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Seismic Performance of Reinforced Concrete External Beam-Column Joints in Ductile Frames

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SEISMIC PERFORMANCE OF REINFORCED CONCRETE EXTERNAL BEAM-COLUMN JOINTS IN DUCTILE FRAMES

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INTRODUCTION

This report describes the experimental testing of eight reinforced concrete external beam-column joint sub-assemblies designed to the current NZ Concrete Standard (NZS3101:1995). The first 4 test units incorporated Grade 430 reinforcing in the beam while the later 4 units used the new Grade 500E reinforcing steel. The aim of this project was to investigate different ways of detailing the beam bar anchorage within the joint region, especially the requirement that the beam bars should be bent into the joint with 90 degree bends, which must be anchored at a distance no greater than ¼ of the column depth from the outside face of the column. Several test units employed the conventional "standard hook" (90 degree bend and a 12 bar diameter (d_b) tail) while 2 units used the continuous U-bar detail as a comparison. The testing also set out to examine the influence of beam elongation, due to plastic hinging on the performance of beam-column joints at the first floor level of multi-storey frames. One test unit had its beam reinforcing spread uniformly down the beam section (with horizontal U-bars) so as to compare its seismic behaviour with that of the conventional detailing with concentrated areas of beam steel at the top and bottom of the section.

TEST UNITS

Units 1-4 with 430 MPa beam bars:

All 8 beam-column subassemblies had beams which were 515 mm deep and 250 mm wide intersecting with a column 500 mm deep by 300 mm wide. The size of test unit was chosen to fit into the testing facilities at the University of Auckland's, Department of Civil & Environmental Engineering Test Hall and represents about a 2/3 scale model.

Units 1, 2 and 3 were all designed with 4-D20H bars (actual f_y = 452 MPa, 430 Grade steel) top and bottom in the section. The steel ratio was 1.06%, about 70% of the maximum steel ratio of 1.55% specified for beams in New Zealand Concrete Structures Standard, NZS 3101 [1] with 430 MPa reinforcement and 30 MPa concrete.

The current Concrete Standard NZS3101 [1] is very specific about the detailing of beam bars within external beam-column joints of ductile reinforced concrete frames. Where a potential plastic hinge zone is located in the beam at the face of the column, longitudinal beam bars must be anchored by a "standard hook" a distance from the inside face of the column of no less than ¾ of the column depth. The first 3 beam-column joint sub-assemblies described in this report had this detail, as shown in Figure 1, so that the significance of anchoring the bars short of the external longitudinal reinforcement in the column could be established. In Units 1 and 2 the beam bars were anchored with "standard 90-degree hooks" with a $12d_b$ tail into the joint region, the outer edge of the bottom beam bar's tail being exactly 125 mm (1/4 of the column depth) from the outer column face. The top beam bar tails were positioned outwardly adjacent to the bottom beam bars, (see Figure 1). The third Unit had continuous 4-D20H U-bars (f_y = 482 MPa) as beam reinforcing, with the clear space from the outer edge of the U to the outer column face being the maximum of 125 mm. In the fourth sub-assembly the longitudinal reinforcement in the beam consisted of 7-D16H U shaped bars (f_y = 465 MPa), which were distributed uniformly over the depth of the beam at vertical centres of 72.5 mm. Each U-bar was in a *horizontal plane* and they were anchored as close as possible to the outside edge of the column face. Two vertical D16H bars, 520 mm long, were positioned in each corner of the U bars to improve their anchorage in the joint zone. The outer edge of these extra vertical bars was 65 mm in from the outer column face. In this unit the beam ties were single R6 stirrups at 50 mm c/c.

The beam ties within the Plastic Hinge Zone (PHZ) were sets of R6 and R8 single ties at 115 mm c/c in Units 1 and 2 and double R6 ties at 90 mm c/c in Unit 3. The change was due to the unavailability of R8 reinforcing bar after the second test. In Unit 4, with the distributed beam bars, the PHZ ties were single R6 ties at 50 mm c/c, the reduced spacing being due to the use of single ties. All spacings were less than the buckling criteria restriction of 6 beam bar diameters (120 mm here). All ties were anchored with 135-degree bends with $8d_b$ tails, as specified in the Standard [1].

The horizontal transverse joint shear ties comprised 4 sets of 4-legged R10 ties in Units 1 and 2. The 1995 NZ Concrete Standard [1], when compared with the previous NZS3101 1982 [2] edition, allowed a considerable reduction in the amount of horizontal and vertical reinforcement required in external beam-column joints. However, there was only limited testing evidence for this reduction. The beam-column subassemblies described here complied with the current 1995 Standard in all aspects except one. The extra confining ties around the 4 intermediate column bars were omitted in the first 2 units but were included in Units 3 & 4. In Units 3 and 4 there were 4 sets of ties comprising a R10 tie around the periphery and two R6 ties around the 4 intermediate column bars. See Figure 1 for the reinforcement details for Units 1 to 4.

The main column bars in the first two units were 12-D16 (f_y = 326 MPa). After the testing of the first 2 units and the resulting joint degradation and plastic hinging which occurred in the column below the joint, the internal face column bars were subsequently increased to 4-D20 (f_y = 334 MPa) for Units 3 and 4. See Unit 1 test results section The column confining and shear ties above and below the joint were detailed as sets of R6 ties (3 ties/set) at 96 mm c/c in Units 1 and 2 and at 90 mm c/c in Units 3 and 4, see Figure 1.

Units 5-8 with Grade 500E beam bars:

Unit 5 was very similar to Unit 1 except that Grade 500E beam reinforcement was used (4-D20H top and bottom, f_y = 572 MPa). The side cover to outer hooked beam bars (55 mm) was less than the 60 mm required in the Standard (for α_1 to = 0.7) and thus the hook development length L_{dh} was 303 mm for Grade 500E steel and 40 MPa concrete. The required $8d_b$ (160 mm) plus L_{dh} plus the 125 mm clear to the outer column face exceeded the actual column depth of 500 mm. Thus the provision of Clause 7.5.2.8 was provided, viz. two extra D20H bars horizontally in each 90-degree bend so as to reduce L_{dh} by 20%. In Units 1 to 3 L_{dh} equalled 211 mm with α_1 = 0.7 (side cover approx 60 mm), f_y = 430 MPa and the concrete strength being 30 MPa and no extra bars in the bends were provided. ($8d_b + L_{dh} + 125$ = 496 mm, just satisfactory). Beam ties in the PHZ required tie-sets @ 90 mm c/c, each tie-set comprising 1-R6 and 1-R8 ties. Outside the PHZ the identical tie-sets increased their spacing to 235 mm c/c.

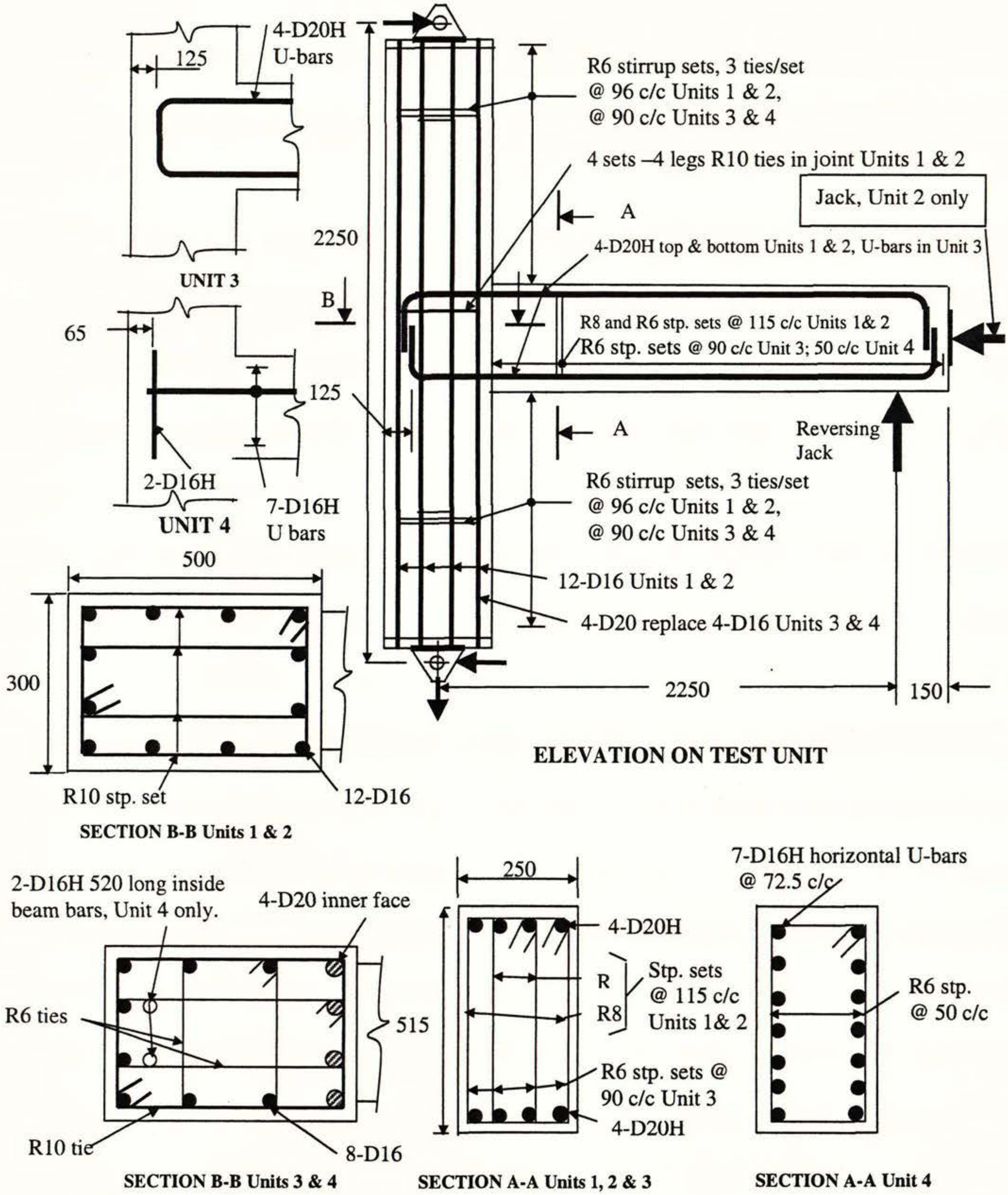

Figure 1. Dimensional and reinforcing details of test beam-column units 1-4.

The column main bars were increased to 10-D16 bars plus the larger 4-D20 bars on the inner column face to allow for the decrease in column capacity experienced in Units 1 and 2 when the inner bars are in tension. This was due to the decrease in the internal lever arm caused by the short beam bar anchorage length across the column. Column ties within the potential PHZs were triple R8 ties + 1-R8 supplementary cross-tie @ 80 mm c/c. Outside the PHZs this spacing was increased to 100 mm c/c.

In the joint region there were 5 sets of R8 ties (3 ties + a supplementary X-tie per set) spaced at about 95 mm c/c (f_{yh} = 369 MPa). 6 legs were assumed when calculating the horizontal joint shear area, A_{jh} requirements, the extra short legs around the inner D16 bars were included. Just over 4 tie-sets were required using the Standard's design formula (Eq. 11-5), thus this joint was overdesigned by about 20% for horizontal joint shear. Tie anchorage was again 135-degree bends with $8d_b$ tails. The reinforcement details for units 5-8 are shown in Figure 2. As the outer tie-sets were not to be positioned right next to the top and bottom beam bars, two extra joint tie-sets (as recommended in the Standard) were not included.

Unit 6:
Unit 6 differed from all the other units in that it had unequal amounts of top and bottom beam steel. This unit had 4-D20H bars in the top and 2-D20H bars in the bottom (f_y = 542 MPa for all bars). Like Unit 5 the beam bars were anchored with "standard hooks" with the tails placed at 125 mm ($h_c/4$) in from the outer column face, as previously. The 2-D20H transverse bars in the 90-degree bends were again included. The beam ties within the PHZ were reduced to single R8 ties @ 80 mm c/c (f_{yt} = 380 MPa) with the spacing increased to 235 mm c/c outside the PHZ.

The main column reinforcement was the same as Unit 5 (10-D16H with 4-D20H on the inner column face). The column ties were also identical to those in Unit 5, both inside and outside the PHZ.

560 mm^2 of horizontal joint ties (A_{jh}) were required in Unit 6 assuming a concrete strength of 40 MPa and using the calculated over strength beam moment (1.15 over strength factor for strain-hardening and using the actual f_y value of the D20H bars). However 5-R8 tie-sets were actually used with a total A_{jh} of 1,510 mm^2 if all the short tie legs are included. In this unit the outer tie-sets were positioned very close to the inner edges of the top and bottom beam bars and therefore 2 joint tie-sets should be neglected in the A_{jh} summation. Thus the amount of over design for horizontal joint shear in Unit 6 was about 60%. If the 6 short tie legs are neglected the over design reduces to nearly 8%. Short tie legs are neglected when their length in the plane of shear being considered is less than a third of the column depth [1]. In these columns the short tie legs are about 0.5 of the column depth and there is little logical reason why they should be included in the A_{jh} area.

Unit 7:
This unit had 4D20H bars top and bottom (f_y = 542 MPa) in the beam with U-bar anchorages (bottom of the U at ¾ of column depth from inner column face). 2 extra D20H transverse bars were placed in each 90-degree bend. Beam tie-sets comprising 1-R6 and 1-R8 ties at 90 mm c/c were placed in the PHZ. The column was identical to previous units, viz. main bars 10-D16 and 4-D20 bars on the inner face.

In the joint region there were 4 sets of 3-R8 ties, this being 1 set less than theoretically required by the Standard. A single R8 supplementary cross-tie was positioned across the joint on the column centreline. Using the actual reinforcing and concrete strengths (with a strain-hardening factor of 1.15) the 4 joint tie-sets represent about 65% of the horizontal joint tie area required by the Concrete Standard if the 2 short tie-legs are ignored. If the 2 short tie legs are included then nearly 98% of the A_{jh} required is provided. The short legs have a length of about 43% of the column depth and would normally be included in the A_{jh} estimation. The assumed axial tension in the bottom column in the above calculation was taken as 150 kN.

Unit 8:
This unit was designed with near the Standard's maximum reinforcing ratio in the beam. For 40 MPa concrete and reinforcing with f_y =500 MPa the maximum steel ratio is 1.67%. Unit 8 had 5-D20H bars top and bottom (in 2 rows, outer row 3 bars and 2 bars in inner row) giving a steel ratio of 1.69%. Beam bar anchorage was provided by the U-bar detail with 2-D20H transverse bars in each 90-degree bend. As in the previous tests the outer edge of the U-bar anchorage was at exactly a quarter of the column depth, measured from the outer face. The PHZ beam ties were sets of single R6 and R8 ties at 90 mm c/c. Outside the PHZ the spacing of tie-sets was increased to 235 mm.

The aim in this unit was to get the horizontal joint shear stress as close as possible to the Standard's limit of $0.2f'_c$. 6 sets of R10 ties (3 rectangular ties + 1 supplementary cross-tie) were placed between the inner rows of beam bars within the joint. The represented an A_{jh} area of 1,885 mm^2 by including the short tie legs but ignoring the outer tie-sets as they were touching the inner beam bars. The Standard [1] required an area of 2,078 mm^2 using the actual yield stress of the beam bars (with overstrength factor of 1.15) and for the ties and 40 MPa concrete strength. Thus about 91% of the Standard's requirement was provided.

The main column bars comprised 10-D16H and 4-D20H, the D20H bars again being placed across the inner column side. In all units the columns were designed to be stronger than the beams ("weak beam-strong column criteria"). In all the tests described here the column above the joint was only supported horizontally at its top and thus the only axial force on the top column was its self-weight, all the vertical loading jack reactions being taken out at the bottom of the lower column. For upward beam loading the column axial force below the joint was equal to the beam jack load in tension reduced by the unit's self weight.

In the column potential PHZs the ties were R10 tie-sets at 55 mm c/c. Each tie-set comprised 3 rectangular ties and a supplementary cross-tie across the section.

