CYCLIC LOAD TESTING OF TWO REFINED REINFORCED CONCRETE BEAM-COLUMN JOINTS

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SYNOPSIS

The paper describes the testing of two reinforced concrete beam-column joint units tested under incremented-static cyclic loading. The full size test units were based upon an interior beam-column joint of a four-storey framed building designed to the current NZ loading code⁽¹⁾ and represent refinements on two previously tested conventional joints of similar dimensions. One unit differed from common practice by having a post-tensioned beam stressed to balance the floor dead load of the prototype structure whilst the second unit was detailed with haunched beams.

Hinge formation occurred in the beams and stable hysteretic behaviour was obtained up to displacement ductilities of 10 for the prestressed unit and 6 for the haunched unit. The test results are analysed in terms of the draft NZ design code, DZ 3101⁽²⁾, and the ACI Recommendations⁽³⁾ for beam-column joint design.

1. INTRODUCTION

1.1 Background to Tests

The behaviour of two previously tested full-size beam-column units⁽⁴⁾ showed that the joint region of conventional reinforced concrete frames could be detailed with closed ties to satisfactorily withstand load reversals with little loss of strength or stiffness up to beam and displacement ductility factors of 8. However the amount of horizontal joint reinforcement required presented difficulties with joint concrete placement.

Several innovative schemes to refine the joint design by reducing the number and size of the joint shear ties have been put forward. These include:-

- (a) the use of additional flexural steel through the joint and into the beam to force plastic hinge formation away from the column face;
- (b) the use of bond plates attached to the flexural steel in the column and beams to assist in transmitting the bar forces to the diagonal concrete compression strut which develops across the joint diagonal;
- (c) the use of partial prestressing through the joint to promote diagonal strut action in the concrete and so increase the shear carrying capacity of the joint;
- (d) the use of haunched beams or crossing over of some top and bottom beam steel to spread the zone of plasticity.

Scheme (a) has been investigated by Andrews⁽¹⁰⁾. Scheme (b) was tested by Fenwick and Irvine⁽⁵⁾. The units tested exhibited higher sustained strength and better ductility performance than conventionally detailed beam-column joints. Park and Thompson⁽⁶⁾ have tested beam-column test units with varying amounts of beam prestress. Park⁽⁷⁾ reports that the use of prestress applied at mid-beam depth improves the shear strength of the joint cores. The prestress in effect acts as shear reinforcement in the joint core while the non-prestressed beam reinforcement ensures that the beam hinges are capable of substantial energy dissipation, as indicated by the development of 'fat' hysteresis loops.

Bertero and Popov⁽⁸⁾ have tested beams and beam-column joints with crossover longitudinal beam steel some distance from the column face. They conclude that the beams in energy dissipating building frames should be designed to form plastic hinges away from the column faces to prevent beam bar yield penetration with consequent loss of bond in the joint region. The additional rotational ductility required by forcing beam hinges to form closer together within a bay could be achieved by special detailing of the beam bars to increase the plastic hinge lengths. The same result can be achieved by beam haunching to match the effective depth of the longitudinal steel to the applied beam moment thereby lengthening the plastic rotation zone.

The purpose of the tests reported here was to provide further experimental information on the behaviour of two of the refined beam-column joint designs outlined above. To counter size effects such as the ratio between bar diameter and member dimensions, which may well have contributed to joint failures in smaller test specimens, full size test units were used.

1.2 Theory of Joint Behaviour

A theory of joint behaviour was

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postulated in the previous report⁽⁴⁾. Since then the ACI-ASCE Committee 352 has published its recommendations for the design of beam-column joints⁽³⁾. Two types of joints are defined; Type 1 being where no significant inelastic deformations are expected, and Type 2 where strength under reversals in the inelastic range is required. For Type 2 joints, as relevant to this study, the concrete is assumed to be able to carry some shear, the expression being a function of axial load and concrete strength and with an allowable 1.4 times increase for the effects of confining members perpendicular to the direction of the shear forces considered. Transverse reinforcement for shear is to be evaluated using a conventional beam shear expression with an implicit assumption of diagonal tension cracks forming at 45° to the axis of the member. No requirements are specified for vertical joint shear steel or for effects of biaxial shear.

Idealisations of joint shear resisting mechanisms in terms of diagonal compressive strut action in the concrete and for a truss mechanism, comprising a series of diagonal compressive struts in the concrete and horizontal and vertical ties in the form of joint ties and vertical column flexural steel respectively, are discussed in ref. 9 and incorporated in the draft New Zealand concrete design code DZ 3101⁽²⁾. That code requires the shear to be considered resisted by the concrete to be zero, because of observed severe deterioration of shear capacity of a joint under large amplitude reversed cyclic loading, except for three favourable conditions. They are, first, where the column axial load is significant; second, when beams are prestressed through the joint, and third, where beams are detailed so that plastic hinges cannot form adjacent to the column core. Transverse reinforcement for shear is to be evaluated assuming a corner-to-corner diagonal tension crack plane as observed in tests. Vertical reinforcement is to be evaluated on a similar basis with due allowance, where appropriate, for shear carried by the concrete diagonal strut. On the basis of the only test performed under biaxial shear, design may be on a uniaxial approach but without allowing the beneficial effects of full axial load simultaneously in each direction. The diameter of the beam bars for an internal column is limited to satisfy bond requirements to $1/25$ or $1/35$ of the column depth for Grades 275 and 380 steel respectively.

