks within the joint region, indicating shear transfer by truss action.

The vertical distributions of joint tie stresses on the longitudinal legs of the tie sets at DF = 6 are shown in Fig. 5(c). Maximum stresses are 70%, 45% and 100% of the nominal yield of 275 MPa, and average stresses for the worst tie sets are 55%, 27% and 100% of the nominal yield value. The low average stress in the external joint is in part a result of the joint steel design being governed by concrete confinement rather than by joint shear. The margin over yield of the tie legs in the unprestressed internal joint unit, and the fact that in the prestressed joint unit yielding of some tie legs did not lead to disintegration of the joint, indicates that design on the basis of average tie stresses rather than peak values is justified and, therefore, the 2/3 "effectiveness" factor is unnecessary.

CONCLUSIONS AND DESIGN RECOMMENDATIONS

1 The performance of the three beam-column assemblies satisfied the anticipated ductility demands of severe seismic loading.

2 Assessment of the joint shear performance from beam and column bar and joint tie stresses gave good agreement with theory based on shear resisting arch and truss mechanisms.

3 There was considerable scatter of stresses within each joint tie set and moderate variation of effectiveness of different tie sets. Although this effect was allowed for in design of Units 1 and 2 by a 2/3 "effectiveness" factor, the results of Unit 3 indicated that yield of isolated tie legs need not lead to joint disintegration and this factor is felt to be unnecessary. However, it is recommended that the concrete be considered to make no contribution to joint shear resistance, other than as diagonal compression struts, in design of conventional reinforced concrete members with moderately low column loads. Where there is significant beam prestressing the improved joint shear transfer may be recognised by normal assumptions of shear carried by the concrete.

4 Very high bond stresses were generated at the beam bars through the joint, and bond failures reported on smaller test members were avoided.

5 It is recommended that column bars be evenly distributed along each face to promote joint shear truss action.

6 Unequal top and bottom beam steel led to wide flexural cracks, large shear displacements and general deterioration of the beam plastic hinge. Details which avoid these problems, such as use of a central prestressing tendon, are desirable. It is recommended that the maximum tie set spacing in the plastic hinge region be 100 mm.

REFERENCES

1 PARK, R and PAULAY, T, "Behaviour of Reinforced Concrete External Beam-Column Joints Under Cyclic Loading", Vol. I, Paper 88, Proceedings, Fifth World Conference on Earthquake Engineering, Rome, 1973, pp 772-781.

2 HANSON, N W, "Seismic Resistance of Concrete Frames with Grade 60 Reinforcement", ASCE, Vol. 97, ST6, June 1971, pp 1685-1700.

3 SANZ, "New Zealand Standard Code of Practice for General Structural Design and Design Loadings for Buildings", NZS 4203, Standards Association of New Zealand, Wellington, 1976, 80 pp.

4 ACI COMMITTEE 318, "Building Code Requirements for Reinforced Concrete (ACI 318-71)", American Concrete Institute, Detroit, 1971, 78 pp.

5 BLAKELEY, R W G, MEGGET, L M, PRIESTLEY, M J N, "Seismic Performance of Two Full Size Reinforced Concrete Beam-Column Joint Units", Bull. N.Z. National Society for Earthquake Engineering, Vol. 8, No. 1, March 1975, pp 38-69.

