

SEISMIC DESIGN AND BEHAVIOUR OF EXTERNAL REINFORCED CONCRETE BEAM-COLUMN JOINTS INCORPORATING 500E GRADE STEEL REINFORCING

Leslie M. Megget¹

SUMMARY

Four external reinforced concrete beam-column sub-assemblages were tested under pseudo-seismic cyclic loading. The approximately 2/3 scale units incorporated the new Grade 500E reinforcing steel as the beam bars. Two different forms of beam bar anchorage were tested, the normal 90-degree "standard hook" and the continuous U-bar detail. In all units the farthest point of the beam bar anchorage was positioned at the minimum limit prescribed in the NZ Concrete Standard (NZS3101), namely $\frac{3}{4}$ of the column depth from the inner column face. All 4 units formed plastic hinges in the beam and joint degradation was minor. Failure occurred at drift ratios between 4 and 6% (approximate ductility factors of between 4 and 6) predominantly due to buckling of the beam bars in the plastic hinge zone. The stiffness of these units was significantly less than similar units reinforced with 300E Grade reinforcing or the recently replaced 430 MPa reinforcement. The decreased stiffness will cause higher lateral drifts during large earthquakes, than those anticipated in current Standards.

INTRODUCTION

The 4 beam-column sub-assemblages (Units 5-8) described in this paper followed 4 previous units tested (Units 1-4) at Auckland University which incorporated the then current high strength reinforcing steel with a 5 percentile characteristic yield strength of 430 MPa, Megget and Fenwick [1]. All 8 sub-assemblies had 515 mm deep by 250 mm wide beams and 500 mm deep by 300 mm wide columns. The first 4 units contained different anchorage details for the beam reinforcing within the joint zones. Units 1 and 2 had conventional "standard hooks" with a 90-degree bend followed by a 12 bar diameter vertical tail, while unit 3 had continuous beam U-bars and Unit 4 had distributed beam steel down the section depth in the form of 7 horizontal U-bars. The outer point of the anchorage of the beam bars in the joint zone was positioned at $\frac{3}{4}$ of the column depth from the inner column face (known as the critical section). The joint shear ties and vertical shear intermediate column bars were designed to comply with the New Zealand Concrete Standard, NZS3101 [2] and all 4 units were tested to the often used New Zealand loading procedure of 2 cycles to $\frac{3}{4}$ of the beam's nominal moment, followed by displacement based cycles to displacement ductilities of ± 2 , 4, 6 and sometimes 8. These displacement ductility values equate almost exactly to interstorey drift percentages (2, 4, 6 and 8% respectively) for the test unit dimensions described here. Failure was assumed when the peak load sustained in any load cycle dropped to below 80% of the maximum load reached in the previous cycles.

In unit 1 a plastic hinge zone (PHZ) formed in the beam at ductility 2 but subsequently some plastic deformation occurred in the column immediately below the joint in the ductility 6 cycles. This was due to the loss of column

strength associated with the short anchorage length before the 90-degree bend, Megget and Fenwick [1]. There was also bond loss along the bottom beam bars in the joint due to yielding of the column reinforcement generating a wide crack along the bottom beam reinforcement in the joint zone.

Unit 2 had a jack positioned so it could apply a longitudinal force to the beam at its free end. During the test forces were applied to the jack so that the end of the beam was held in the same vertical plane. With this arrangement elongation of the beam is restrained by shear forces in the column as it is pushed out (Fenwick & Megget [3]). This resisting action occurs in the first floor beams of multistorey frames when the columns are essentially fixed into the foundations [3]. In this test the beam elongation pushed the column outwards and this caused extensive yielding of the column bars on the outer face. Strain gauge readings showed that beam bar yield penetration extended beyond the 90-degree bend in the anchorage length in Unit 1 but in Unit 2 this did not occur.

In Units 3 and 4 the column bars on the interior face were increased in size (from 4-D16 to 4-D20, D denotes deformed bar, 16 and 20 is the bar diameter in mm) to eliminate the column hinging. There was little joint cracking in Unit 3 and the beam PHZ continued to form in the ductility 4 cycles over a length exceeding 500 mm. In the subsequent cycles the beam shear deformations increased steadily until beam bar buckling occurred with a subsequent loss of strength.

Unit 4, with uniformly distributed beam bars, was less stiff than the previous units due to a reduction in section stiffness of about 15%, even though the nominal strength was nearly identical. A PHZ formed in the beam and failure occurred in the ductility 6 cycles due to excessive spalling over the hinge length and buckling of the smaller diameter D16H bars (all beam bars were D20H in units 1-3).