2. DESIGN

2.1 Prototype Building

As for the previously tested joint units⁽⁴⁾, the test specimens were based on the Standard Minitech Teaching Block Structure designed in the Hamilton District Office of the Ministry of Works and Development. The structure is a 3 by 3 bay, 4 storey frame with exterior spans of 8.84 m, an internal longitudinal span of 11.30 m and an internal transverse span of 8.38 m. The interstorey height is 3.66 m on all levels.

2.2 Test Units

2.2.1 Member Properties

The details of the full size test units

were based on the first floor level internal joint of an external frame. To minimise the joint damage which occurs under lateral earthquake loading, the original design was refined and a partially prestressed beam unit and a haunched beam unit were detailed for testing. Reinforcement and member sizes of the test units are shown in Figs. 1 and 2. The 4.42 m length from the column centreline to the pull down pivot point represents half the 8.84 m external span. The flexural capacity of the beams was increased by approximately 20% over that of the earlier interior joint test unit in order to impose higher intensity shear forces on the joint.

2.2.2 Beam Design

2.2.2.1 Prestressed Unit

The partial prestress in the beams was designed to balance the dead load moment while the live load and seismic contribution to the beam moment was designed to be carried by the mild steel reinforcing bars. The seismic contribution 'E' represented 71% of the $1.05D + 1.27L_R + E$ design ultimate negative beam moment. The prestressing cable was positioned 505 mm from the beam soffit to enable the placement of a prestressing duct in the transverse direction. The beam stirrup design followed the Code requirements⁽²⁾ used for the earlier test units, but the stirrup pairs were placed at 100 mm as opposed to 150 mm spacing to give extra concrete confinement and provide the longitudinal bars with additional restraint against buckling.

2.2.2.2 Haunched Unit

The beam haunch was designed so that theoretical yielding would first occur at the end of the haunch, 915 mm from the column face, and then spread towards the column face under both the positive and negative applied load. This detailing was designed to increase the length of the plastic hinge and to force plastic rotation away from the joint region. The cover to the bottom beam bars was increased from 50 mm at the end of the haunch distant from the column to 89 mm at the column face to facilitate positioning of transverse beam bars below the longitudinal steel. The 5-D32 bottom bars in the beams were placed in one row to enable the placement of the required number of joint ties. Note that since these tests the new draft concrete design code, DZ 3101⁽²⁾, has limited the diameter of Grade 275 beam steel passing through interior joints to $1/25$ th of the column depth, that is 28 mm in this case. In both test units, transverse beam steel was placed in the joint to simulate the practical difficulties of reinforcement cage construction and placement of concrete in the congested joint region.

In addition to the R10 beam stirrup pairs at 100 mm spacing, five extra 4ϕ (12.7 mm diameter) stirrups were placed at the ends of the haunch to carry the bursting forces caused by the bend in the bottom longitudinal beam steel. As the likely effectiveness of the stirrups was uncertain at the time of the design, two methods of fixing were detailed. To obtain a snug fit around the longitudinal steel, the stirrups of the west beam were site lap welded. The east beam was detailed with conventional closed stirrups which necessitated stirrups of varying depth over the length of the haunch. Additional R10 U bars

not shown in Fig 2, with vertical legs 445 mm long, were used to anchor the bottom longitudinal beam bars against bursting. Each bottom bar bend was held by three pairs of these anchors giving a total of 40 vertical legs in each haunch. Because it was possible to achieve a relatively tight contact between the U bars and the beam bars, it was thought that these would provide better resistance against bursting than the stirrups.

2.2.3 Joint Design

2.2.3.1 Prestressed Unit

Horizontal shear in the joint of the prestressed unit was calculated using an overstrength of $1.25 f_y$ ($=345$ MPa) for the top and bottom beam steel and a calculated stress in the prestressing steel from a strain compatibility-force equilibrium evaluation of $0.87 f_{pu}$ ($=1530$ MPa).

Because of the advantages of prestressing on joint shear performance it was assumed that a shear stress of $0.18\sqrt{f'_c}$ could be resisted by the concrete in accordance with ACI 318-71. (The present draft concrete Code DZ 3101⁽²⁾ permits a joint shear of 0.7 times the force after all losses in the prestressing steel within the central-third of the beam depth to be assigned to the concrete, around 50% greater than that used in the design). As the design axial column load of the test unit ($0.05f'_c A_g$) was small, it was considered that no additional concrete shear carrying capacity was allowable.