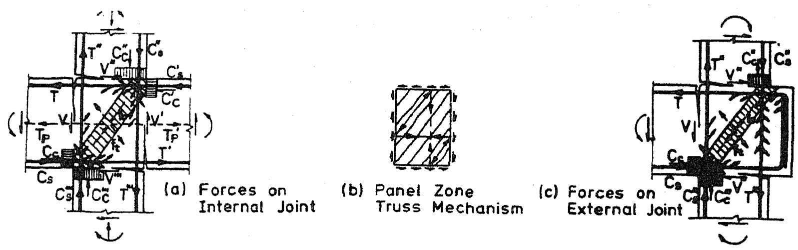

Fig.1 Beam-Column Joint Forces

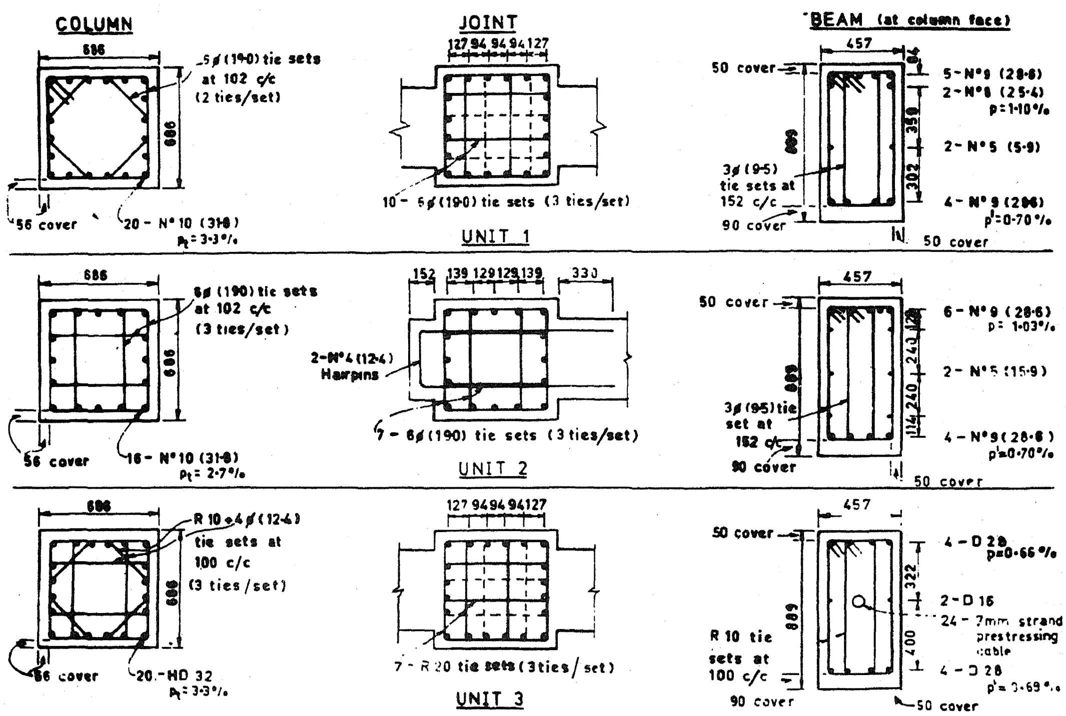

Fig.2 Test Unit Details

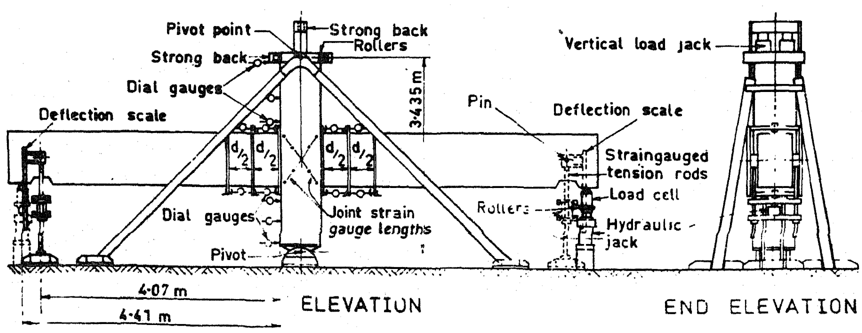