¹ Dept. of Civil & Environmental Engineering, University of Auckland, Private Bag 92-019, Auckland (Fellow).

TEST UNITS 5-8

The objective of the second group of 4 tests was to repeat some of the earlier tests using the newly introduced Grade 500E reinforcing bar (5% characteristic yield stress = 500 MPa). The NZ Concrete Standard [2] is very specific about how beam bars should be anchored in exterior beam-column joints when the beam plastic hinge is detailed to form at the column face. As well as the anchorage tail not being positioned within the $\frac{3}{4}$ column depth from the inner column face, the Standard requires that the bars' development length must be measured from a point, the lesser of 8 bar diameters (160 mm) in from the column face or half the column depth, to allow for yield penetration into the joint. The bar development length (L_{dh}) was calculated to be 303 mm for 500E bars and concrete with a 28 day compressive strength of 40 MPa. The length $8d_b + L_{dh} + h_c/4$ (160+303+125 mm), which is equal to 588 mm, was greater than the column depth of 500 mm. However, the Standard allows the L_{dh} length to be decreased by 20% when 2 extra transverse bars are positioned in each 90-degree bend. This also allows a better joint diagonal strut to form (Megget [4]). The extra D20H transverse bars were used in Units 5-8 to enable the column dimension to be maintained at 500 mm. Table 1 details the main beam and column bars and shear ties in units 5-8, as well as the actual reinforcing and concrete strengths at the time of testing, found for each unit. Figure 1 shows the reinforcing details of the units 5-8.

Unit 5 required five R8 tie-sets (6 legs/set) within the joint between the top and bottom beam bars. The current Standard allows short tie legs to be included in the horizontal joint tie area, A_{jh} , if they are longer than a third of the column depth in the shear direction being considered. The actual length of the short tie legs was about $0.5h_c$ in these joints. Therefore the A_{jh} provided was 1,507 mm², while the required horizontal joint tie area was 1,150 mm², about 31% overdesigned. A strain-hardening factor of 1.15 was incorporated into the calculations of the horizontal joint shear force instead of the accepted overstrength factor of 1.40 for the yield strength of Grade 500E reinforcing. The lower factor was used because the actual steel yield stress was included in the calculations. Since these tests were completed the new Draft Concrete Standard (DZ 3101.1 [5]) has been published for public comment. This Draft incorporates minor changes only to the current 1995 Concrete Standard, with respect to beam-column joints. One of the changes specifies that the short horizontal tie legs in joints should be excluded from A_{jh} if the leg does not pass through the "potential diagonal failure plane". In Unit 5 only the middle tie-set's short legs passed through the diagonal line between the joint's extreme corners. Thus the provided amount of horizontal joint ties was 1,105 mm² if only the middle tie-set's short legs are included. Table 2 gives the A_{jh} areas required for NZS 3101:1995 and the Draft Standard and the ratios of area provided divided by the 1995 Standard and 2004 Draft requirements respectively for Units 5-8. The required tie areas were calculated using the actual material

strengths found from testing, not the lower characteristic strengths, as would normally be done in design. In all 4 units the provided A_{jh} areas included the short legs for the 1995 Standard but excluded the legs which didn't pass through the joint's diagonal section for the 2004 Draft Standard calculations.

Unit 6 had unequal top and bottom beam reinforcement as few beam-column joints have been tested with this arrangement, and the current Standard's joint shear tie requirements is largely dictated by the *smaller* amount of beam steel [2]. If the short tie-legs around the intermediate column bars are included in the horizontal joint shear calculation then this joint was over designed by about 65%, but if the non-complying short legs were neglected the over design was about 47%. Two of the 5 tie-sets were excluded in the provided area calculation because in Unit 6 the ties were spaced further apart and the outermost sets were actually touching the top and bottom beam bars. The draft Standard [5] clarifies the situation where the outer tie-sets are close to either the top or bottom beam bars by recommending that they only be included in the joint shear area when "the space between the innermost layer of beam bars and the adjacent set of ties should preferably not be less than one half of the vertical spacing between sets of joint ties". The 1995 Standard [2] provided no guide how "close" was too close and it was left to the designer to guess.