A scheme of three horizontally lap welded R20 ties per set, similar to the reinforcement in the earlier tests⁽⁴⁾, was used for joint shear reinforcement. On the assumption that only the long tie legs (four of six available in each tie set) were effective in resisting the horizontal joint shear, and using a tie steel yield stress of 276 MPa and a capacity reduction for shear, $\phi = 0.85$, the design required 7 tie sets. The $2/3$ effectiveness factor used in the design of the joint ties of the previously tested units⁽⁴⁾, was not used on the basis of indications in those tests that yielding of isolated tie legs would not lead to joint failure. To provide effective restraint to all of the internal column bars by a 90° bend of a tie, the rectangular internal ties were alternatively displaced as shown in Figs 1 and 2.

2.2.3.2 Haunched Unit

Using an overstrength factor of 1.25 on the beam steel yield strength, the joint shear was evaluated as for the prestressed unit by subtracting the column shear of the actual test unit from the sum of the concrete compression and the top steel compression and tension forces at beam flexural strength. None of the horizontal shear was assigned to the concrete. On the basis of a steel yield stress of 276 MPa and a capacity reduction factor, $\phi = 0.85$, the design required 9 R20 tie sets, (as for the prestressed unit, only four of the six legs of each tie set were considered to be effective in resisting joint horizontal shear). However, only 7 tie sets were used in the construction to make joint shear critical with the objective of assessing if existing procedures could be relaxed.

Vertical joint shear was assumed to be carried by the intermediate column bars which were expected to remain in the elastic range throughout the test.

2.2.4 Column Design

The "weak beam-strong column" approach was used in the design of the prototype building. The prototype columns were designed and reinforced for concurrent beam yielding but only tested under uniaxial flexure. Because of the approximate 20% increase in the negative moment capacity of the beams in the tests reported here as compared with the interior joint of the earlier tests⁽⁴⁾, high strength column steel, $f_y = 380$ MPa, was used in place of mild steel to ensure the column remained elastic.

The constant axial load of $0.05f'_c A_g$ applied to the test unit columns represented the minimum axial load on the prototype column under dead and live load, together with the maximum upward beam shear derived from uneven span lengths. Minimum axial load in the column creates the most severe conditions for beam steel bond within the joint and reduces the effectiveness of the shear resisting concrete compressive strut action in the joint region.

3. TEST DETAILS

3.1 Construction

Construction and testing of the two beam-column joint units was carried out at the Ministry of Works and Development Central Laboratories. Steel was supplied bent to shape by a steel fabricator and assembled by laboratory staff. The placing of the relatively high concentrations of joint reinforcement was made feasible by the detailing of horizontally as opposed to the more common vertically aligned welded laps on the internal ties. Photos showing details of the reinforcement cages are presented in Fig. 3.

Each test unit was cast upright in two pours, using concrete supplied by a ready-mix contractor, with a construction joint at the level of the beam top. No particular problems were encountered in placing the joint concrete of the prestressed unit largely because 14 mm maximum aggregate size concrete was used and because the slump of the concrete supplied was 40 mm greater than specified. The concrete used for the haunched unit had a 20 mm maximum aggregate size and as supplied had a slump of 80 mm, significantly less than the specified 100 mm. This concrete proved very difficult to place in the congested joint and grouting was subsequently necessary to fill an extensive void below the bottom beam steel in this region. The boxing of each unit was struck at 24 hours and the units were covered with moist sacking for 7 days to promote curing.

The beam of the first unit was post-tensioned to a concrete stress level of 2.70 MPa at 38 days. Loss of prestress between jacking and anchoring, indicated by strain gauges on the 7 mm diameter prestressing strands, was 0.08 MPa. The leads to these strain gauges were broken during grouting of the cable, and thus further prestress

PRESTRESSED UNIT

HAUNCHED UNIT

FIG. 3 CONSTRUCTION AND REINFORCING DETAILS

Test Set-up for Prestressed Unit

FIG. 4 OVERALL VIEW OF PRESTRESSED UNIT BEFORE TESTING

losses between anchoring and the time of testing were not available. For the purpose of moment capacity calculations, a further loss of 5% was assumed over this 12 day period.

3.2 Material Properties

100 mm x 200 mm concrete test cylinders were cured in a fog room and tests for the 28 day crushing strength and for the crushing strength, tensile splitting strength and modulus of elasticity at the time of testing each unit were carried out.

Five samples taken from the steel batch for each diameter reinforcing bar used in the construction of the test units were tested to establish tension stress/strain characteristics.

Average measured material properties are summarised in Table 1. The steel yield strengths are in most instances close to the specified values, but the 28 day concrete strengths for both units were approximately 40% in excess of the specified 27.5 MPa.

3.3 Load Application and Reaction

The test arrangement used for this series of beam-column tests is shown in Fig. 4 and was essentially the same as that used for the previous tests⁽⁴⁾. Load was applied to the beam ends by independent hydraulic jacking systems providing downward load (negative moment at the column face) through a strain-gauged tension yoke and upward load through a compression load cell and a rocker-roller system. The arrangement differed from the previous tests in that the push-up load was positioned on the outside of the pull down load to better simulate the influence of gravity load effects on the beam moment gradients under lateral loading of the prototype building frame.