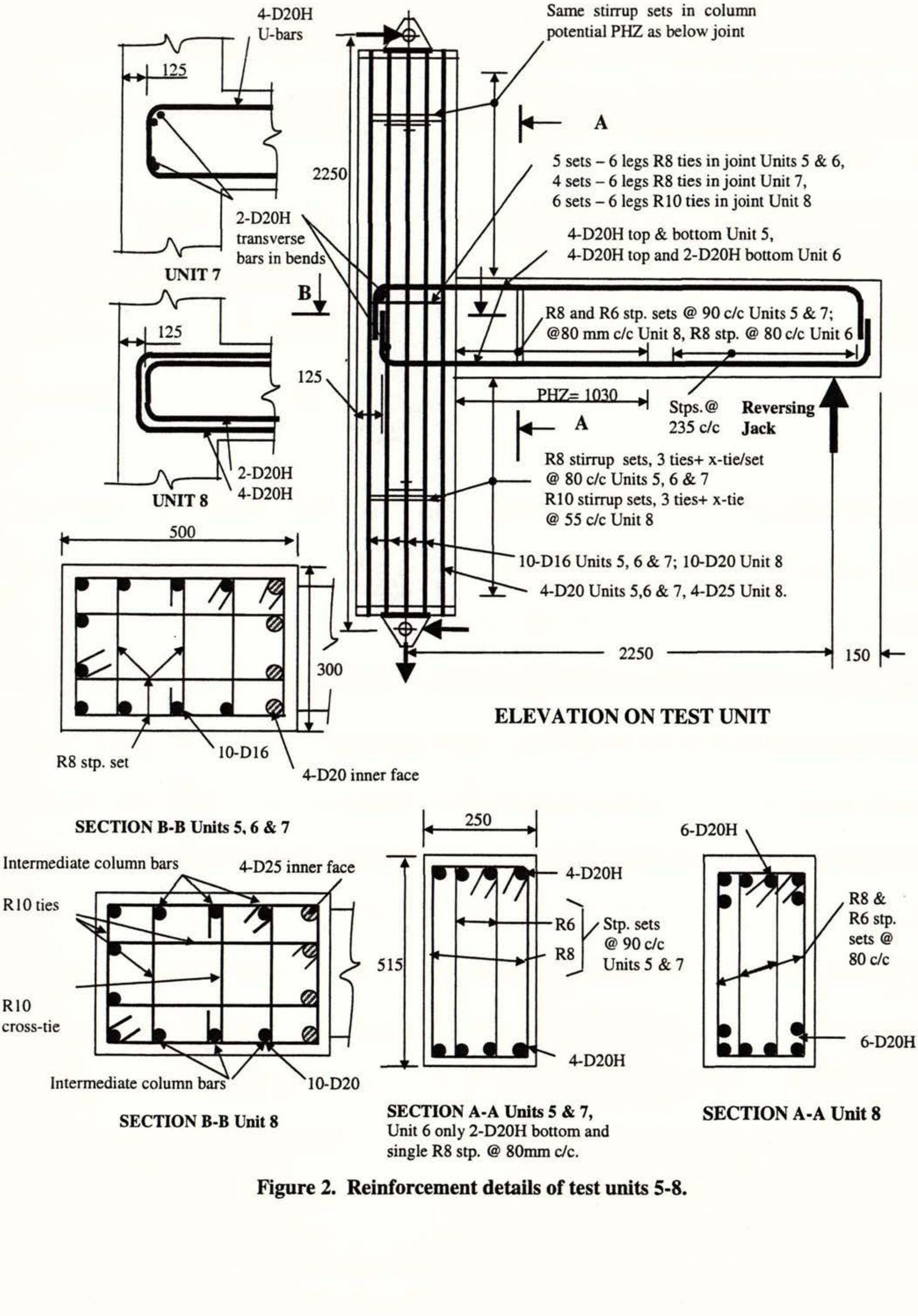

Figure 2. Reinforcement details of test units 5-8.

Fabrication of test units:
The reinforcing cages were fabricated in the University of Auckland Test Hall, the ties and main beam bars being pre-bent by Pacific Steel Ltd. The 6 mm diameter measurement studs, for the portal transducers, were arc welded to the main beam and column bars in positions shown in Figures 3 and 4. No preheating of the 500 Grade reinforcing was done for Units 5-8 before welding on the studs. Pacific Steel have since informed the author that preheating of the main bars to at least 100 degrees C should have been done before arc welding commenced.

The units were cast in the non-prototypical in-plane position on the test floor within timber boxing using ready mix concrete. The specified 28 day compressive strength was 30 MPa for units 1 to 4 and 40 MPa for units 5-8 and the concrete was vibrated with a pencil vibrator.

The units were covered with hessian, which was kept continually moist for about 2 weeks before the shuttering was removed from the sides. 300 mm by 100 mm diameter concrete cylinders were cured in the same way. Samples of the main beam and tie reinforcing steel were tensile tested in an Avery universal testing machine to check the yield strengths and to calculate nominal strengths before unit testing began. The concrete cylinders were usually tested immediately after unit testing had been completed. Material values for all 8 units are presented in Table 1.

The first two units were tested by Badira [3], Units 3 and 4 by Khono [4], supervised by Richard Fenwick and Jason Ingham, and Units 5 to 8 by Barton [5], supervised by Les Megget. The reports written by Badira and Khono were fairly brief, incomplete and much of the test data was unavailable to the author, none of it in electronic form.

Test Arrangement:
All units were tested in a vertical plane with the column vertical as shown in Figure 5. The reversing hydraulic loading jack was fixed to the strong floor and the load was applied to the beam at a distance of 2 m from the inner column face. Applied load was measured using a load cell positioned between the jack's ram and the pin joint attached to the beam. Portal displacement gauges were used to measure any movement of the loading frame relative to the strong wall, which resisted the horizontal reaction at the top of the unit. There was no applied column axial force applied. Vertical measurements of the beam tip (at the point where the load was applied) were measured with an LVDT, a turnpot potentiometer, a portal gauge at small displacements and a steel rule for checking purposes. Lateral bracing was supplied at the column top and at the beam end by pin-ended struts.

Portal displacement gauges were positioned between each steel stud to measure deformations used to calculate flexural deformations in the beam and column, shear deformations in the beam, column and joint zones and beam elongation. Most of these 40-odd gauges had a displacement capacity of +/- 25 mm and were measuring change of length over a gauge length of about 450 mm. The portal gauges are much more reliable than strain gauges glued to reinforcing bars, as the glued on gauges often fail after bar yielding and when cracking predominates at high displacement ductilities. All load cells, LVDT and portal gauge measurements were recorded at preset time intervals during testing by a datalogger/computer setup. Load-deflection curves were continually shown on a monitor throughout testing.

In the first 4 units 6 mm diameter studs were welded to the 4 column beam bars on one side column face at a gauge length of 100 mm. In unit 1 there were 8 strain lengths, 4 above and below the beam centreline, while in unit 2 there were 12 lengths (6 above and below the beam

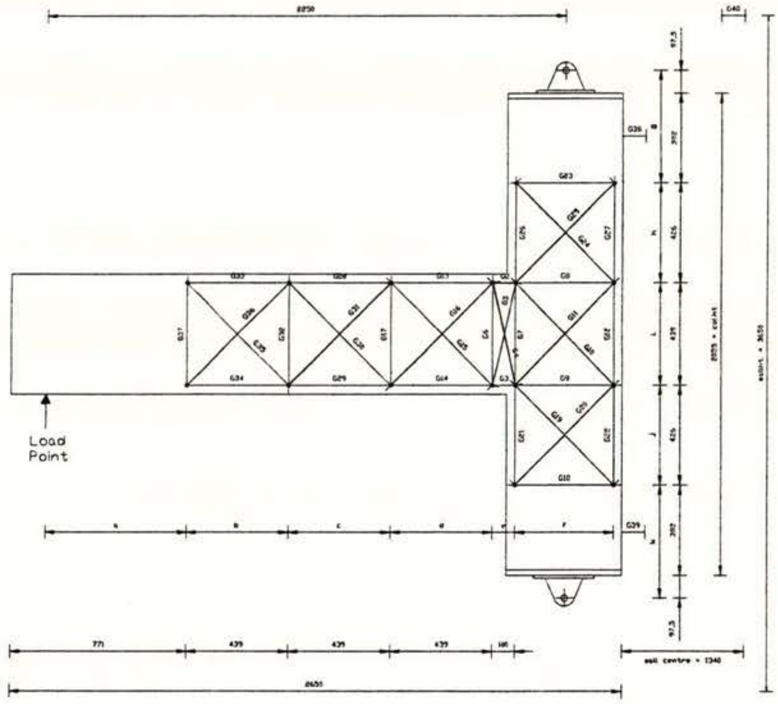

Figure 3. Portal gauge setup on Units 5, 6 and 7.

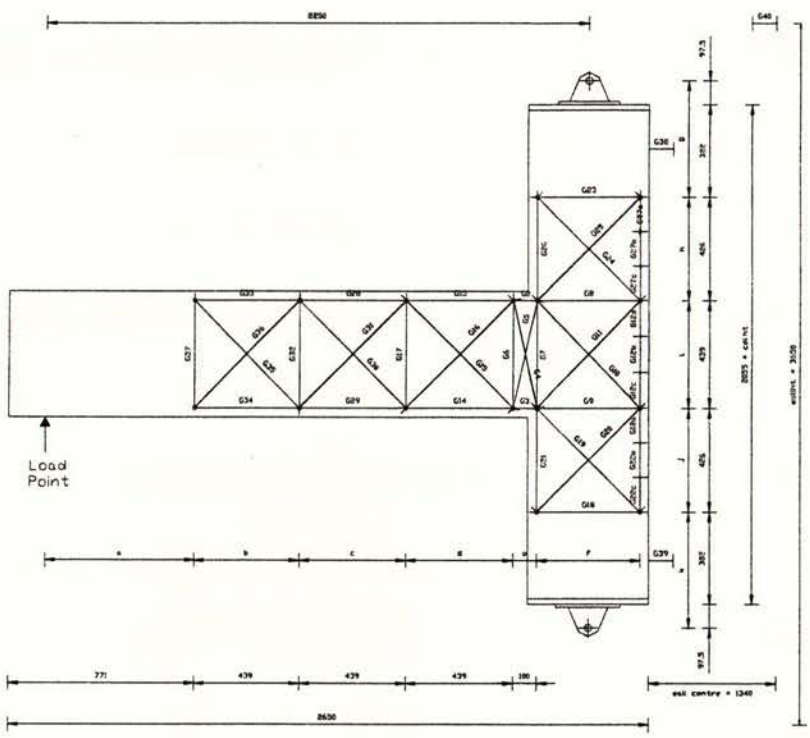

Figure 4. Portal Gauge setup on Unit 8.

centreline). Demec points were glued to these studs and Demec gauge readings were manually taken during testing, usually at the cycle peaks.

TABLE 1: Concrete and reinforcement material properties

UNIT	Concrete Compression Strength f'_c (MPa)	Beam Reinforcing Yield stress, f_y (MPa)	Column Reinforcing Yield stress, f_y (MPa)	Beam ties in PHZ Yield stress, f_{yt} (MPa)	Joint ties Yield stress, f_{yh} (MPa)
1	46.7	452.0 (D20H)	326.0 (D16)	357.0 (R6) 329.0 (R8)	318.0 (R10)
2	48.0	452.0 (D20H)	326.0 (D16)	357.0 (R6) 329.0 (R8)	318.0 (R10)
3	33.9	482.0 (D20H)	333.0 (D16) 334.0 (D20)	Unknown* (R6)	Unknown* (R6 & R10)
4	33.9	465.0 (D16H)	333.0 (D16) 334.0 (D20)	Unknown* (R6)	Unknown* (R6 & R10)
5	36.0	572 (D20H)	302.9 (D20) 323.4 (D16)	342.1 (R6) 385.8 (R8)	385.8 (R8)
6	32.0	542 (D20H)	302.6 (D20) 323.4 (D16)	380.6 (R8)	380.6 (R8)
7	39.3	543.2 (D20H)	302.9 (D20) 323.9 (D16)	400.1 (R6) 341.3 (R8)	341.3 (R8)
8	41.8	541.2 (D20H)	293.3 (D25) 288.1 (D20)	328.4 (R6) 379.9 (R8)	354.6 (R10)

* The yield strength of the ties is not included in Kohno's report but is probably the same as those shown for Units 1 and 2.

In units 1 and 2 electronic strain gauges were attached to one top and bottom beam bar over the anchorage length in the joint region. There were 10 strain gauges at 5 positions on each beam bar spaced at between 100 and 160 mm apart. A strain gauge was placed on each side of the bar (in a vertical plane) so as to obtain an average strain at each position.

The joint ties were also strain gauged at the column centreline. These gauges were positioned on one long leg only of each tie, a total of 8 ties were gauged between the upper and lower beam bars in the joint.

In unit 8 nine extra studs were welded to one outer corner column bar to ascertain the strain profile in that bar during testing. These studs were 200 mm apart with the central one being on the beam's centreline. Portal gauges were placed between each pair of studs and the deformations recorded by the datalogger.

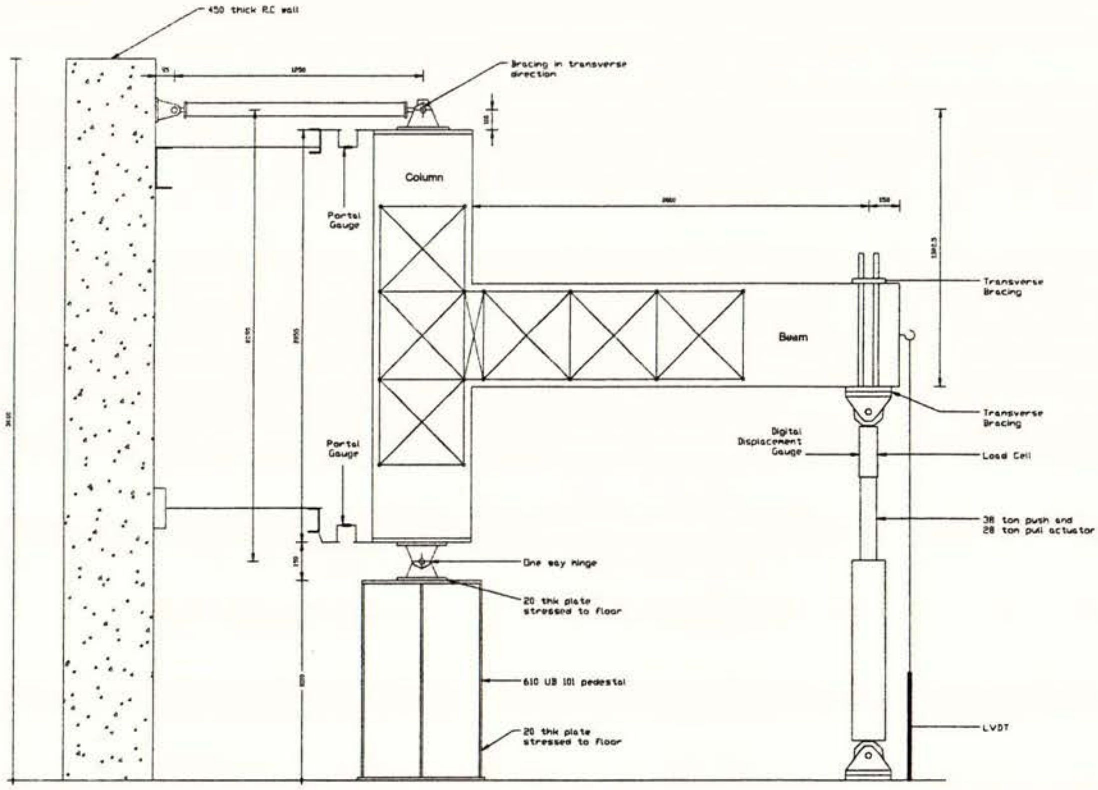

Figure 5. Test unit set up for units 5-8. Units 1-4 identical except for horizontal jack at beam free end in unit 2.

Loading Sequence:

Before unit testing samples of the beam bar reinforcement about 800 mm long were tested in tension in the Avery 500 kN testing machine. From these tests the reinforcing yield stress was calculated and the beams nominal bending strength estimated.