Fig.3 Test Set-Up for Internal Joints

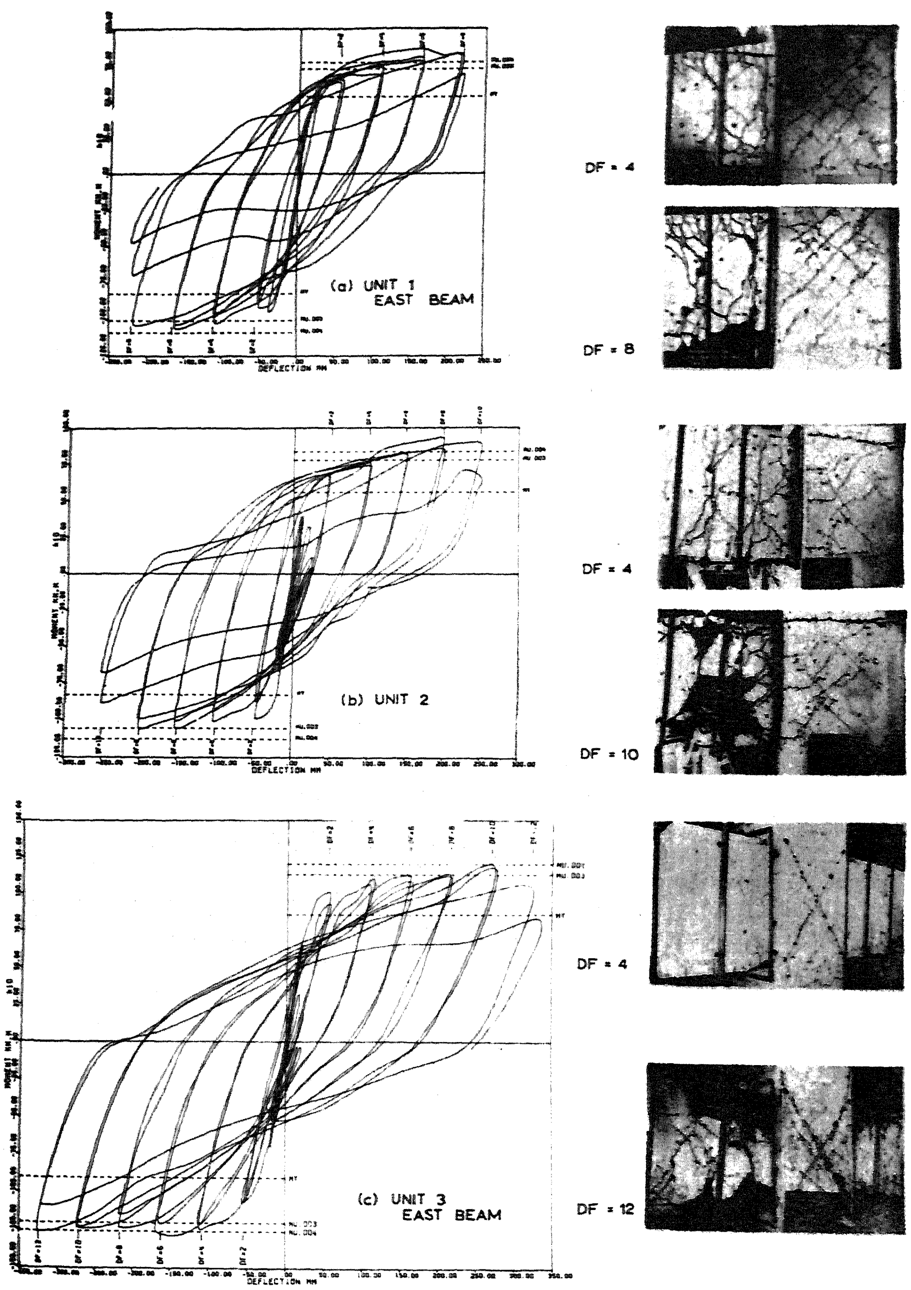

Fig.4 Moment - Deflection Behaviour

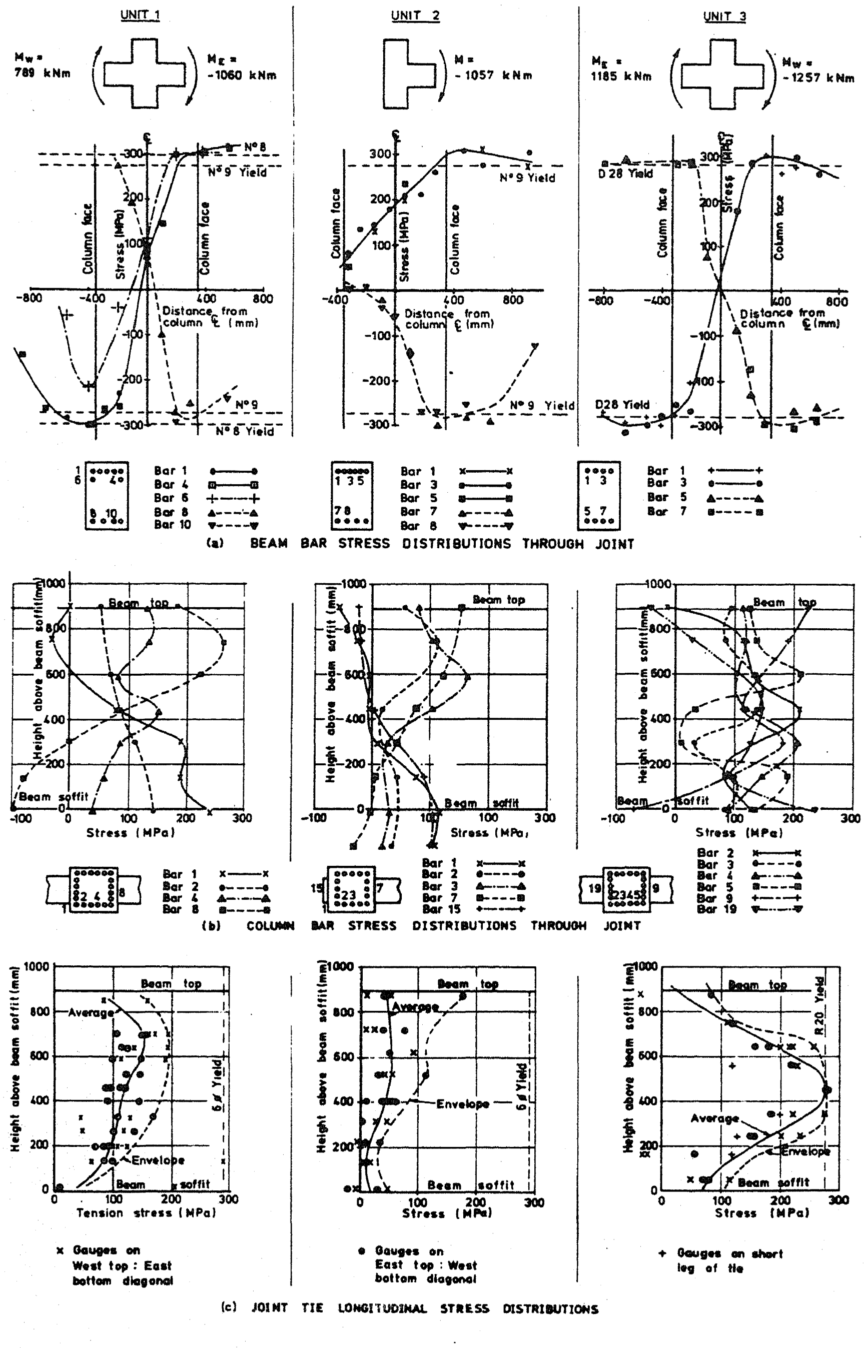

Fig.5 Steel Stresses at Displacement Ductility Factor = 6