3.4 Instrumentation

Beam end deflections were measured to an accuracy of 0.1 mm by optically sighting through a surveyor's level onto scales attached to each pull down yoke. Beam rotations for the prestressed unit were measured over two successive $d/2$ gauge lengths, where d is the effective beam depth, by 50 x .01 mm dial gauges attached between frames connected to the beam. For the haunched unit, beam rotations were measured over 3 equal gauge lengths covering each haunched section of the beams. Demountable mechanical strain gauges with 250 mm gauge lengths were used to measure surface strains along the joint panel diagonals.

Electrical resistance strain gauges were extensively used in both test units to measure strains of beam and column flexural reinforcement of column ties within the joint region and of beam stirrups adjacent to the column face. The strain gauge locations used for the prestressed unit are shown in Fig 5. Care was taken to ensure that the area of the reinforcement covered by the waterproofing used on the strain gauges was as small as possible to minimise bond losses. The strain gauges were connected to a 300 channel Dynamco datalogger which was sensitive to a 1 microstrain change and had a strain range

of $\pm 4\%$. Data was recorded on punched tape and analysed on a IBM 370/168 computer. Of the 190 gauges placed on the steel of the prestressed unit, only 134 were working satisfactorily at the start of the testing and at a displacement ductility factor of 8 a further 44 had failed. The number of open circuits evident after the placement of the concrete indicated that the initial failure of so many gauges could be largely attributed to damage during vibration of the concrete. Unsatisfactory strain gauge glue may have been a contributing factor to the relatively poor performance after the test had begun. The 320 gauges placed on the haunched unit fared somewhat better. At the start of testing 300 were functioning satisfactorily and at a ductility factor of 8, 255 were still working.

3.5 Test Sequence

The testing of both units was carried out in accordance with the test sequence outlined in Ref 4. Following two load controlled cycles to approximately $3/4$ of the yield load, a series of two displacement controlled cycles at a specified ductility factor (DF) followed by a single DF = 0.75 cycle, were carried out. The specified ductility factor for the displacement controlled cycles started at DF = 2 and was successively incremented by 2 until failure. The behaviour of the test units was thus investigated for a simulated seismic loading of increasing intensity. The intermediate cycles of DF = 0.75 provide information about the working load response following each post elastic excursion.

The yield displacement for each test unit was experimentally determined by extrapolating from the measured load/deflection curve for the beam ends from the last experimental reading, at approximately $3/4$ of the yield load, to the theoretical first yield, the theoretical load at which the beam steel first reaches yield strain.

The zero load condition for the strain and deflection readings was obtained by applying to each beam end an upward load to counter the beam dead load moment (minus the prestress moment for the prestressed unit) at the column face.

4. RESULTS

4.1 Prestressed Joint

4.1.1 General Description of Behaviour

Flexural cracking of the beams occurred during the first loading cycle at applied column face moments of + 390 kNm and - 370 kNm, both values close to the computed cracking moment of ± 375 kNm. During the first two $3/4$ yield cycles, fine diagonal cracks of up to 0.4 mm maximum width appeared at the centre of the joint panels. The significant joint panel cracking was aligned corner to corner.

DF = 2: Flexural cracking which had developed at the top and bottom of the beams intersected and some horizontal cracking along the longitudinal beam steel (bond splitting) was observed. The existing diagonal cracks in the joint panels increased to a maximum width of 0.8 mm.

Degradation of Hinge Zone,
West Beam

Buckling of top longitudinal
bars. East Beam

FIG. 6 PRESTRESSED UNIT: CONDITION AT END OF TEST

FIG.7 PRESTRESSED UNIT: MOMENT DEFLECTION BEHAVIOUR

FIG. 8 PRESTRESSED UNIT: WEST BEAM MOMENT DEFLECTION COMPONENTS

DF = 4: Concrete crushing at the top and bottom of the beams close to the column faces was observed. There was no appreciable increase in the maximum joint panel crack widths.

DF = 6: Bottom concrete cover close to the column face was lost from the beams as the result of extensive bond splitting along the longitudinal beam steel.

DF = 8: Concrete cover at the beam tops was reduced to a rubble over a length of 500 mm from the column faces. The ends of the main joint diagonal cracking deviated from their corner to corner alignment and extended along the line of the outside column bars. The maximum joint panel crack widths were recorded at 1.5 mm. Flexural crack widths indicated that yielding of the beam steel extended approximately 2d from the column faces at peak load and closure of these cracks was observed under reversed loading.

DF 10: Loss of beam steel cover extended back 300 mm from the column faces and bond splitting along the line of the beam steel into the joint region was observed. At the column face these crack widths opened to a maximum of 2.0 mm while the maximum joint diagonal crack widths were recorded at 3.0 mm. Significant bulging was observed at the centre of the joint panels at peak load.