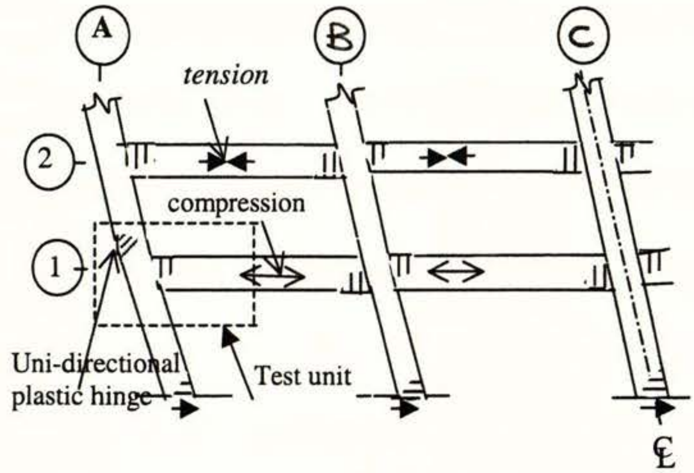

Figure 6. Deformation of column due to elongation.

All the test units were loaded in the same sequence. The loading sequence was that typically used in NZ for sub-assemblage testing for seismic conditions. This has 2 or 3 reversed cycles to about ¾ of the load to cause the nominal beam moment to be reached at the critical section, the column face here. The first loading direction was upward (positive bending moment) in these tests. These "elastic" load-controlled cycles are to ascertain the pre-cracking and post-cracking beam stiffness as well as to check the reliability of all the gauges being used. Often one or 2 portal gauges are found to be malfunctioning and they are replaced at this point. Any movement in the test rig was measured by portal gauges at the load reaction points. These gauges were fixed back to the strong wall, and this "slop" was removed when calculating the actual beam deflections.

From the beam deflection at ¾ of the nominal moment (M_n) the expected first yield deflection was extrapolated and it was to twice this deflection that the beam was pushed during the first ductility 2 cycle (inelastic cycles). These were displacement-controlled cycles. Two reversing cycles to twice the + and – beam yield deflection were completed. Subsequent double reversing cycles to 4, 6 and sometimes 8 times the first yield deflection were then usually completed or until the unit reached less than 80% of the load sustained in the previous cycle. If this load limit was not reached in the current displacement cycle then the unit was assumed to have "failed" and testing stopped.

Unit 2 beam elongation jack:
A second hydraulic jack was used in this test to load the beam (in compression) along its longitudinal axis to resist any beam elongation by keeping the beam mid-depth point at the beam's free end on the same vertical line throughout the test. This was accomplished by adjusting the force in the horizontal jack during testing, usually at the peak of each cycle and when the vertical beam force was removed. The applied vertical force was made up of two components, the load in the vertical jack and the small vertical component of the "horizontal" jack force. The second jack did not remain horizontal when the beam was deflected up and down due to its mounting condition. In multi-bay frames the outer columns are resisted from moving outwards as plastic beam hinges form next to the internal columns. This puts compression into the first floor beams and may cause plastic hinging to occur in the exterior columns, either just below the first floor beams (see Figure 6) or in the first floor joint region. The aim of unit 2 was to model this phenomenon.

Unit 1 Test Results:

Before any load was applied to the unit 3 sets of all gauge zero readings were taken. In the "elastic cycles" flexural cracks formed in the beam with the predominant crack at the column face. Two fine diagonal cracks formed in the joint and there were also cracks along the beam bear anchorage length in the joint.

The beam bars yielded in the first cycle to ductility factor (DF) 2 as expected, as shown in the Applied load-beam tip deflection hysteresis loops in Figure 7. The applied moment at the column face was equal to twice the applied force plus about 7 kNm due to the self-weight of the beam. In the first DF2 cycle the load applied reached 144 kN, 14% above the load to reach the theoretical nominal moment at the column face (126 kN) using the actual steel and concrete strengths.

Strain-hardening of the beam reinforcing bars at the column face would have caused this increase.

The beam had a small increase in strength in the first DF4 cycle in both directions with a subsequent 5% drop in sustained load in the second cycle in each direction. In the first upward cycle to DF6 there was decrease in strength to about 15% less than M_n but in the downward cycle the previous DF-4 load was actually exceeded. There was further degradation in load sustained in both the +/-DF8 cycles with the loss being greater in the downward cycles.

Figure 8 shows the components of shear and flexural displacements calculated from the portal gauges in each block along the beam and column and in the joint. It can be seen that the beam flexure predominates with the joint and beam shear deformations and the column flexure being

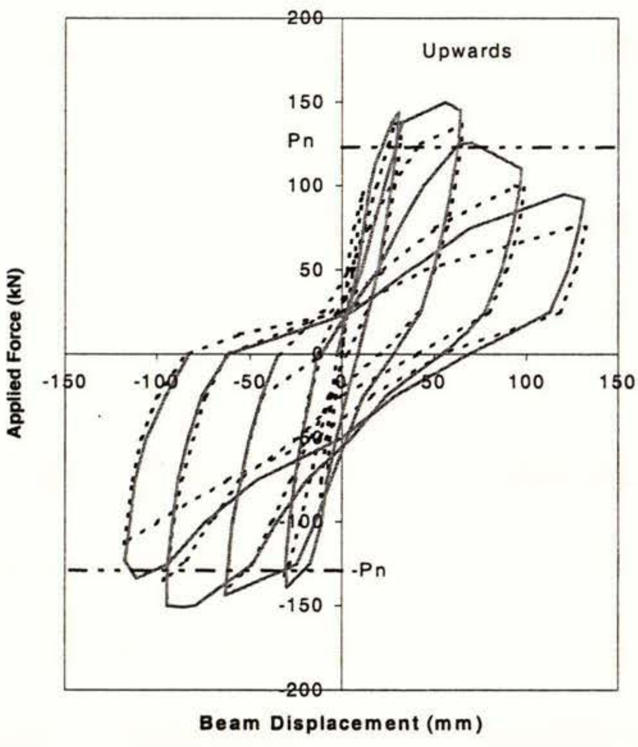

Figure 7: Unit 1
Force-Displacement hysteresis loops

much smaller. Most of the flexural deformation occurred in the beam plastic hinge zone and it was estimated that the top beam bars were yielding to a length of 300 mm from the column face while the bottom bars were only yielding out to about 140 mm.

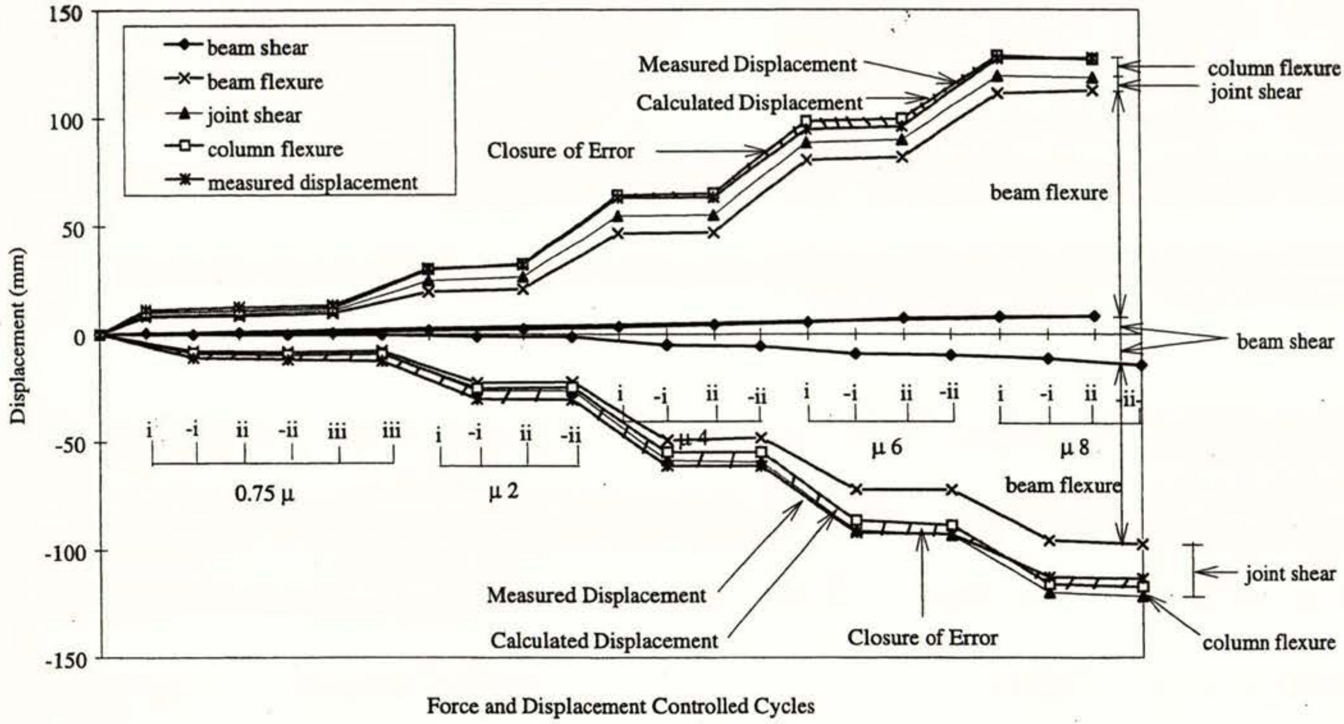

Figure 8. Unit 1 Components of flexure and shear deflections at cycle peaks.

The column face crack steadily increased in width till it was about 18 mm wide at the level of the bottom beam bars in the second DF6 cycle. Substantial beam bar slip within the joint had occurred to get this substantial crack width. Appreciable spalling had occurred in the joint zone and in the column immediately under the joint. The loss of strength in the unit was associated with this spalling. Figure 9 shows Unit 1 in the second cycle at DF6 with obvious damage to the column and the formation of a column plastic hinge just below the joint.

Mechanical strain gauges on the column reinforcement indicated that extensive yielding occurred in the column bars closest to the inside face of the column. In the ductility 2 cycles the average strains over a 100 mm gauge length reached about 5.5 yield strains on these bars at the level of the lower beam bars, while during the ductility 4 and 6 cycles these values increased to approximately 12 yield strains. Localised strains could have been appreciably greater than these average values.

The high strains on the inside longitudinal column bars in the locality of the bottom reinforcement in the beam caused wide cracks to form along these bars in the joint zone. This reduced the bond resistance and was responsible for the slip of the reinforcement and the large crack at the face of the column. The strains measured on the column bars indicated that this reinforcement yielded prematurely. A strut and tie diagram showing the flow of the forces in the joint zone is shown in Figure 10. It can be seen that anchoring the beam reinforcement short of the column bars reduces the internal level arm of the flexural forces in the column on the underside of the joint zone (section 1-1). The diagonal compression force in the joint zone, "D", which balances the compression forces in the beam and upper column, anchors against the tension force, "T", at the hook in the bottom beam bars. As indicated the compression force in the lower column is forced to deviate in towards the centre of the column to balance the vertical component of the diagonal force, "D". This reduces the internal lever-arm resulting in a localised loss of flexural strength. This reduction could be avoided by increasing the amount of longitudinal reinforcement placed on the inside face of the columns. To allow for this effect the four 16 mm bars placed on the inside face of the columns in units 1 and 2 were replaced with four 20 mm bars in units 3 and 4.

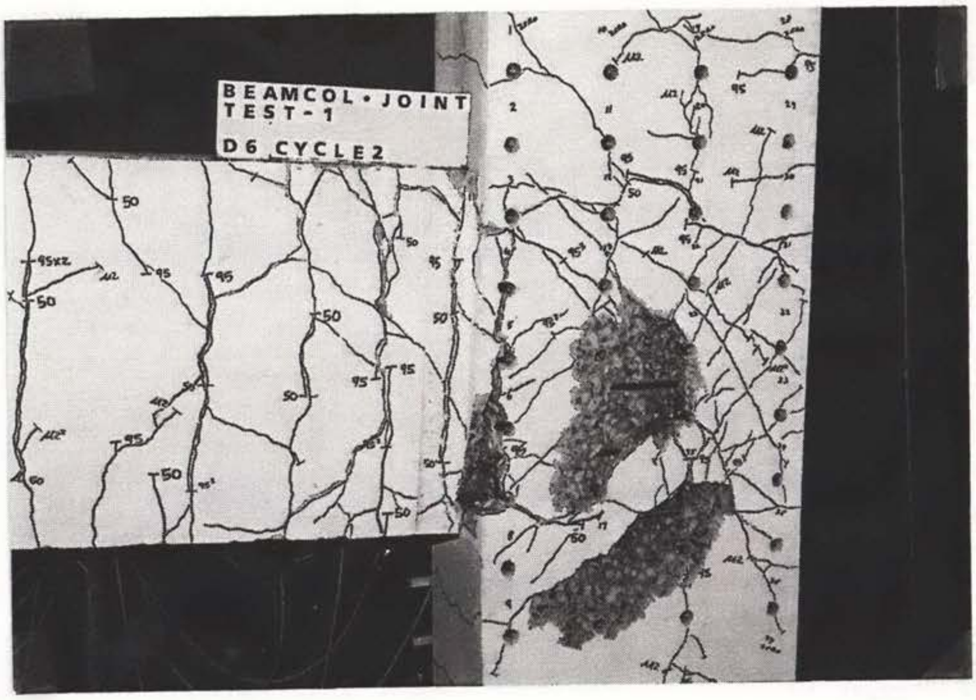

Figure 9. Unit 1 at Ductility 6, second cycle.

The measured strain in the column bars located next to the inside face of the column, immediately under the joint zone, was of the order of 0.02 when the maximum upward load was sustained by the unit. Under this condition the axial tensile load in the lower column was 133 kN. This value is equal to the jacking force of 144 kN minus the dead load of the beam and upper column, which was equal to 11 kN. Using these two values the theoretical bending moment resisted by the section is found to be 135 kNm, which was close to 10 percent greater than the actual bending moment resisted by this section under the action of the maximum measured jacking force. From this it was concluded that this strength loss was due to the reduction of the internal lever-arm, which is illustrated in Figure 10. To compensate for this loss in strength the reinforcement can be designed using standard theory for an increased bending moment, M_n'. The loss in strength is assumed to be proportional to the ratio of the distance between the external column bars and the plane containing the bent up beam reinforcement and the distance between the inside and outside column bars. This ratio is given by "a/gh", where the dimensions a and gh are shown in Figure 10. In the test of unit 1 the ratio a/gh was equal to 0.22 and the strength loss is 10 percent. Hence the value of M_n' is given by-

$$M_n' = M_n\,(1 + 0.46\,a/gh) \qquad (1)$$

Using standard theory and the M_n' value should give a flexural strength of M_n. This is achieved by increasing the area of the longitudinal reinforcing bars located adjacent to the inside face of the column.

As can be seen in Figure 7, the strength degraded more rapidly for the upward loaded half cycles than when the loading was downwards. Yielding in the joint tie legs only occurred in the top leg of the outer joint tie during the second ductility 4 cycle. However in that cycle all tie legs had recorded strains exceeding 60% of the yield strain, measured at the column centreline, with the outer tie legs having higher strains than the corresponding inner legs.

One feature common with exterior beam-column joints is the separation of the cover from the back of the column in the joint region. This mode of deformation, which significantly reduces the column's flexural strength did not occur in unit 1, due to the anchorage tails of the beam bars being a quarter of the column depth inside the column. Normally these legs would be close to the outer column bars and as they are stressed they attempt to straighten out causing the columns back cover to split off.

Unit 2 test results:
As earlier described Unit 2 had an additional loading jack in the horizontal direction applied at the beam's free end to counter the beam elongation during plastic hinge formation.