In Unit 7 the joint had only 4 sets of R8 ties although the 1995 Standard required 5. This was to check any over conservatism in the Concrete Standard as it relates to joint shear ties. Lin & Restrepo [6] recommended that the NZ Standard requirements for low axially loaded beam-column joints could be reduced significantly. The area of ties provided represents about 65% of the horizontal joint tie area required if the 2 short tie legs/set are ignored. Note that the tie area provided is only 81% of that required for the Draft Standard with 50% of the short tie legs excluded.

Unit 8 was designed to have close to the maximum amount of beam steel allowed by the Standard ($p=1.67\%$ for $f_y=500$ MPa and $f'_c=40$ MPa), which would also approach the Standard's nominal joint shear stress limit of $0.2f'_c$. This resulted in 5-D20H bars top and bottom in 2 rows, U-bars being incorporated here to reduce the amount of congestion within the joint zone. 6 sets of R10 ties (3 ties + a supplementary cross-tie/set) were provided in the joint zone but only 4 of the ties counted towards the transverse horizontal joint ties, A_{sh} due to the other 2 sets touching the beam bars. The tie-sets touching the beam bars will contribute a proportion of their strength to resist the horizontal joint shear force but the Concrete Standard assumes no contribution. Assuming the area of the short legs was included then the 4 tie-sets provided about 95% of the Standard's [2] requirements but 79% of the Draft Standard's requirement with the non-complying short legs excluded.

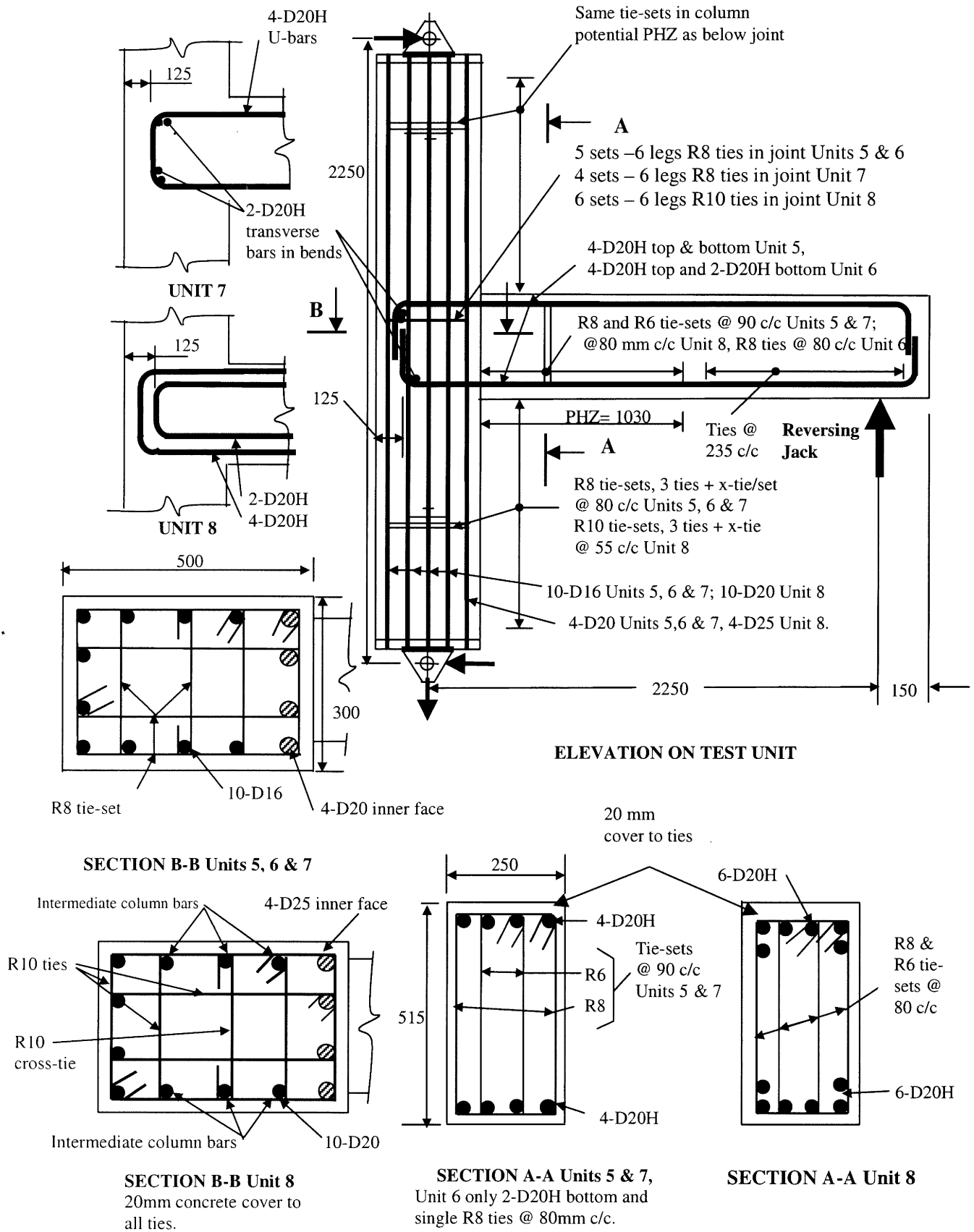


Figure 1. Details of test units 5-8.