DF = 12: Alternate lateral buckling and straightening of the beam steel occurred, more noticeably during the second DF = 12 cycle. The maximum width of the diagonal cracks in the joint region increased to 6.3 mm and although grinding action was evident along the diagonal cracks no significant spalling of joint concrete occurred. The integrity of the joint was maintained throughout the testing as indicated by Fig 6 which shows the condition of the prestressed unit at the completion of testing. While the beam hinge zones suffered stiffness degradation, no significant sliding shear displacement was observed in these regions.

4.1.2 Moment/Deflection Behaviour

Fig 7 shows the column face moment/beam end deflection hysteresis loops for the two beams of the prestressed joint together with photographs showing the physical condition of the joint at successively increasing ductility factors. Also indicated are three moment limits designated by MY, MU003 and MU004 which represent the theoretical moments at which steel strains first reach yield and the ultimate moments based on extreme fibre compression strains of 0.3% and 0.4% respectively. The difference between the two ultimate moments results mainly from the degree of strain hardening of the tension steel which was based on the measured stress/strain relationships.

Very satisfactory behaviour is indicated by the hysteresis loops with no significant degradation in load capacity or stiffness at ductility factor less than 12. In most instances the peak moments lie between the theoretical MU003 and MU004 moment capacities. The observed degradation in the moment capacity for the DF = 12 cycles corresponded to lateral buckling of the longitudinal beam steel.

The components of the west beam end deflection resulting from beam rotation and joint shear rotation are shown in Fig 8. It can be seen that by far the largest contribution came from rotation of the first d/2 gauge length from the column face and that the joint shear contribution was negligible.

4.1.3 Steel Stresses

(a) Beam Steel

The beam steel stress distribution through the joint of the prestressed unit at the peak loading of the first $\frac{3}{4}$ yield cycle and the first forward cycle for DF = 2, 4 and 6 is shown in Fig. 9. Exceptionally high bond demands were met by the joint concrete, particularly at higher ductility factors where progressive failure of bond at the outside of the column caused the anchorage points of the beam bars to move further into the joint panel. At DF = 6 the tensile yield had advanced approximately 150 mm inside the column face. The development length for twice yield force was then approximately 500 mm and the average bond stress for the D28 bars was 7.7 MPa. Although it cannot be confirmed, because of extensive gauge failure at high ductility factors, the fact that no observed slippage of the main steel occurred up to the end of testing indicated that the joint continued to provide sufficient bond to enable complete reversal from yield in tension to yield in compression for the beam steel. Note that the stresses within the joint region are generally symmetric and show a linear rate of change with no enhancement of bond conditions beneath the compression zone of the column, consistent with previous tests⁽⁴⁾.

For the purpose of the steel stress distribution plots, post yield strains were converted to stresses using a Bauschinger analysis computer program. A typical stress/strain history for one strain gauge is given in Fig 10. The tendency for hysteresis loops to centre on successively higher tensile strains at higher ductility factors, which was evident from a similar plot of a beam steel gauge from the interior joint of the previous test series⁽⁴⁾ did not occur. This was largely due to the effect of beam prestress and the use of equal top and bottom steel areas which promoted flexural crack closure under reverse loading.

(b) Joint Ties

Fig 11 shows the vertical distribution of longitudinal stresses in the joint ties at gauges located along the joint diagonals at the peak loading of the first $\frac{3}{4}$ yield cycle and at DF = 2, 4 and 8. There is considerable scatter in the recorded stresses but the general trend is for an increase in the tie stresses close to the centre of the joint panel as indicated by the stress envelopes. Although yielding in many of the longitudinal tie legs occurred for ductility factors in excess of DF = 4, the joint remained intact throughout the testing, indicating that yielding of joint tie legs need not be seen as limiting the overall integrity of the joint.

FIG. 9 PRESTRESSED UNIT
BEAM STEEL STRESS DISTRIBUTION THROUGH JOINT

FIG. 10 TYPICAL STRESS-STRAIN HISTORY FROM BAUSCHINGER ANALYSIS
PRESTRESSED UNIT, EAST BEAM TOP BAR

FIG. 11 PRESTRESSED UNIT
JOINT TIE LONGITUDINAL STRESS DISTRIBUTION

FIG. 12 PRESTRESSED UNIT
COLUMN BAR STRESS DISTRIBUTION THROUGH JOINT

Confinement stresses indicated by the strain gauges on the transverse legs of the joint ties showed a similar pattern to the longitudinal stresses with yield being reached at DF = 4 for one of the short tie legs adjacent to a beam for the tie set at mid-joint depth.

(c) Column Bars

The longitudinal stress distribution for the column bars through the joint region at the peak loading of the first $3/4$ yield cycle and at DF = 4 and 8 is shown in Fig 12. The tensile peaks, particularly noticeable for the intermediate column bars, is thought to indicate that these bars are providing the vertical tensile component of the truss action transfer of shear across the joint.