The pre-yielding cyclic behaviour of Unit 2 was similar to the first unit but the inelastic displacement cycles showed marked differences. Figure 11 shows how the elongation in beams 1-4 increased steadily with the ductility level to reach a value of 16 mm in Unit 2 (close to 3% of

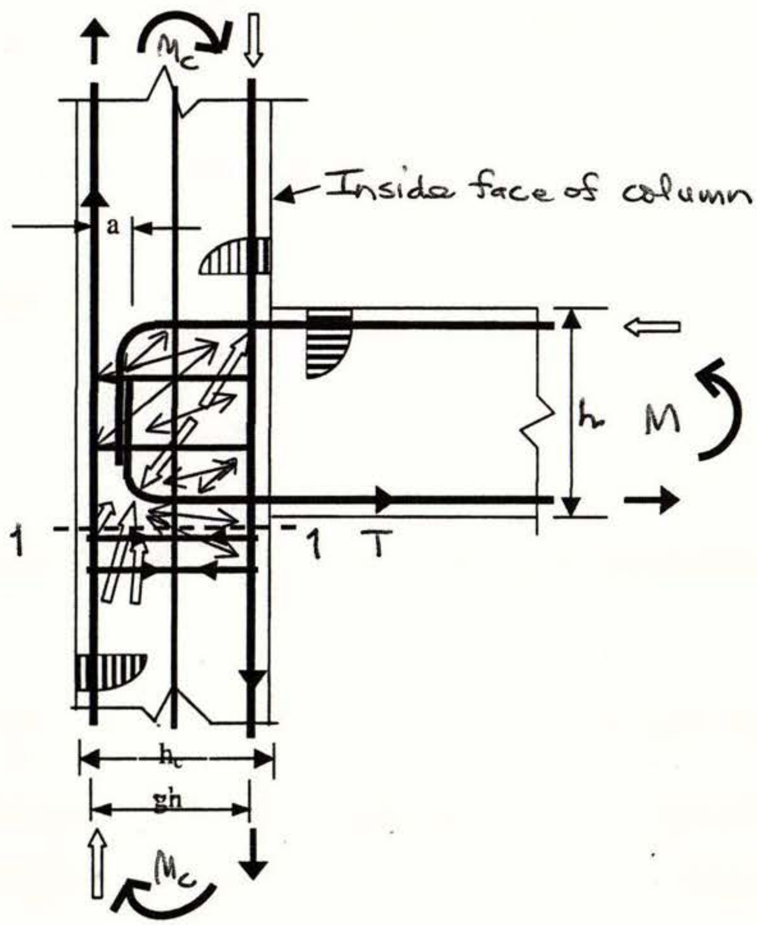

Figure 10. Actions in an external beam-column joint.

the beam depth) at the end of the test. The beam axial force ranged between 110 and 210 kN, but it averaged close to 150 kN. It was typically 25 kN higher for downward loading than for upward loading.

The beam elongation pushed the column outwards and this was associated with extensive yielding of the longitudinal column bars placed close to the outside column face. In the ductility 4 cycles this reached 12 yield strains and in the ductility 6 and 8 cycles it was of the order of 18 yield strains. The bending of the column due to beam elongation induced compression, or reduced the tension forces, on the reinforcement on the inside face of the column. As a result the bond conditions for the beam bars were very much better than in unit 1. This was reflected in greater length of yielding reinforcement in the beam in unit 2 when compared with unit 1, and a greatly reduced crack width in the beam at the face of the column. The tensile curvatures reached in the beam between 60 and 495 mm (beam effective depth, d) out from the column face were over twice those reached in Unit 1. The damage is shown in Figure 12 at the second cycle during testing to ductility 6.

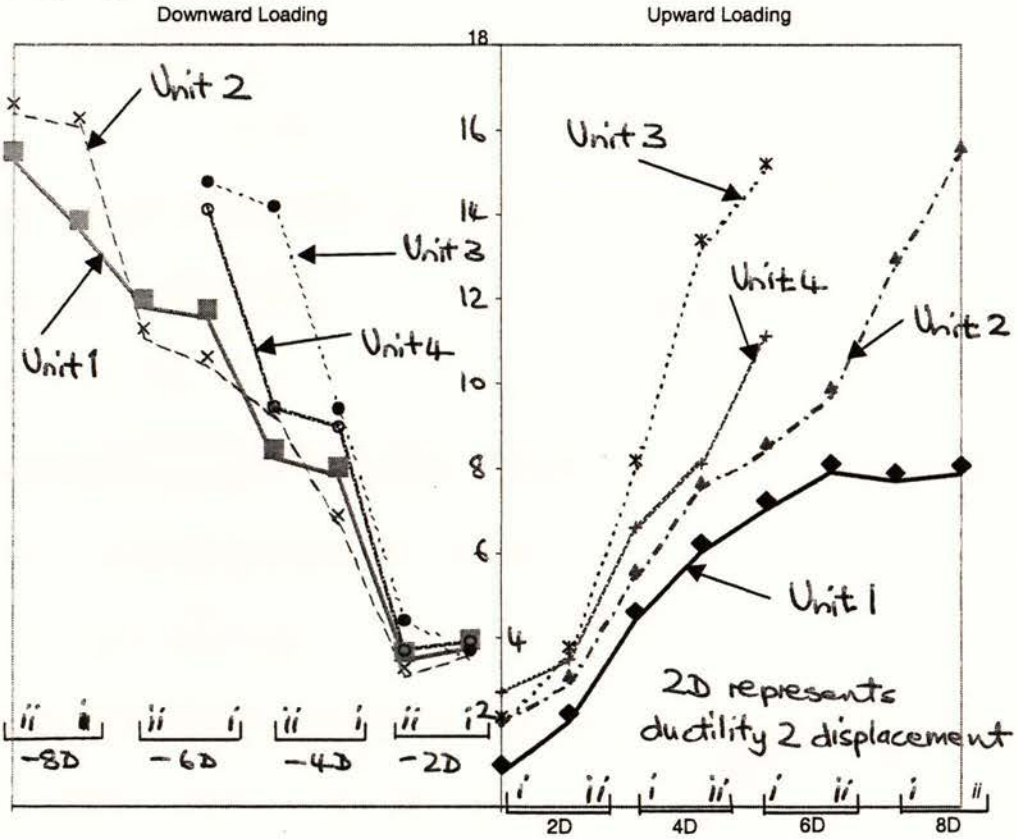

Figure 11. Beam Elongation for Beam-Column units 1-4.

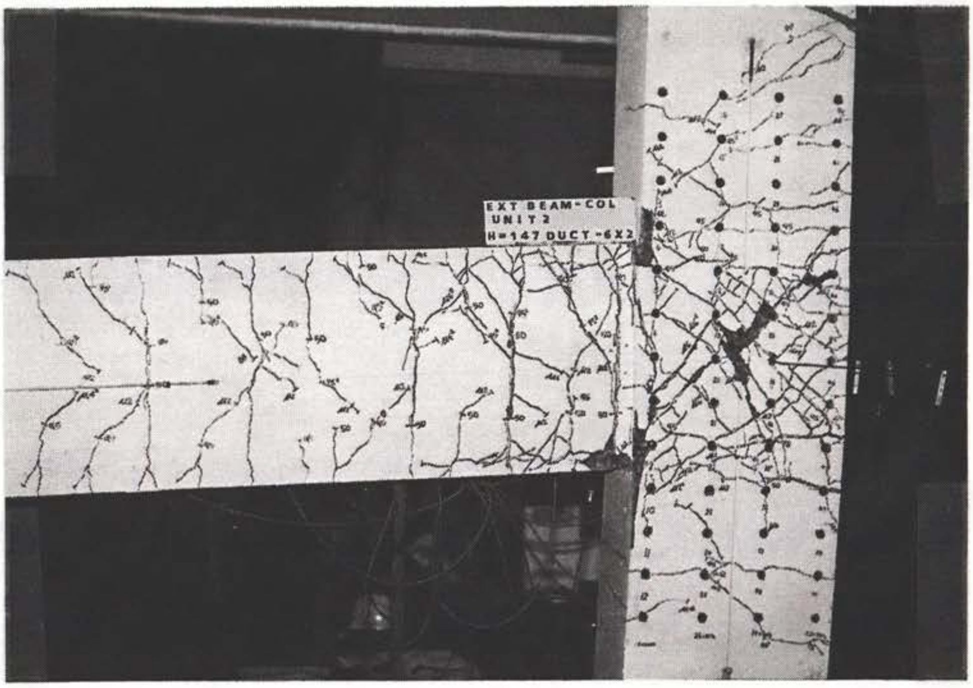

Figure 12. Unit 2 at Ductility 6, second cycle.

During the testing of Unit 2 substantial inelastic deformation developed in the joint zone and spalling occurred in this region during the ductility 6 and 8 cycles, which led to the loss of strength. All the joint ties yielded in this unit by or during the ductility 6 cycles, with the mid-joint ties reaching strains of about 5 yield strains.

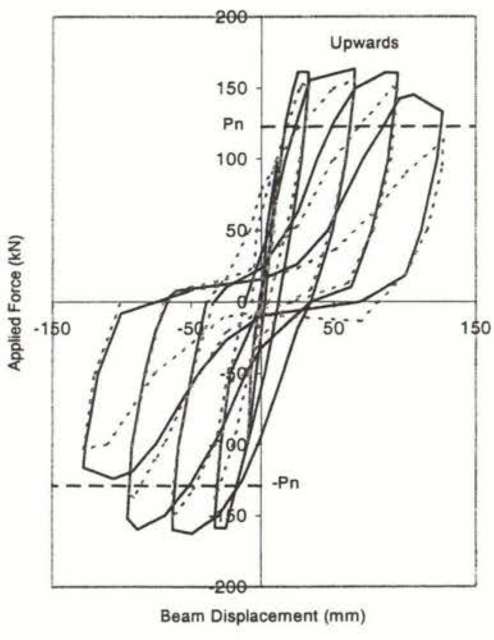

Figure 13. Applied Force- Beam Displacement loops for Unit 2.

Figure 13 shows the force applied to the beam versus the displacement of the load point for unit 2. It can be seen that the strength was greater than for unit 1. This increase arose from the action of the axial compressive load on the beam. At a value of 150 kN this gives an increase in beam flexural strength of close to 32 kNm. Strength degradation did not occur until the ductility 8 cycles. The enhanced strength of this unit arises from 2 factors, namely;

- The axial compressive force increased the beam's flexural strength;
- The axial force in the beam suppressed the flexural tensile forces on the inside of the column adjacent to the beam. This prevented the structural performance of the joint zone being reduced by bond degradation of the beam reinforcement in the joint zone.

Analysis of the results indicates that the axial force, arising from elongation of the beam, was a function of the flexural strength of the column. For downward loading, with an axial force of 200 kN acting horizontally on the beam and the theoretical flexural strength of the beam acting at the column face, a bending moment of 201 kNm is induced in the column on the lower face of the joint zone. This can be compared with the theoretical section strength of the column of 202 kNm (see Table 2). With upward loading the flexural strength is reduced due to the drop in the column axial force in the critical section, which with this direction of loading is now located immediately above the joint zone. This was the reason for the observed decrease in beam axial force for upward loading.

Figure 14 is the components of shear and flexural displacements compared with the measured beam tip displacement. The relative amounts of joint shear deformations are considerably greater than those in Unit 1 (Figure 8) while the corresponding beam flexural deformations are less, especially for upward loading. These observations confirm the greater joint and column damage experienced in unit 2.

From the strain gauges on the beam bars in unit 1 yield penetration extended past the 90-degree bend in the anchorage length but in unit 2 it didn't extend as far as the bend.

Table 2. Theoretical flexural strengths of beam and column sections Units 1-4.

Unit	Beam	Column (kNm)				
	M_n (kNm)	Lower column under maximum axial tension	Lower column under maximum axial compression	Upper column under dead load, outside edge in tension	Upper column under dead load, outside edge in compression	Strength ratio, beam to column strengths
1	252	145	202	173	173	1.32
2	252	145	202	173	173	*
3	268	202	206	173	233	1.49
4	270	202	206	173	233	1.48

- Dependent on beam axial load.

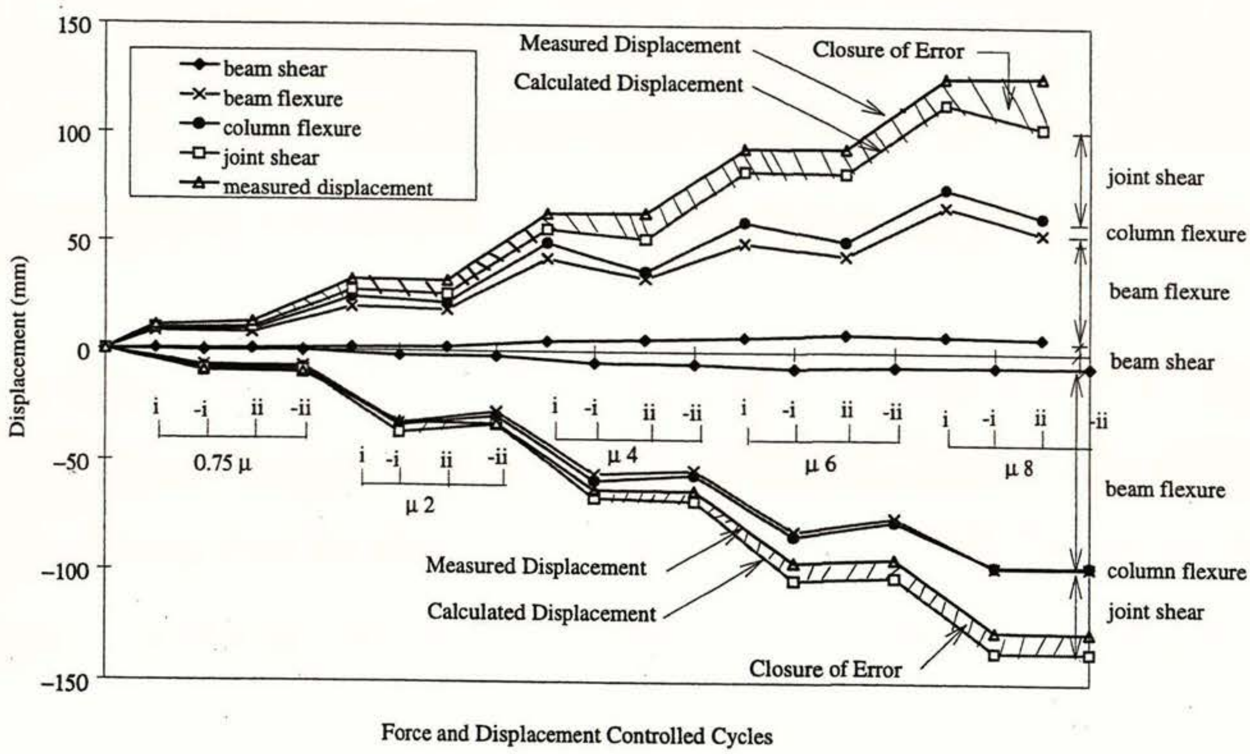

Figure 14: Unit 2 components of flexure and shear deformations (Henao [3]).

Unit 3 test results:

Unit 3 had U-bar beam anchorage and 4 larger D20 bars on the inner column face (replacing 4-D16 in units 1 and 2). This unit preformed well up to the ductility 6 cycles with a beam plastic hinge forming with only minor cracking in the joint region (maximum crack width of 0.2 mm). Figure 15 reproduces the applied beam force-beam displacement loops for unit 3. There was a substantial decrease in applied force in the second cycle to ductility 6 after the nominal strength had been exceeded in all the preceding inelastic cycles.

The column face crack widened steadily during the test while the majority of the deformation was due to beam flexure. As the test progressed the proportion of the beam deflection due to joint shear increased, while the beam flexural deformations remained about constant from the ductility 4 cycles on. This implied some loss in strength occurred in the joint zone and plastic hinge zone of the beam.

Unit 3 had the larger D20 bars on the column's inner face in an attempt to overcome the local reduction in column flexural strength at the joint zone that was observed in unit 1. The mechanical strain gauges indicated that only very limited yielding, if any, occurred on the column bars during this test.

The components of shear and bending displacements are shown in Figure 16 from which it can be seen that the beam shear deformations continued to increase during testing whereas the beam flexural deformations remained approximately constant from ductility 4 onwards. At the end of testing the beam plastic hinge zone extended out to 530 mm along the top of the beam from the column face. Spalling of the cover concrete along the bottom of the beam occurred during the later cycles.

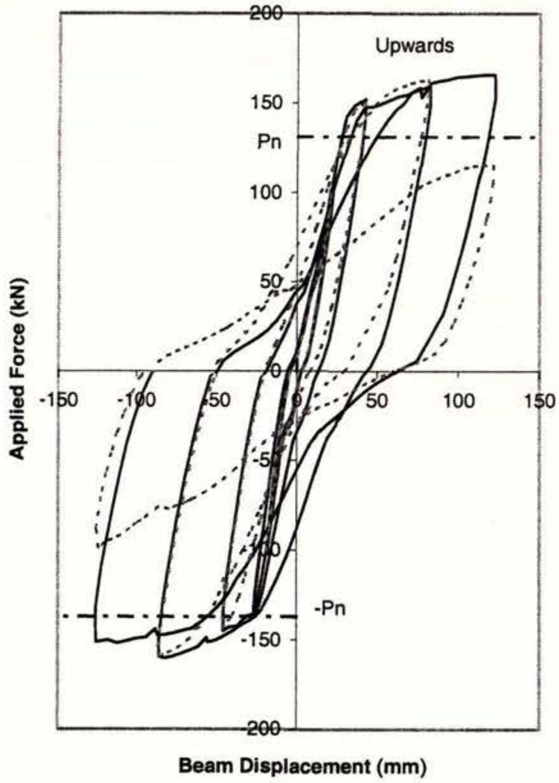

Figure 15. Unit 3 Applied Force- Beam Displacement loops.