Table 1. Units 5-8 reinforcing details and concrete strengths.

Unit	Beam main Reinforcing	Beam bars Yield Stress (MPa)	Beam ties in PHZ	Column main reinforcing	Column shear ties in potential PHZ	Joint Horizontal Shear ties (f_{yh} , MPa)	Concrete strength after testing (MPa)
5	4-D20H Top, 4-D20H Btm. L-bars	572	1-R6 + 1-R8 ties @ 90 mm c/c	10-D16 and 4-D20 on inner face	R8 tie-sets @ 80 mm c/c (3 ties + supp cross-tie/set)	5-R8 tie-sets @ 95 mm c/c (386)	37.9
6	4-D20H Top, 2-D20H Btm. L-bars	542	R8 ties @ 80 mm c/c	10-D16 and 4-D20 on inner face	R 8 tie-seats @ 80 mm c/c (3 ties + supp cross-tie/set)	5-R8 tie-sets @ 95 mm c/c (380)	31.4
7	4-D20H U-bars	543	R6 & R8 tie-sets @ 90 mm c/c	10-D16 and 4-D20 on inner face	R8 tie-sets @ 80 mm c/c (3 ties + supp cross-tie/set)	4-R8 tie-sets @ 125 mm c/c (341)	40.7
8	6-D20H U-bars	541	R6 & R8 tie-sets @ 80 mm c/c	10-D20 and 4-D25 on inner face	R10 tie-sets @ 55 mm c/c (3 ties + supp cross-tie/set)	6-R10 tie-sets @ 60 mm c/c (354)	42.5

The vertical joint shear requirements (A_{jv}) in the 2004 Draft Standard [5] are identical to those in the current 1995 Standard [2]. Table 3 shows the required A_{jv} requirements for both Standards and the ratios of actual intermediate column bars provided to the required amounts of vertical joint steel, again using the actual reinforcing and concrete strengths

from testing. It can be seen that Unit 6 was provided with considerably more area of intermediate column bars than required, but the other 3 units were marginally under designed. Fuller details of the sub-assembly design, detailing and testing can be found in Barton [6].

TABLE 2. Areas of horizontal joint ties required by NZS3101:1995 and 2004 Draft Standard and tie areas provided in the test units.

UNIT	A_{jh} from NZS 3101:1995 and DZ3101:2004 (mm^2)	1995 $\underline{A_{jh, provided}}$ A_{jh} required	2004 $\underline{A_{jh, provided}}$ A_{jh} required
5	1,150	1.31	0.96
6	547	1.65 *	1.47 *
7	1,235	0.98	0.81
8	1,989	0.95 *	0.79 *

* 2 tie-sets ignored in the provided area due to the top and bottom joint ties touching the beam bars.

TABLE 3. Areas of vertical joint shear reinforcement required from NZS3101: 1995 and the 2004 Draft Standard.

Unit	A_{jv} 1995 3101 Standard and DZ3101 2004 (mm^2)	A_{jv} provided (mm^2)	1995 & 2004 A_{jv} provided / A_{jv} required
5	1,018	1,206	1.18
6	470	1,206	2.56
7	965	1,206	1.25
8	1,825	1,885	1.03

TEST SETUP

The units were cast on their sides and lifted into prototypical position. The column reaction at the top was a pin so that the vertical reaction of the applied beam load was carried by the bottom reaction only. Thus the bottom column and joint segments experienced small axial tension and compression alternatively during every loading cycle. The intended concrete compressive strength for Units 5-8 at the time of testing was 40 MPa.

To measure displacements in the test unit 6 mm studs were welded to the main reinforcing and separated from the cover concrete, and portal displacement gauges were fixed to these. From the electronic data collected from the gauges it was possible to calculate the flexural, shear and elongation displacements for segments of the beam, column and joint zone. These deformations were summed and checked against the beam-tip displacement measured directly. The Grade 500E bars were not preheated before welding of the studs as recommended by manufacturer and this led to problems in unit 6. Unit 8 had extra studs welded to an outer column bar to ascertain the strain profile through the joint.