(d) Beam Stirrups

The stresses recorded in the two strain gauged stirrup sets either side of the joint increased with ductility factor. At DF = 6, the average stress in the vertical legs was around 80 MPa, although yielding at two of the gauged locations occurred during the two DF = 8 cycles.

4.2 Haunched Beam

4.2.1 General Description of Behaviour

The testing commenced with three $3/4$ yield cycles and at the completion of this loading the flexural cracks initiating in the top and bottom of the haunched sections of the beams had met, and fine cracking, aligned corner to corner, had developed in the joint panel. Flexural cracks developed in the column above and below the beam sections. The maximum crack widths, 0.3 - 0.4 mm at peak loading, were recorded in these areas.

DF = 2: At the completion of the two DF = 2 cycles the maximum flexural cracks in the beam, close to the column face, had a width of 3.0 mm and the joint panel cracks had formed into a continuous corner to corner pattern with a maximum width of 0.2 mm. The corresponding maximum width of the column flexural cracks was 0.6 mm.

DF = 4: Horizontal cracking along the longitudinal beam steel developed and some spalling of small pieces of concrete occurred from the flexural cracks. Up to five main flexural cracks had developed along the beam haunches. The flexural cracks in the bottom of the beams at the column face had maximum openings of 5.0 mm and the other flexural cracks along the haunches had maximum crack widths of 1.6 to 2.2 mm.

DF = 6: Large shear displacements across 7 mm wide cracks running through the beam longitudinal top steel cut-off points occurred in both beams during the first DF = 6 cycle. There was little change in the beam flexural and the joint panel shear crack widths.

DF = 8: Most of the concrete cover on the underside of the beam haunches spalled off during the two DF = 8 cycles. Some spalling occurred on the shear cracks that developed in the beams during the first DF = 6 cycle

and these cracks reached a maximum width of 10 mm. It became obvious during the second DF = 8 cycle that the bottom longitudinal beam bars were slipping through the joint panel allowing the beam cracks at the column face to open to a maximum width in excess of 40 mm. The joint panel remained intact with a maximum opening of the corner to corner cracks of 0.6 mm.

DF = 10: The testing was concluded after two DF = 10 cycles. In the final cycle the bottom beam bars slipped through the joint a total of 91 mm between the loading peaks. The condition of the specimen at the end of the testing is shown in Fig 13 which illustrates the extent of the spalling and the effects of the bar slipping.

4.2.2 Moment/Deflection Behaviour

The moment deflection plots for the haunched beam are shown in Fig 14 together with photographs showing the joint condition at successively increasing ductility factors. The moments are plotted for the column face and the deflection refers to the beam pull down point.

Reasonably satisfactory behaviour is demonstrated by the hysteresis loops with no significant loss of stiffness or load capacity through to the end of the DF = 6 cycles. The degradation in the moment capacity following the first DF = 8 cycle is mainly a result of the slipping of the bottom beam bars through the joint with some influence from the beam shear cracks.

The components of the west beam end deflection arising from the beam rotation over three successive gauge lengths of 450 mm are shown in Fig 15. The influence of the slipping of the beam bars is demonstrated by the large components in the first gauge length during the DF = 8 cycles. The displacement component from the joint shear is not shown but this contribution was small with a maximum value of less than 2.5 mm.

4.2.3 Steel Stresses

(a) Beam Steel

Beam steel stresses through the joint region of the haunched unit are shown in Fig 16 for the peak loading of the third forward cycle at $3/4$ yield and at the first forward cycle for DF = 2, 4 and 6. Compression yielding of the beam longitudinal reinforcement did not occur until the DF = 6 cycles. The point at which tension yielding commenced moved from just inside the column face at DF = 2 to about 100 mm into the beams at DF = 6. At DF = 6 the development length from compression yield to tension yield was approximately 600 mm. Because of gauge failures, the change in the beam steel stress distribution that would have occurred with the commencement of the bottom bars slipping through the joint was not recorded.

(b) Joint Ties

The stresses in the longitudinal legs of the joint ties measured with gauges located on the panel diagonals are shown in Fig. 17. There is a general increase in tie stresses with increasing applied load. Yield was reached in some ties at DF = 4 and in some

Shear failure – East beam

Longitudinal bar slip & spalling-
West beam

FIG. 13 HAUNCHED UNIT CONDITION AT END OF TEST

DF = 2

DF = 4

DF = 6

DF = 8

DF = 10

FIG. 14 HAUCHED UNIT: MOMENT DEFLECTION BEHAVIOUR

FIG. 15 HAUCHED UNIT: WEST BEAM MOMENT DEFLECTION COMPONENTS

FIG.16 HAUNCHED UNIT: BEAM STEEL STRESS DISTRIBUTION THROUGH JOINT

FIG.17 HAUNCHED UNIT: JOINT TIE LONGITUDINAL STRESS DISTRIBUTION

FIG.18 HAUNCHED UNIT: COLUMN BAR STRESS DISTRIBUTION THROUGH JOINT

legs of the majority of ties at $DF = 8$. However, as for the prestressed unit, the performance of the joint panel was satisfactory throughout the testing.