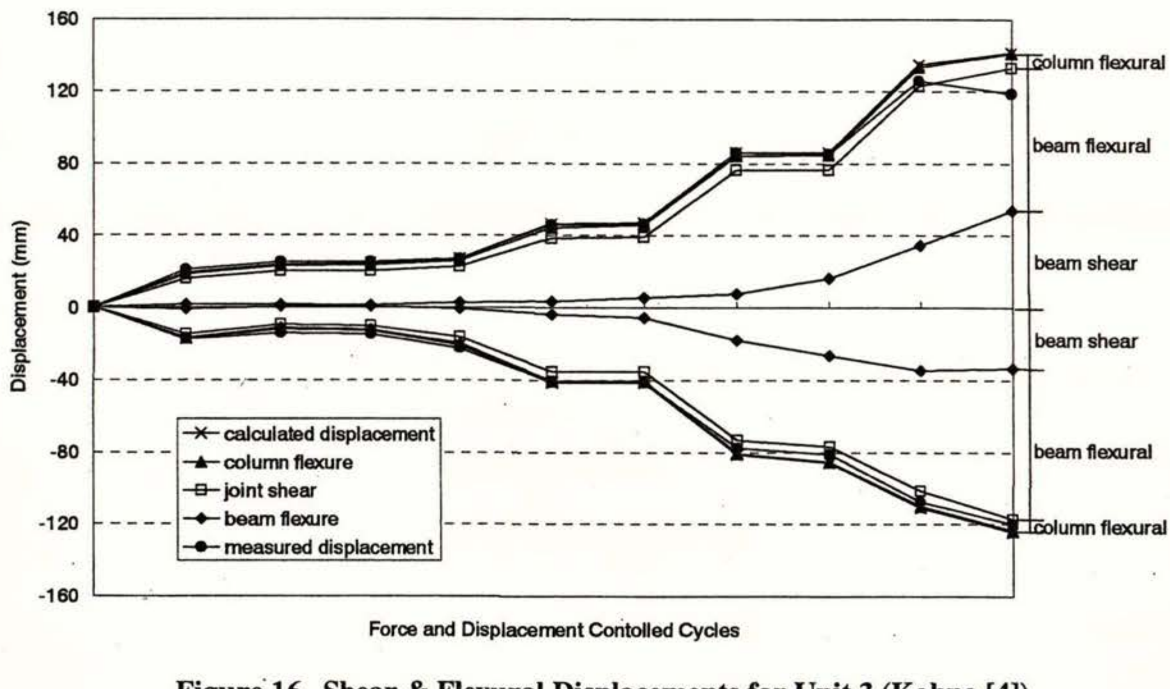

Figure 16. Shear & Flexural Displacements for Unit 3 (Kohno [4]).

Unit 4 test results:

This unit had the beam bars spread uniformly over the beam depth by incorporating horizontal U-bars (D16H). The loading sequence included an extra single reversed cycle to ductility 1 deflection before the usual double cycles to ductility 2 and beyond.

The ductility 1 deflection in this unit (nearly 1.2% drift) was greater than in the previous three units. This is in part due to the uniform distribution of longitudinal reinforcement in the beam, which reduced the section stiffness, based on the cracked section, by close to 15 percent. However the theoretical flexural strength was almost identical to that of unit 3. The jacking force applied to the beam versus displacement relationship is shown in Figure 17. The theoretical over-strength force (151 kN) to reach the beam section's theoretical over-strength moment was reached in the first upward ductility 2, 4 and 6 cycles and in the first downward ductility 4 cycle (108 mm deflection). In the second ductility 6 cycle the jacking force decreased to about 50-60% of that corresponding to the beam's over-strength moment due to the extensive spalling in the plastic hinge zone and buckling of the main beam reinforcement.

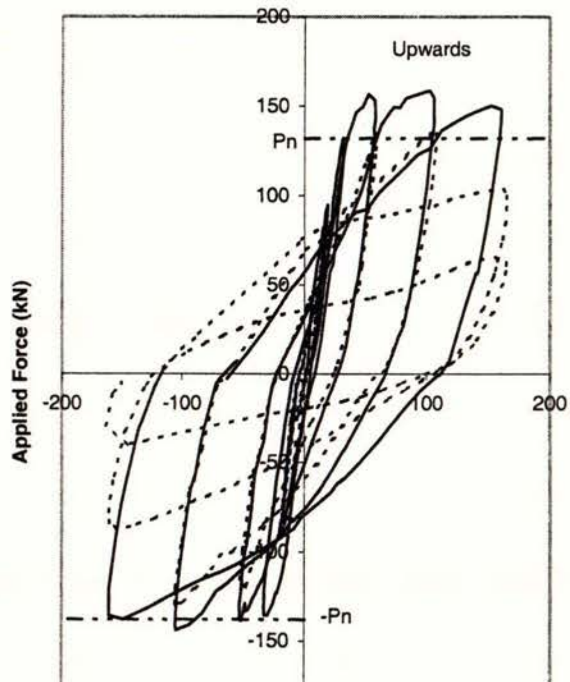

Figure 17. Unit 4 Applied Force- Beam Displacement loops.

The beam flexural component of the total beam deflection was the largest contribution, although the beam shear and joint shear components steadily increased at each successive inelastic cycle. This is shown in Figure 18. However the beam and joint shear components of the deflection was only about half of that measured in unit 3.

Figure 11 shows the beam elongation for the first 4 units tested. The peak values of elongation range between 8 and 16.5 mm. The greatest elongation occurred in unit 2 where there was an axial restraining force on the beam. The lowest value occurred in unit 1, where extensive slip of the bottom beam bars prevented appreciable yield penetration along the beam, thus causing lower elongation.

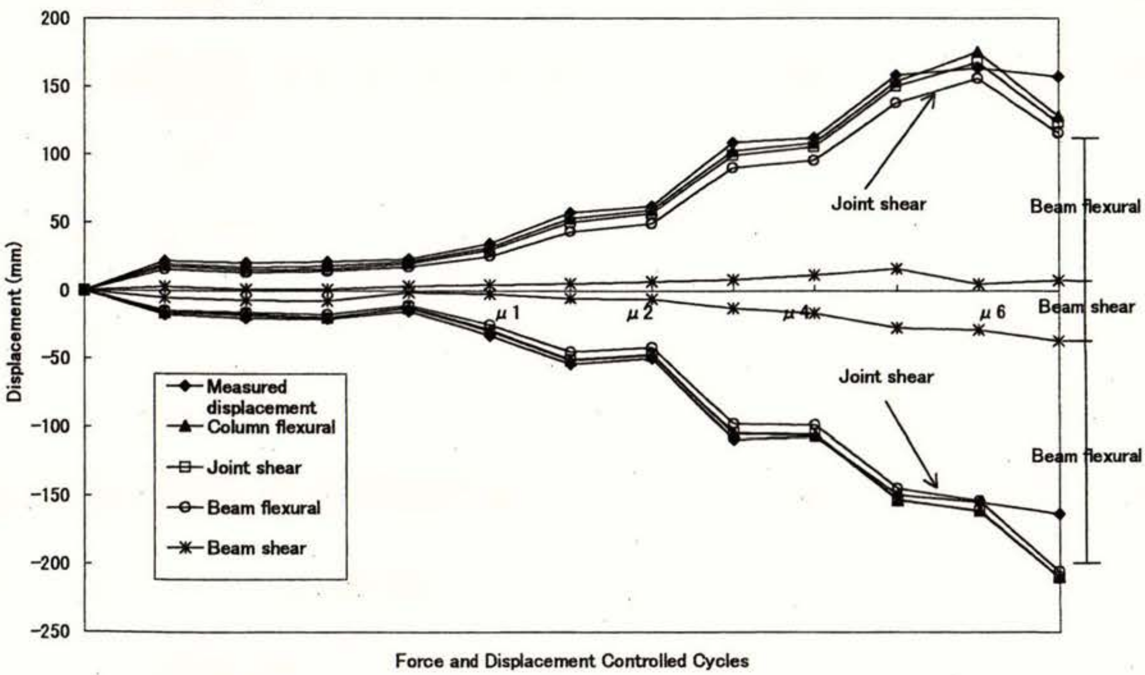

Figure 18. Flexure and shear components of deflection for Unit 4 (Kohno [4]).

The next lowest values were in unit 4, which contained longitudinal reinforcement spread over the depth of the beam. In this case the area of reinforcement in tension exceeded that in compression, and as a result the compression reinforcement tended to yield back in compression to a limited extent, which reduced the elongation.

From these tests it can be seen that, with the exception of upward loading in unit 1, at the design level of ductility for ductile structures the elongation in each plastic hinge zone lay between 2 and 2.8% of the beam depth, as predicted by Fenwick & Megget [6]. The main results found from the testing of Units 1-4 are described in a recent paper by Megget and Fenwick [7].

Unit 5 test results:

Unit 5 was very similar to unit 1 with standard hook anchorage for the beam bars but used the newly available Grade 500E reinforcing first produced by Pacific Steel Ltd. late in 2002. The 4-D20H bars top and bottom in the beam had a yield strength of 572 MPa. The inner row of column bars was increased to 4-D20 bars (instead of D16 bars) to eliminate the column and joint degradation seen in unit 1 caused by the short beam bar anchorage length to ¾ of the column depth.

During the ductility 2 cycles the beam was seen to twist. The reason for this was a slipping support bolt, which was impossible to fully tighten in the bottom connection to the test rig. An extra transverse brace was added to reduce this torsion in later cycles. This twisting caused a small discrepancy in the deflection readings between the portal gauges and the turnpot potentiometer.

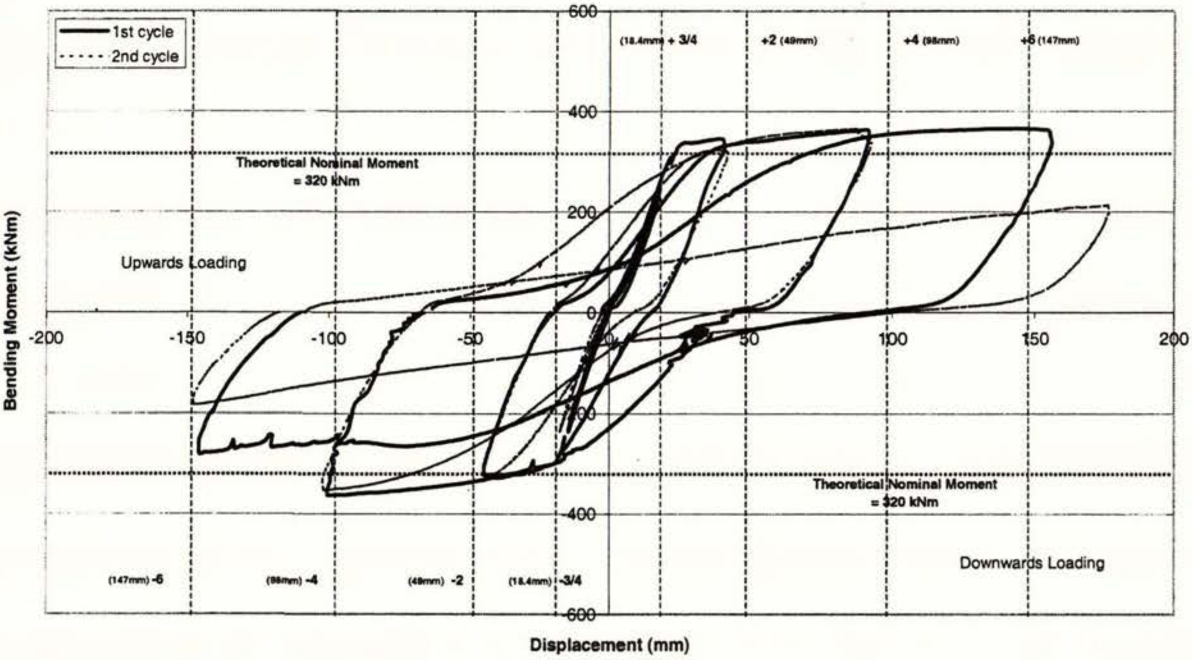

Figure 19. Unit 5: Beam moment (applied force x 2 metres) at column face versus beam deflection at the position of loading.

The column face moment-beam end displacement hysteresis loops are given in Figure 19. As anticipated beam yielding occurred in the first ductility 2 cycles in both directions, the difference in moment (about 30 kNm) was due mainly to the effect of the dead load moment of the beam. This is the reason why the upward yield moment is larger than the downward applied moment at yield. Note that the first yield beam displacement (24.5 mm) is equivalent to a lateral drift of 1.09%. Thus the ductility 4 displacements are equal to nearly 4.4% drift.

The unit exhibited good stable, ductile hysteresis loops up till the second cycles to ductility +/-6 where a substantial decrease in sustained load was measured. There had been a 20% drop in applied load between the first upward and downward cycles at ductility 6 but this decrease was greater in the second reversed cycle. There was concrete cover loss up to 125 mm out from the column face and this exposed severe buckling of both the top and bottom beam bars. This buckling caused the sustained load to drop to about half of the maximum load reached during the test in both directions.

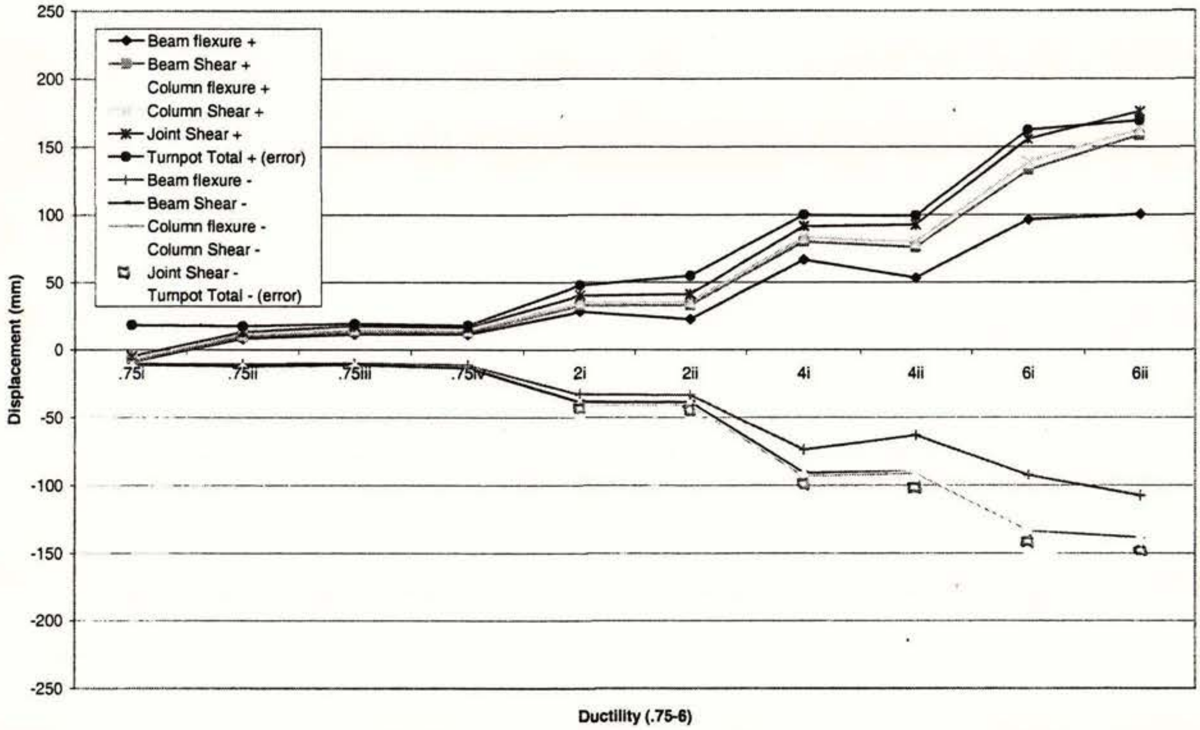

Figure 20. Unit 5: Components of flexure and shear deflection at cycle peaks.