Joint confinement stresses measured on the transverse legs of the joint ties were approximately one half of the longitudinal stresses and at $DF = 8$ none of these legs had yielded.

(c) Column Bars

Stresses measured on the Grade 380 MPa vertical column bars through the joint panel showed wide fluctuations over the joint depth and reached peak tension values of 350 MPa during the $DF = 8$ cycles, as shown in Fig. 18.

(d) Beam Stirrups

Strains were recorded in the beam stirrup sets at the beam bottom bar bends and at locations 300 mm either side of this location. Fourteen of a total of 28 vertical legs were gauged. At $DF = 4$ the average stress in the vertical legs was 105 MPa and stresses increased with yield being reached in eight legs at $DF = 8$. There was no significant difference between the stresses recorded in the welded-in-place stirrups on the west beam and the conventional closed stirrups used on the east beam. This similarity in performance was also demonstrated by close agreement of the crack patterns in both beams.

(e) Beam U Bars

Strains were measured on 6 of the 40 U bars intended to prevent bursting at the bends in the bottom longitudinal bars of the beam. Yield stress was reached in one of these bars at $DF = 4$ and in two bars at $DF = 8$. The average stress level in the gauged bars at $DF = 4$ was 120 MPa.

5. COMPARISON OF TESTED SPECIMENS WITH CURRENT CODE REQUIREMENTS

5.1 Joint Design

The joint steel requirements given by DZ 3101⁽²⁾ and the ACI Recommendations⁽³⁾ are compared in Table 2 with the steel provided in both the prestressed and haunched beam units. A comparison is also made between the horizontal and vertical shear assumed by the codes to be carried on the concrete and the equivalent shear from the idealised joint resisting strut and truss mechanism discussed in Reference 9.

The code values given in Table 2 have been calculated using the specified methods but the joint forces were assessed using the beam moment strengths, obtained from the measured stress/strain relationship for the reinforcement and an outer fibre concrete strain of 0.004, in place of the 1.25 over-strength factor used in the codes. The joint steel requirements and shears carried by the concrete were obtained using the measured steel yield stresses and concrete crushing strengths.

DZ 3101⁽²⁾ makes allowance for the beneficial effect of beam prestress, which promotes strut action transfer of joint

shear in the concrete and acts to restrain diagonal cracking, by assigning the joint concrete to resist a horizontal shear force equal to 0.7 times the beam prestress force after losses. At low levels of column compression, DZ 3101 makes no allowance for the column axial load in evaluating the component of the joint horizontal shear carried by the concrete. The ACI Recommendations⁽³⁾ specify the joint concrete to carry a shear stress of $0.29\sqrt{f'_c}$ MPa increasing with axial load, but make no special allowance for beam prestress. A further difference between DZ 3101 and ACI Recommendations in evaluating the joint horizontal steel requirements is that DZ 3101 assumes corner to corner cracking across the joint panel as observed in tests, whereas the ACI Recommendations use the familiar beam shear reinforcement equation which implicitly assumes 45° cracking.

Although the testing indicated that the presence of the haunch was beneficial to the performance of the joint, neither code recognises this effect.

The purpose of vertical joint steel distributed across the width of the joint is to provide reinforcement to carry the vertical component of the truss action transfer of joint shear. While the ACI Recommendations do not cover this aspect of joint design, specific reference is made in DZ 3101. Where the compressive force in the column is small, DZ 3101 assumes approximately one-half of the vertical joint shear force to be resisted by the concrete while the remainder is assigned to the intermediate column bars.

The ACI Recommendations prescribe transverse confinement steel in the joint where the column load is greater than 0.4 times the 'balanced failure axial load of the column'. DZ 3101 uses a modified form of the ACI formulae to prescribe the required confinement steel linearly increasing with column axial loads. As the axial load applied to the column of the test unit was relatively small, to represent the most severe joint conditions under seismic lateral loading of the building frame, no direct comparison of the respective transverse confinement steel requirements in the joint can be made.

5.2 Beam Shear

In the haunched unit 36% of the beam top flexural steel stopped off at a point approximately midway between the end of the haunch and the load application point near the end of the beam, and it was at this location that the wide shear cracks (see Fig 13) formed during the first $DF = 6$ cycle. To satisfy the development and shear requirements of DZ 3101 (similar requirements to ACI 318-71 and 77) it is necessary for the applied shear to be less than two-thirds of the shear capacity at the steel cutoff point. The shear capacity at the cutoff was calculated using the measured material properties and a capacity reduction factor of 0.85. The test peak shear loads producing negative beam moments were applied to the beams in the first $DF = 6$ cycle and the ratios of the applied shears to the shear capacity at the cutoffs for the respective beams were :-

West beam : $\frac{\text{applied shear}}{\text{capacity}} = 0.74$

East beam : $\frac{\text{applied shear}}{\text{Capacity}} = 0.64$

Both beams had equal shear capacity but a higher shear load was applied to the west beam which was loaded in a downward direction and formed the shear crack before the east beam was subjected to downward load. Although the east beam satisfied the code requirements, the wide shear crack also formed.