The flexural and shear components of the beam deflection at peak loads are shown in Figure 21. It can be seen that the predominant beam flexure continues to increase during testing but that the beam shear component increases at a greater rate in the ductility 4 cycles on. The major diagonal beam shear crack opened to a width of 13 mm during the ductility 4 cycles emphasising the increased shear deformation and degradation in the plastic hinge zone. Figure 21 shows the plastic hinge zone in the ductility 6 cycle, with the obvious global buckling of the bottom beam bars shown in more detail in Figure 22. On load reversal these buckles would straighten out but with considerable loss of core concrete and subsequent loss of load carrying capacity.

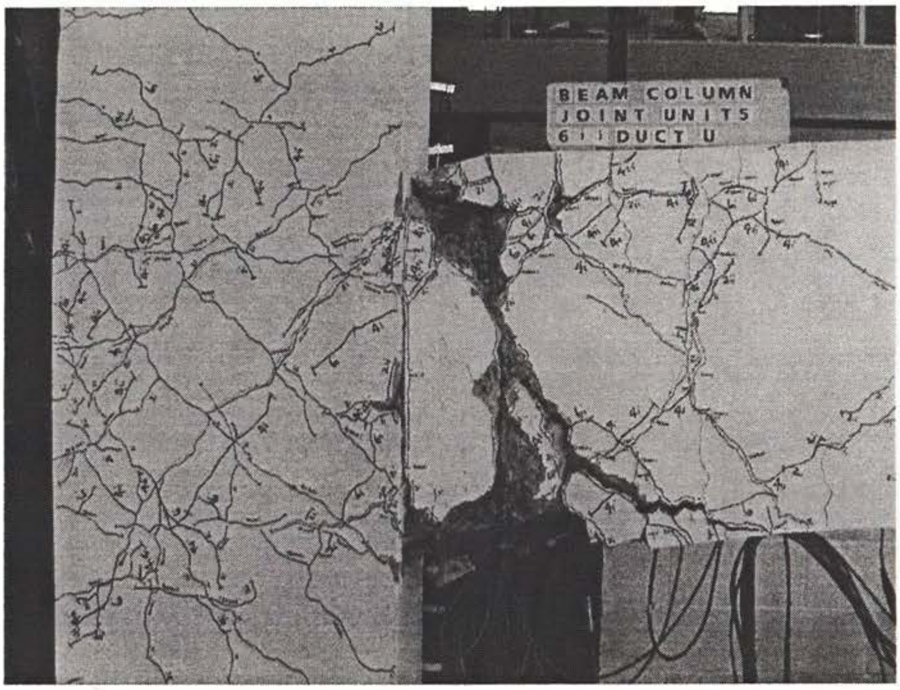

Figure 21. Unit 5 beam PHZ at end of the second downward ductility 6 cycle.

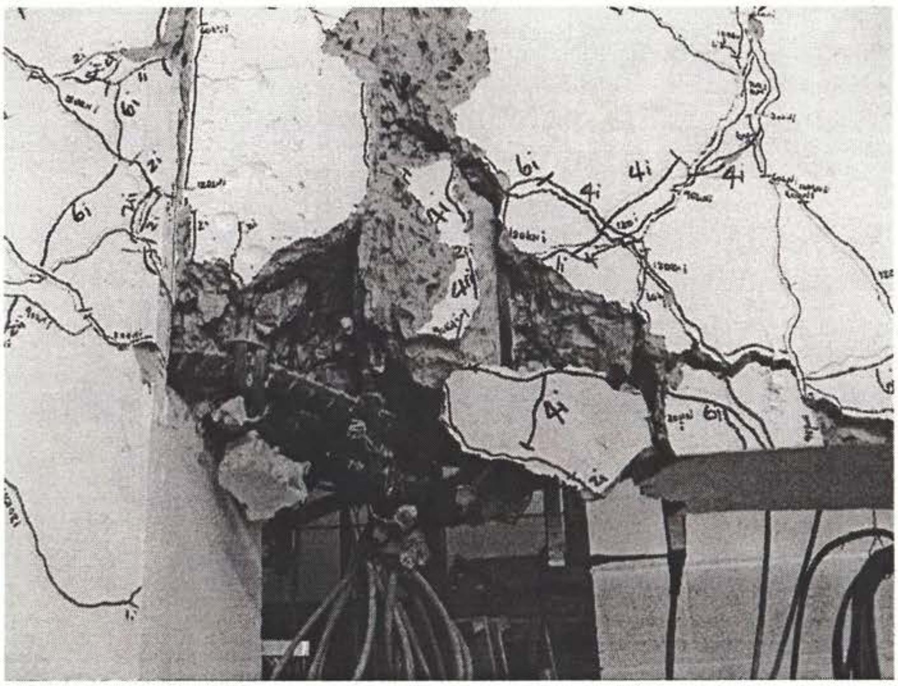

Figure 22. Unit 5 showing buckled beam bar detail, at the end of the second downward ductility 6 cycle.

The joint region of unit 5 remained virtually undamaged during testing with only minor diagonal cracking. The diagonal crack was approximately a maximum width of 1 mm across the joint and there was negligible change in the joint beyond the ductility 2 cycles. No beam bar slip was apparent in this unit, unlike that seen in unit 1. Concrete crushing occurred at the bottom of the beam at the column face and penetrated into the column cover concrete but not into the column core.

It can be concluded that unit 5 with the larger column bars on the inner face (and the double transverse bars within the 90-degree anchorage bends) behaved as expected by the NZ Concrete Standard without severe degradation of the joint region, while unit 1 failed primarily in the joint due to beam bar anchorage loss caused by the column forming a plastic hinge near the bottom of the joint. (The loss of column strength was exacerbated by the short horizontal beam bar anchorage length).

Unit 6 test results:

This unit had unequal top and bottom beam reinforcement, namely 4-D20H top and 2-D20H bottom. The ductility 1 displacement (viz. first yield) for both loading directions was extrapolated from the ¾ ductility displacement for the larger down ward load (negative moment, 4-D20H bars yielding). The average first yield displacement was taken as 22.8 mm (1.01% drift) from the 4 downward "elastic" ¾ yield displacement cycles completed.

Unit 6 suffered a premature failure and sudden loss of strength when one of the bottom beam bars snapped in a brittle way during the second upward cycle to ductility 4, at a displacement approaching 2% drift. This fracture occurred at the position where a stud had been arc welded on for attaching a portal gauge. The recommended preheating (personal correspondence by Keith Towl, Pacific Steel) had not been done to the beam bars prior to the welding and this appears to be the reason for the bar failure. The test was stopped after the subsequent downward cycle to ductility 6 displacement. The beam force-displacement loops are shown in Figure 23 with the loss of strength recorded immediately after the bar fractured being clearly apparent. Note that the bar fractured at a position where it was also buckling downwards, see Figure 24.

Before the bar failed the unit behaved in a ductile manner with the nominal moment (positive M_n = 147.3 kNm and negative M_n = 300 kNm) being reached in both directions in the first cycle to +/- ductility 2. There was some apparent loss of stiffness in the second ductility -2 cycle downwards) and also in +2 ductility prior to the bar fracturing. This appears to be due to an increase in the beam shear deformations during the ductility 2 cycles as seen in Figure 26. This increase continued till the test was stopped prematurely. The beam shear contribution to the total beam deflection was only about 10% greater in the downward direction when compared to the upward loading direction. Early in the test shear cracking only occurred in the beam when loaded downwards due to this load being about twice the yielding upward load. Some (2 or 3) diagonal cracking occurred under upward loading as extensions of the vertical flexural beam cracks during the ductility 4 cycles. In these cycles the shear cracks in the opposite direction had widened to 2.5 mm maximum, 5 times wider than during the ductility 2 cycles. Figure 25 shows Unit 6 at the end of the test at the only downward cycle to ductility 6 (~6% drift). From Figure 26 it can be seen the deformation contributions from column and joint flexure, and joint and column shear are negligible.

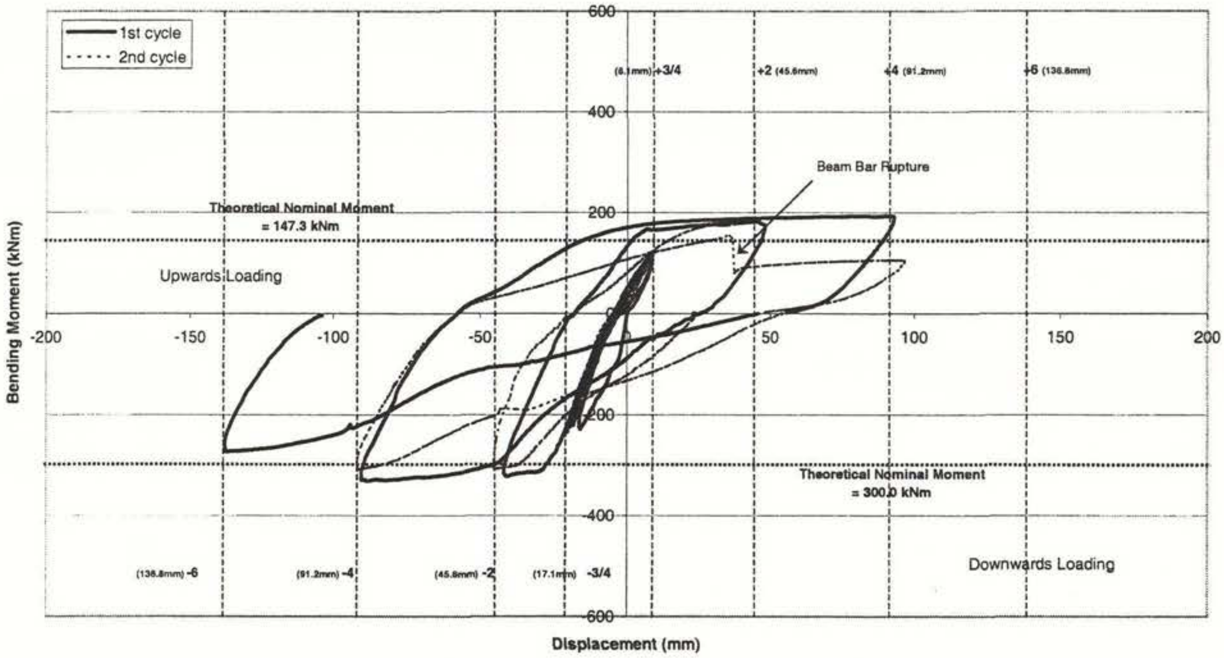

Figure 23. Unit 6 Beam bending moment at column face – Beam tip deflection hysteresis loops. Shows sudden drop in applied force when beam bar fractured.

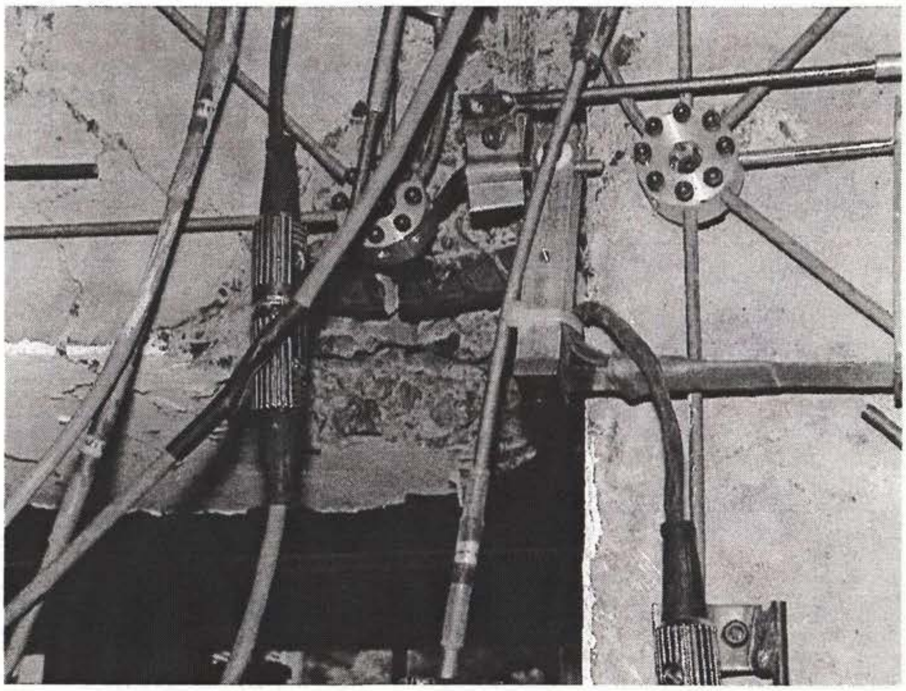

Figure 24. Unit 6 showing the beam bar fracture at stud weld location. Note fracture is at location of bar buckle.

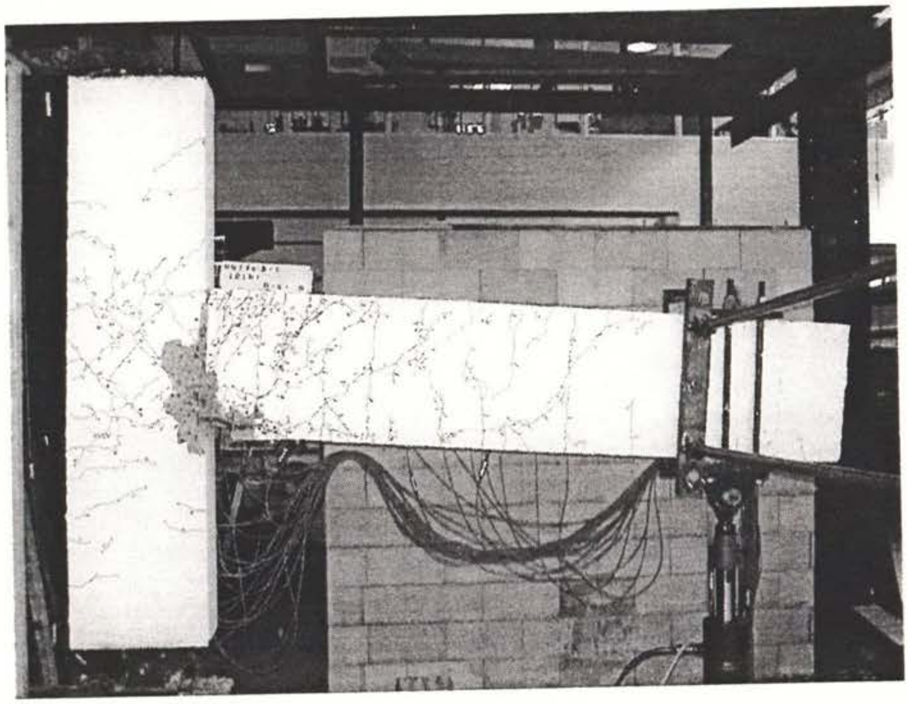

Figure 25. Unit 6 at the end of the downward cycle to ductility 6.

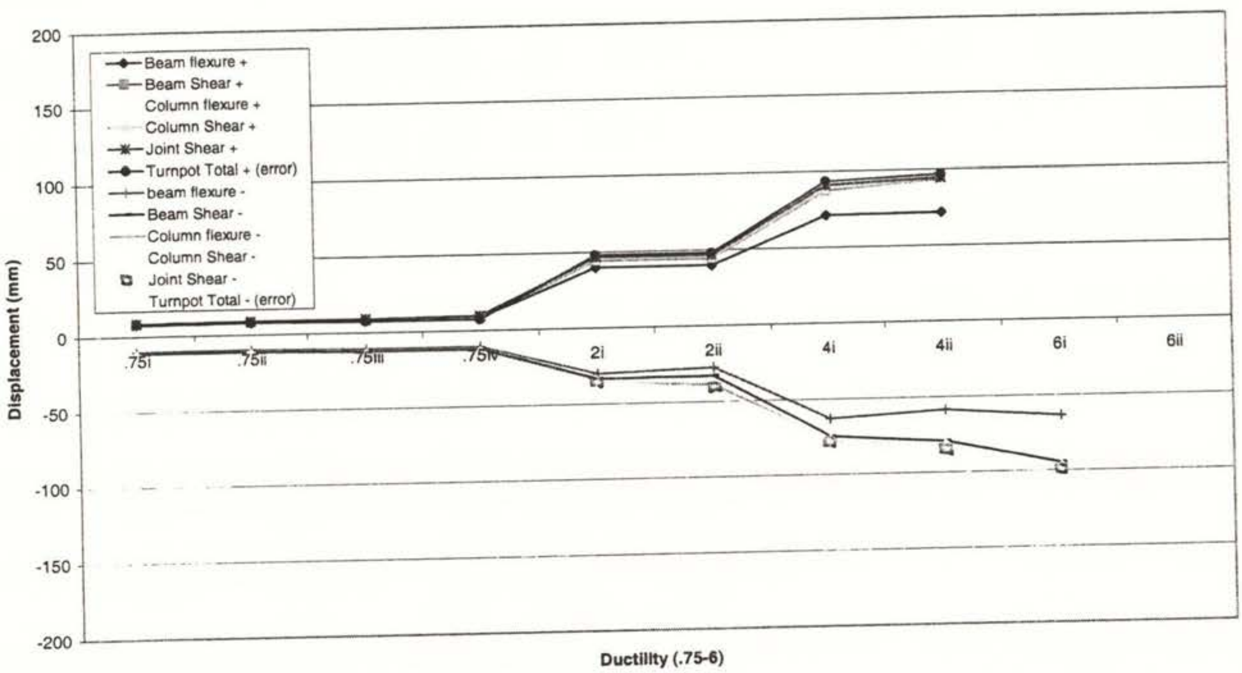

Figure 26. Components of flexural and shear deformation for Unit 6.

Diagonal cracking across the joint faces occurred in both directions but the maximum crack width was 0.8 mm at the centre of the joint during the ductility 2 cycles. Most cracks were about 0.1 mm wide. Concrete spalling occurred during the ductility 4 cycles at the joint/beam interface with the inner corner column bar being exposed near the bottom of the joint. Other than this minor cracking there was no other apparent damage or bar slip in the joint.

Unit 7 test results:

Unit 7 was identical to unit 5 except that U-beam bar anchorage were used in place of "standard hooks" and only 4 tie-sets were used as horizontal joint shear reinforcement, where 5 were theoretically required by the Concrete Standard.

First yield was again extrapolated from the 3/4 yield displacement cycles and averaged out at 22 mm (0.98% drift). This unit showed the best cyclic behaviour of any of the 7 units tested up to this point, with "fat" energy absorbing hysteresis loops right up to and including the first cycle to +/- ductility 6, as shown in Figure 27. The nominal moment in each direction (304 kNm) was exceeded in both directions during the ductility 2 cycles, reaching face moments of 340 and 370 kNm in the upwards and downwards loading directions, respectively. Only in the second cycle to ductility 6 was there an obvious decrease in reloading stiffness and the load sustained at ductility 6 was about 85% of that in the previous cycle. In the subsequent downward cycle to ductility 6 the load reached 78% of the load sustained in the first cycle to +6 ductility.

From the contributions of deformation to the beam deflection plots at each cycle peak, Figure 28, it can be seen that the beam flexure and beam shear contributions continued to increase during testing with the beam shear showing a relatively greater increase during each cycle from the ductility 4 cycles on. The other flexural and shear deformations remained almost unchanged from the first inelastic cycles (ductility 2, 2% drift). The width of the 3 main flexural cracks at the column face, 100 mm and 250 mm out from that face were 11, 3 and 2.6 mm wide during the ductility 4 cycles and plastic hinging appeared to have formed over a length of about 500-600 mm from the column face. The beam shear contribution to the displacement increased from 13% at the end of the ductility 2 cycles to about 27% of the total deflection at the peak of the final ductility 4 cycles. During the final cycle (cycle 2 to ductility 6) the beam flexure contribution dropped to 48% (from 70%) while beam shear increased from 20 to 40% of the total deflection. Beam bar buckling occurred in the plastic hinge zone during this cycle.

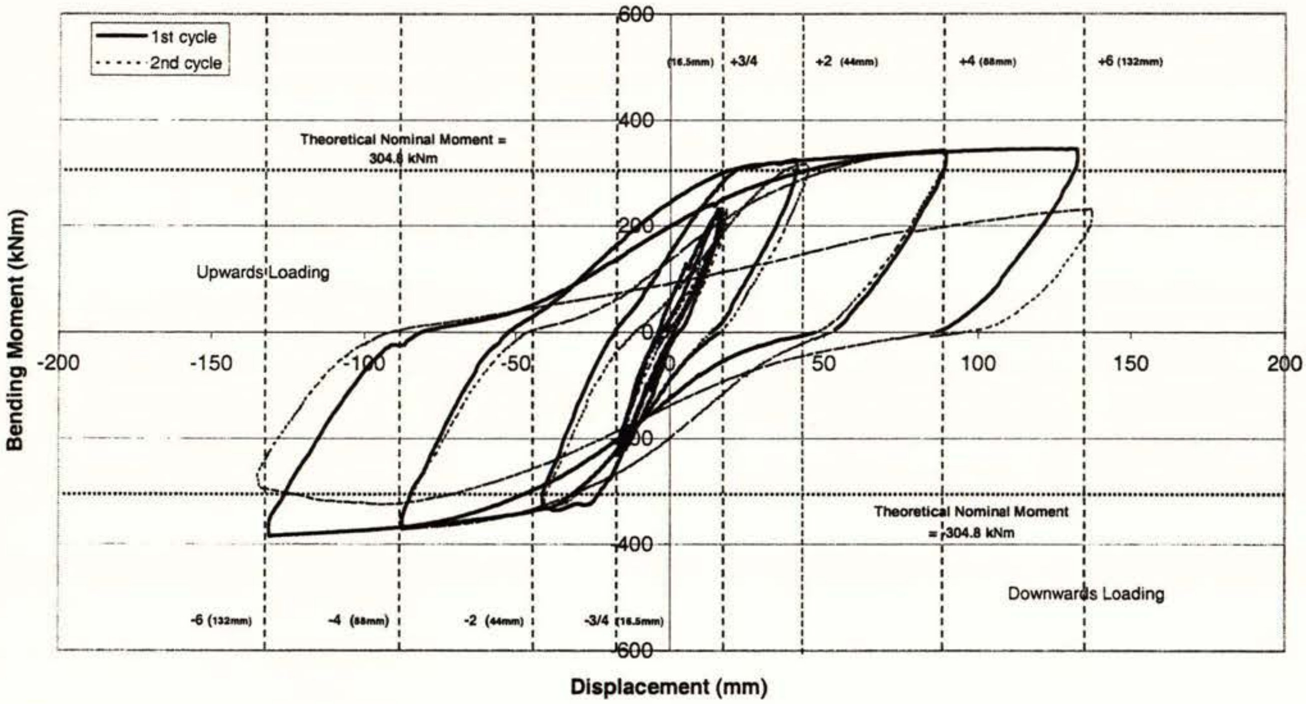

Figure 27. Unit 7 Beam bending moment at column face – Beam tip deflection.

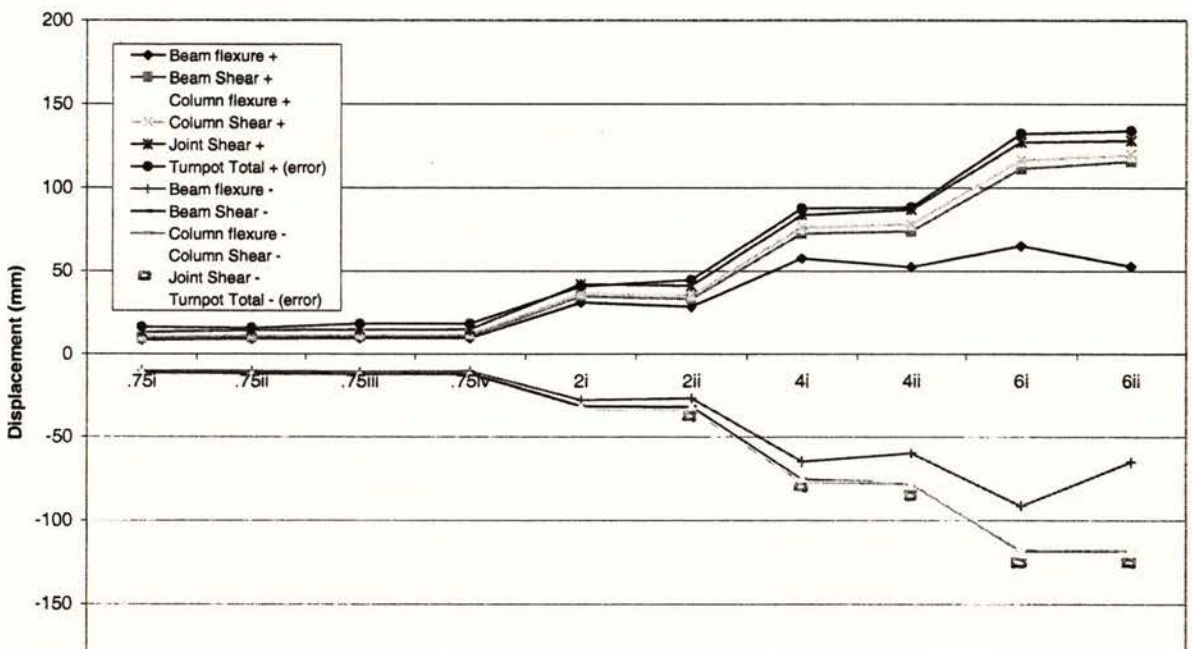

Figure 28. Flexural and shear components of deflection for Unit 7.

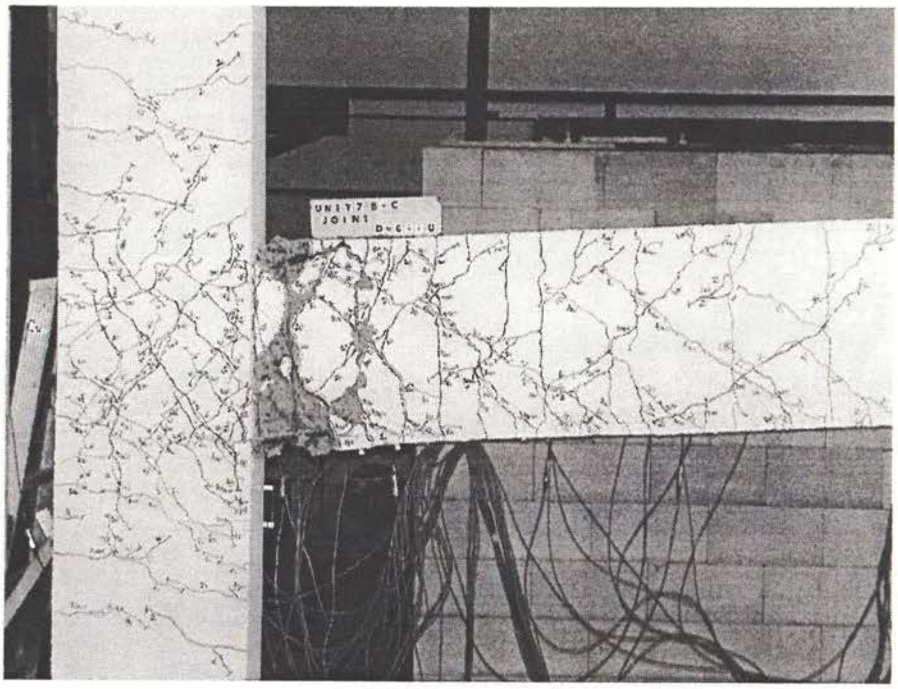

Figure 29. Unit 7 after completion of the second cycle to ductility 6.

The joint region again showed little evidence of distress in this unit except there were a greater number of fine cracks diagonally across the joint in both directions when compared with earlier units. Also there were about twice as many flexural cracks propagating from the back of the joint when compared to unit 6. None of the cracks in the joint were greater than 0.2 mm wide, except for the main diagonal crack from joint corner to opposite corner, which was 1.5 mm wide (0.9mm wide in positive direction of loading). There was almost no change in the joint cracking from the ductility 4 to 6 cycles. Figure 29 shows the unit at the end of the ductility 6, 6% drift cycles. Also there was no apparent slip of the beam bars out of the joint and the whole joint behaved favourably, indicating that a reduction in the current Standard's joint shear requirements may be feasible for low axially loaded columns.

Unit 8 test results:

Unit 8 had near the maximum reinforcement ratio allowed by the Concrete Standard (A_s/bd maximum = 1.67%, actual A_s/bd = 1.63%). Both top and bottom beam bars were 6-D20H.

The theoretical nominal moment for this beam was 431 kNm in both directions using the actual yield stress of the D20 bars (541 MPa) and the concrete compressive strength at testing (41.8 MPa). First yielding occurred in the first cycle to +/- ductility 2 at a displacement of approximately 30 mm (1.35% drift) under downward loading. In the upward loading ductility 2 cycle there was a sudden shear fracture of the bolts between the unit and the bottom column support, which meant removing all the gauges so that the steel plates could be welded together with the unit lying flat. The gauges were then repositioned and the gauges set to their previous values before continuing with the test. Good energy absorbing load-deflection loops were produced for the ductility 2 and 4 cycles as seen in Figure 30 with maximum column face moments of –478 and 472 kNm being sustained at deflections of about -/+ 120 mm (-/+ 5.3% drift). In the second cycle to -/+ 4 ductility there was a drop off in load sustained, falling to near the nominal column face moment (431 kNm) at the cycle peak in both directions. There was a 14% drop in load carried in the negative direction with the corresponding drop being 11% in the other direction, compared to the previous cycle. This unit displayed a drop off in strength before any of the others. All other units, except Unit 6, had reached ductility 6 before a reduction in load carrying capacity was measured.

In the first ductility 6 cycle the load only reached 47% of the maximum reached in the previous cycle. A similar decrease in strength was apparent in the reversing cycle and the test was stopped at 140 mm deflection (6.2% drift) when the load was only about 39% of that reached in the previous cycle.

As with units 5 to 7 Unit 8 exhibited a substantial increase in the beam shear deformations during the first cycles to -/+ ductility 4 and greater, as shown in the peak components of beam deflection in Figure 31. There was actually a decrease in the beam flexural component in the second positive ductility 4 and the first ductility 6 cycles. A similar decrease occurred in the negative direction between the first and second cycles to ductility 4. The decrease occurred as the beam shear component was increasing. As is previous units the components of deflections due to column flexure and column and joint shears were small and hardly increased throughout the test. Figure 32 shows the unit at the end of the ductility 4 cycles with a large vertical crack close to the column face and concrete crushing of the column cover near the beam soffit.

Major beam buckling occurred in the one completed ductility 6 cycle (see Figure 33). There was considerable concrete loss from the inner column face to 150 mm along the beam. At this point the beam deformation contribution had decreased to 41% of the total, while the beam shear component of deflection had reached 56%. Yielding of the shear ties in the beam over a length of about 200 mm out from the column face had occurred with subsequent disintegration of the beam core concrete and buckling of the beam bars, due mainly to the substantial shear deformations in this region. The cyclic performance of this beam hinge zone was not up to the anticipated performance for fully ductile beams in multi-storey frames designed for a structural ductility factor of 6, although the drifts reached far exceeded the current Standard's limits of interstorey drift. This poor cyclic strength may mean that the maximum reinforcement ratio recommended for ductile beams incorporating Grade 500E reinforcing should be decreased.

Beam Elongation:

Figure 34 plots the beam elongations calculated from the portal gauges placed along the outer top and bottom beam bars. Only the elongations at each cycle peak after yielding are shown. The magnitude of the elongations were very similar to those in Units 1-4 (Figure 11) with maximums approaching 20 mm (~3.9% of the beam depth) with a low value of 13 mm in Unit 8 where bar buckling occurred prematurely between ductility peaks of 4 and 6. The high elongation shown for Unit 6 under upward loading is not representative, as this elongation was recorded after the bottom bar had fractured. Details of Units 4-8 are presented in a recent conference paper by Megget, Barton & Fenwick [8].