6. CONCLUSIONS AND DESIGN RECOMMENDATIONS

6.1 The performance of the two beam-column assemblies satisfied the anticipated ductility demands of severe seismic loading. The units formed stable hysteresis loops at displacement ductility factors of 6 and 10 for the haunched and prestressed units respectively, while sustaining loads in excess of the theoretical maximum. The haunched unit suffered beam bottom bar slippage through the joint, due mainly to construction problems of placing the low slump, large aggregate concrete in the area of congested reinforcement, although the size of these bars was larger than is now allowed by the new draft New Zealand concrete design code, DZ 3101. This unit also formed large shear cracks at the beam top steel cut-off points. These problems however, did not reduce the energy dissipation capacity or strength repeatability of the unit at ductility factors of 6 or less.

6.2 The performance of the joint ties in the partially prestressed unit was adequate, the joint remaining intact although some horizontal tie legs yielded. Although 12% more ties were provided than are necessary using DZ 3101, the draft clauses for prestressed beam-column joints seem satisfactory.

The joint shear requirements in DZ 3101 when applied to haunched beams appear conservative and they could be amended to allow some joint shear to be resisted by the concrete because of the beneficial effects of the spread of the hinge region. From the test unit's performance this seems allowable even when some yielding occurs at the column face. No recommendations are made here for joints under biaxial shear conditions as only uniaxial beam yielding was tested.

6.3 The detailing of equal top and bottom beam steel in the partially prestressed unit eliminated the 'growing beam' effect and subsequent beam deterioration which manifested itself in the earlier tests where unequal amounts of steel were detailed. The haunched beam unit had unequal top and bottom steel and "grew" by approximately 40 mm. However, there was less deterioration than in the earlier tests, attributable to the spread of plastic hinges and the shear resisting effects of the inclined bars.

6.4 The closer spacing of the beam stirrups in hinge regions detailed in these tests effectively reduced the amount of concrete lost between the ties although the beam bars in the partially prestressed unit still buckled horizontally at high displacement ductility factors.

6.5 The extra stirrups and U bars provided at the end of beam haunch to restrain the bent bottom bars from bursting, were effective, and there was no noticeable difference in behaviour between the closed or the site-welded stirrups.

6.6 The code clauses relating to the cutoff of bars in beams subject to seismic load reversals should be examined and amended.

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This paper was presented at the South Pacific Regional Conference on Earthquake Engineering held in Wellington on 8, 9 and 10 May, 1979.

TABLE 1 : MATERIAL PROPERTIES

(a) Steel Properties

Bar Size (mm)	Yield Stress (MPa)	Ultimate Stress (MPa)	Young's Modulus (GPa)	Yield Strain Ratio at onset of Strain Hardening
HD 32	374	-	183	4.5
D 32	265	456	193	10.2
D 28	278	461	194	12.1
No 8 (25.4 mm)	291	465	190	9.1
D 16	302	465	199	14.7
R 20	276	452	206	14.6
No 4 (12.7 mm)	362	541	208	11.2
R 10	304	423	180	8.8
7 mm tendon	Linear elastic to 1300 MPa		202	-

(b) Concrete Properties

	Prestressed Unit	Haunched Unit
Max. aggregate size	14 mm	19 mm
f_c^t 28 days	39.0 MPa	39.2 MPa
f_c^t at testing	44.3 MPa	45.8 MPa
f_t at testing	3.7 MPa	-
E at testing	25.2 GPa	32.6
Density	2380 kg/m ³	-
Slump	140 mm	80 mm

TABLE 2 : COMPARISON OF ACTUAL JOINT REINFORCEMENT WITH CURRENT RECOMMENDATIONS

	DZ 3101 ²	ACI ³ Recommendations	Actual
Prestressed Unit			
V_{ch} kN	707	1790	1300*
A_{jh} mm ² (Grade 275)	7840	8390	8800
V_{cv} kN	1660	-	2160*
A_{jv} mm ² (Grade 380)	5510	-	6430
Haunched Unit			
V_{ch} kN	0	1790	1180*
A_{jh} mm ² (Grade 275)	13080	10310	8800
V_{cv} kN	2087	-	2040*
A_{jv} mm ² (Grade 380)	6930	-	6430

* Estimated in accordance with Reference 9. The corresponding compressive stress in the principal diagonal compression strut was approximately 14 MPa for both units.

NOTATIONS IN TABLE 2:

- A_{jh} = total area of effective horizontal joint shear reinforcement
 A_{jv} = total area of effective vertical joint shear reinforcement
 V_{ch} = allowable horizontal joint shear force resisted by concrete shear resisting mechanism only
 V_{cv} = allowable vertical joint shear force resisted by concrete shear resisting mechanism only