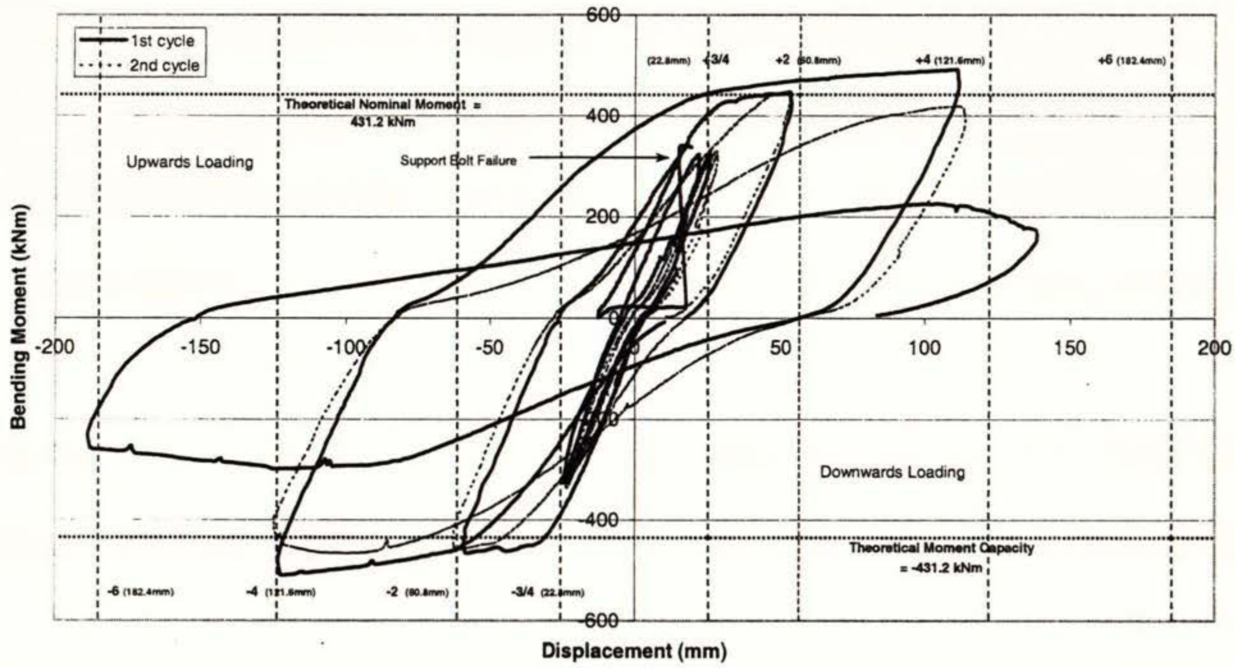

Figure 30. Unit 8 Beam bending moment at column face – Beam deflection hysteresis loops.

Storey drift at Ductility 1:

Figure 35 shows how the ductility 1 story drift varies with the proportion of flexural tension reinforcement times the ratio $f_y/\sqrt{f'_c}$. It can be seen that there was a marked increase in the ductility one storey drift with the flexural strength of the beam rather than just the yield strength of the reinforcement. It can be seen that for units reinforced with both reinforcement grades the ductility one drift values are high. Working back from permissible drift limits in codes of practice would give ductility values considerably less than are commonly assumed in design [10]. Fenwick and Megget [10] recommended that for buildings less than 15 metres and over 30 metres high the maximum interstorey drift limits should be 1.6 and 1.2%, respectively when Grade 500E reinforcing was used. For the external beam-column joint units tested in this project those limits would relate to a structural ductility factor of less than 2, that is less than a third of the assumed design ductility factor.

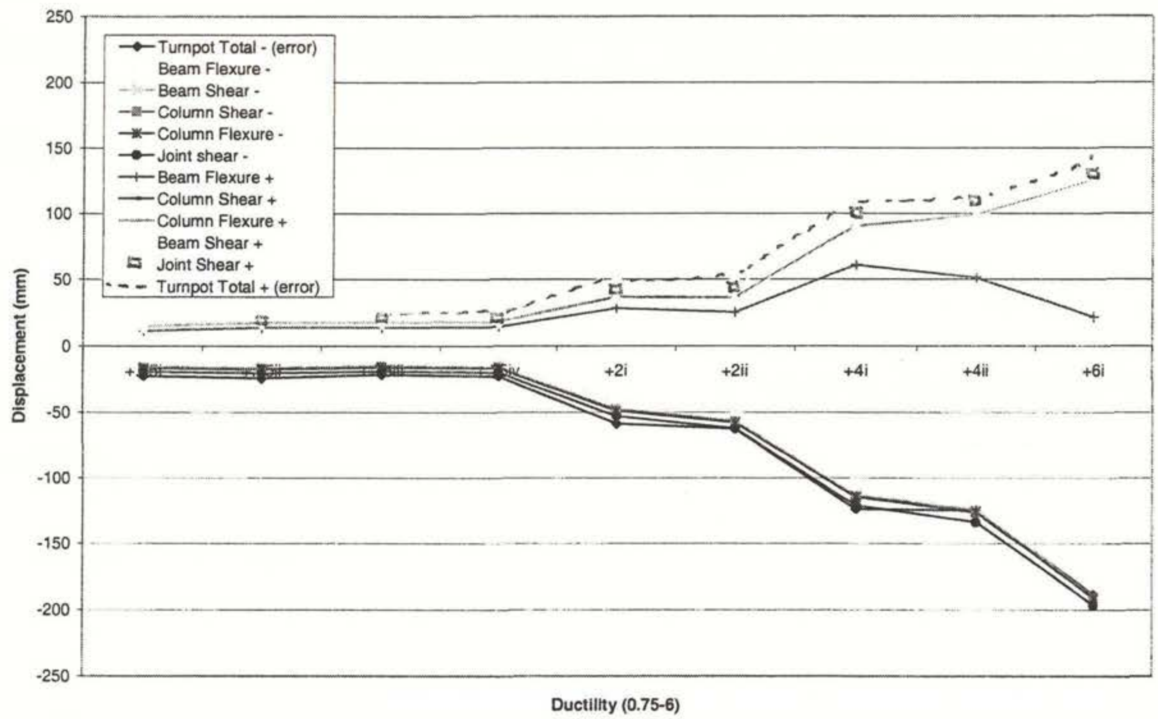

Figure 31. Flexural and shear deflection components for Unit 8 at cycle peaks.

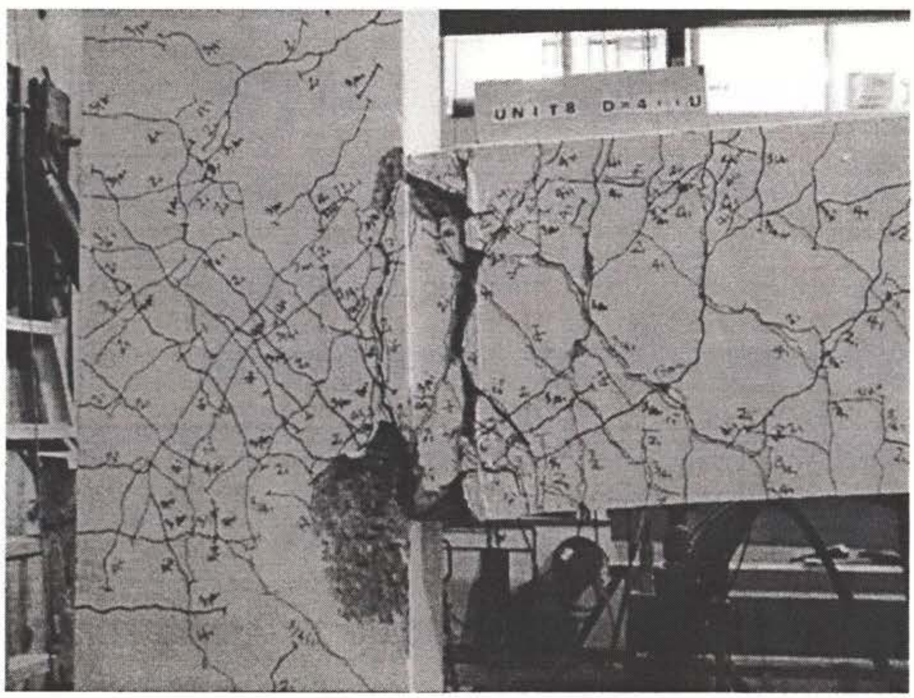

Figure 32. Unit 8 at the completion of the second ductility 4 cycle.

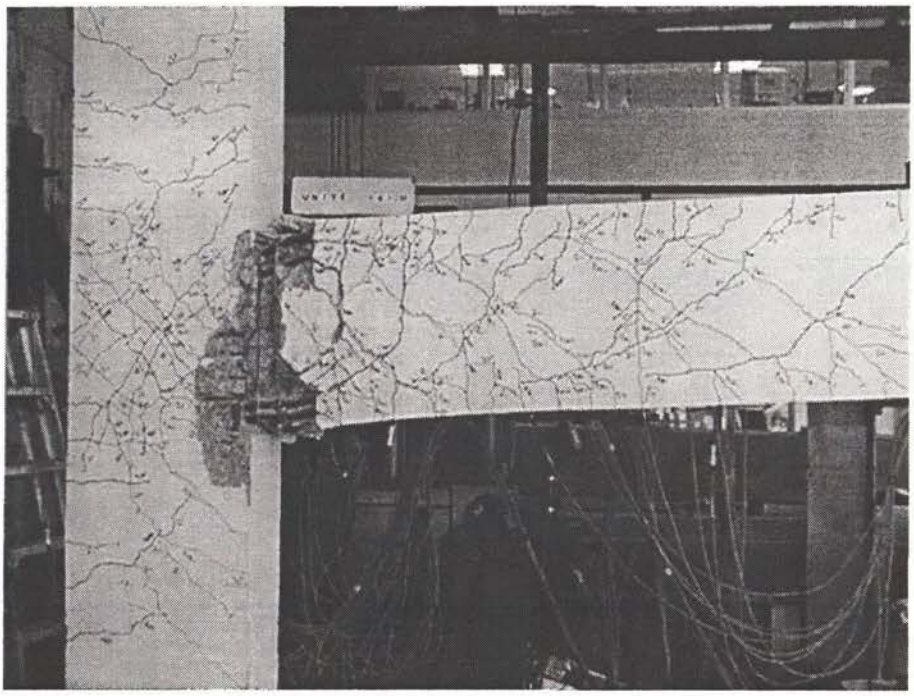

Figure 33. Unit 8 at the end of testing (completion of the only ductility 6 cycle).

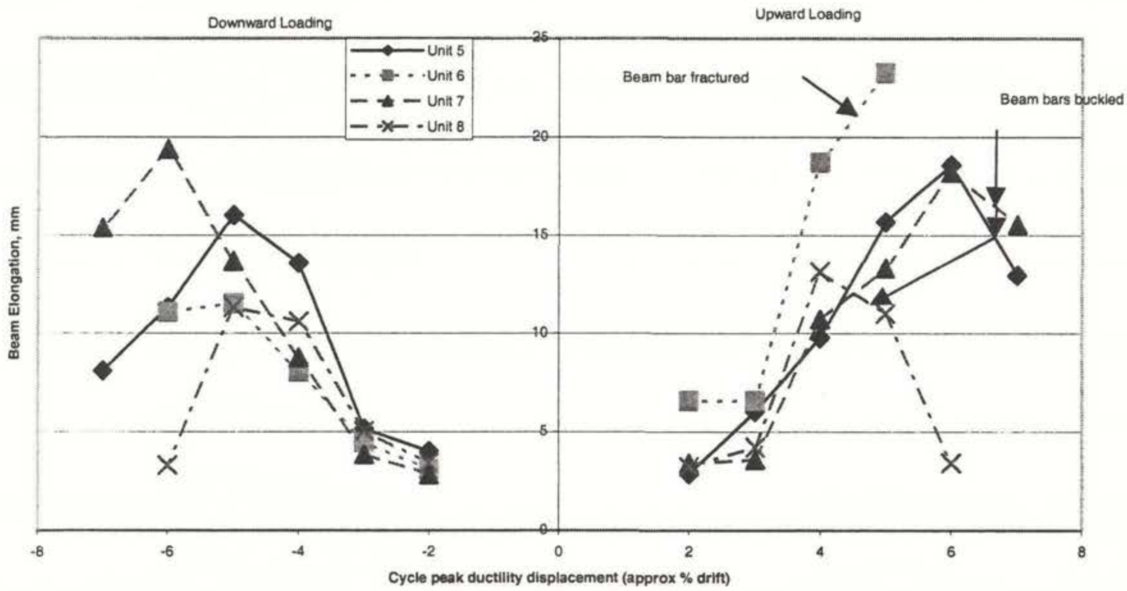

Figure 34. Measured beam elongation in Units 5-8.

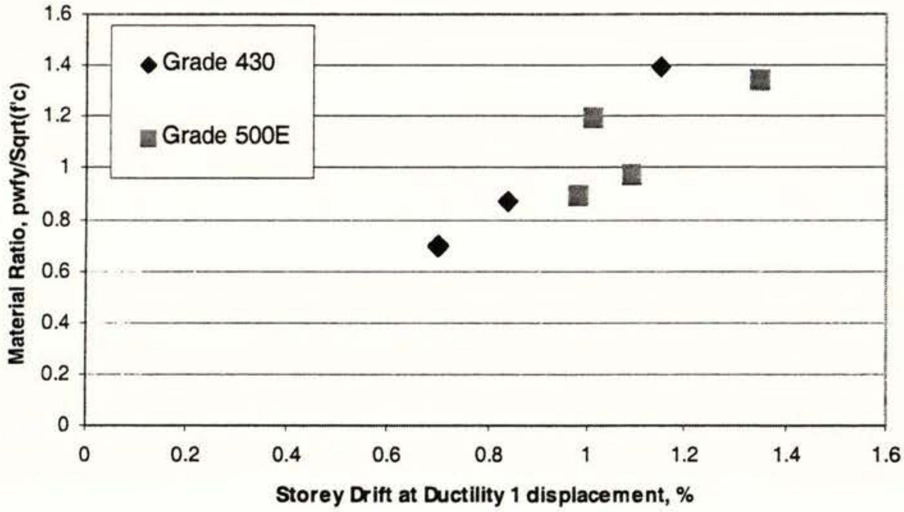

Figure 35. Ductility 1 storey drift verses Material Ratio, $p_w(f_y/\sqrt{f'_c})$.

Conclusions:

1 Anchoring the beam bars in external beam column joints short of the outside column bars locally reduces the flexural strength of the column at the face of the joint zone when the inside bars are subjected to tension. In Unit 1, where the bottom beam bars were anchored at a depth of $^{3}/_{4}$ of the depth of the column from the inside face, (as permitted by the NZ Concrete Standard 3101) the reduction in flexural strength resulted in extensive yielding of the inside bars. The cracking associated with this yielding led to the formation of wide cracks along the bottom beam bars in the joint zone and this reduced bond resistance and adversely affected the performance of the joint zone.

2 The adverse effects described in 1 above can be overcome by increasing the area of longitudinal column reinforcement placed on the inside face of the column. A simple expression is given for an enhanced design strength for the column to compensate for this loss in strength.

3 In all the 8 units tested the plastic hinge zone in the beam elongated by close to 3% of the beam depth at design ductility levels. In multi-story moment resisting frames this elongation induces a unidirectional plastic hinge in the external columns at the level of the first floor. The elongation of the beam did not adversely affect the performance of the joint zone in these tests. However, the same conclusion may not hold where a high axial load acts on the column.

4 Detailing the anchorage of the longitudinal beam reinforcement in the joint zone by bending the top bars in a U shape, so that they are continuous with the bottom bars, was found to slightly improve the performance compared with that obtained when the bars were individually anchored using standard 90-degree hooks.

5 The performance of the unit with the beam reinforcement distributed over the depth of the beam was similar to that obtained with the unit with conventional placing of beam bars. The main change was in the reduction of stiffness of the

beam associated with the distributed steel. This resulted in the unit having to sustain greater displacements to reach the nominated ductility values.

6 The joint zone ties did not appear to be over strained in these tests. This observation supports the reduction in the area of joint zone ties that is required in the 1995 edition of the concrete Standard [1] as compared to the 1982 edition [2].

7 The tests completed on joints with Grade 500E reinforcement in the beams behaved in a similar manner to the earlier test units with grade 430 steel. With the larger column bars on the interior face included ductile beam plastic hinges formed with only minor cracking of the joint zone. Due to the higher strength reinforcing the first yield drifts recorded were proportionally higher than the first 4 units with 430 Grade steel. At first yield drifts over 1% the potential ductility of these units was less than 2 before the interstorey drift limits specified in the Loadings Code [9] are exceeded.

8 Beam failure in the plastic hinge zones occurred with the Grade 500E reinforced units due to buckling of the beam bars (after substantial shear deformation), usually between lateral drifts of 4 and 6%.

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