UNIT 5 RESULTS

During the ductility 2 cycles (2.2% drift but actually greater than 3% due to slack in the testing frame) the beam yielded and formed a plastic hinge in the beam adjacent to the column face. The peak moment reached approximately 1.2 times the beam's nominal moment strength (M_n) in both directions. Figure 2 shows the column shear versus drift

response for unit 5. Good, stable hysteretic loops formed until the second cycle to ductility ± 6 where there was a greater than 20% strength reduction. Figure 3 shows Unit 5 at the completion of the 4.4% drift cycles. Concrete cover was lost over a length of 125 mm adjacent to the column face and severe beam bar buckling was observed in the 6% drift cycles, as shown in Figure 4. The joint region remained virtually undamaged during the test with only minor diagonal cracking. There was no apparent beam bar slip as had been seen in unit 1 and no yielding of the column reinforcement.

UNIT 6 RESULTS

This unit had unequal areas of top and bottom beam reinforcing and suffered a premature failure in the second upward cycle to ductility 4 when one of the 2 bottom beam bars fractured. The failure occurred at a position where a stud had been welded to a bar. It was assumed that this occurred due to the welding and the lack of preheating. Testing concluded in the subsequent downward half cycle to ductility 6. Bar buckling was apparent where the bar fractured. The column shear-drift ratio hysteretic loops are shown in Figure 5. A beam plastic hinge formed during the ductility 2 cycles (2% drift) and the strengths recorded exceeded the nominal strength by about 5% and 22% in the downward and upward directions, respectively. Beam shear deformations increased relative to the flexural and joint shear deformations during the ductility 2 cycles prior to bar fracture. Again there was only minor cracking in the joint region and no bar slip was observed as can be seen in Figure 6 taken at the completion of the 4% drift cycles.

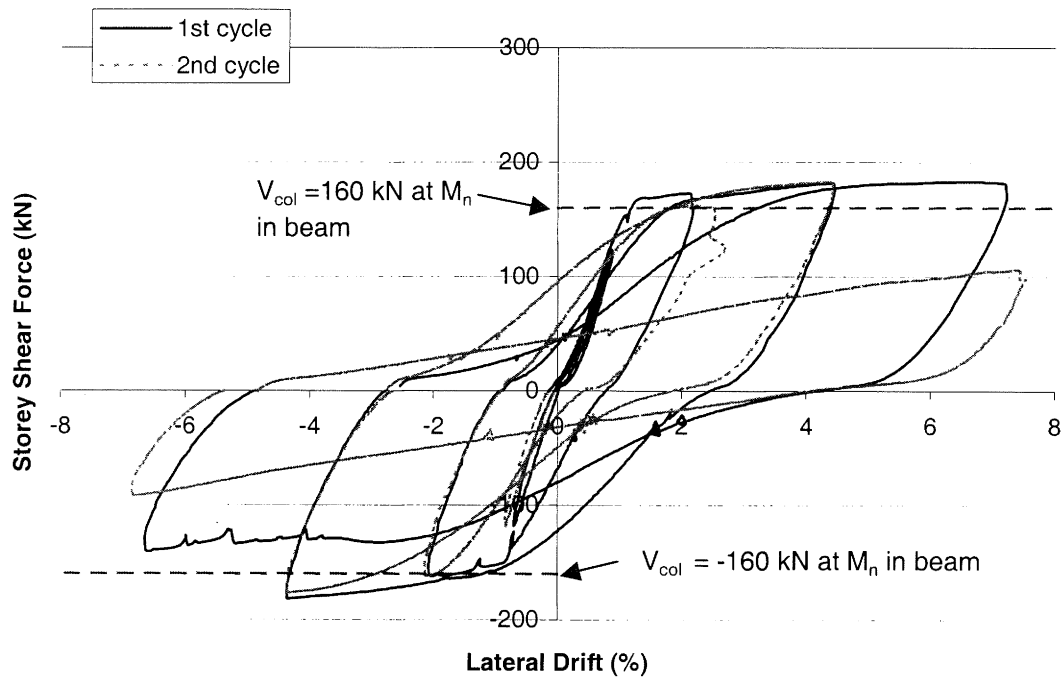


Figure 2. Unit 5 Column Shear - % Storey Drift Ratio. hysteresis loops.

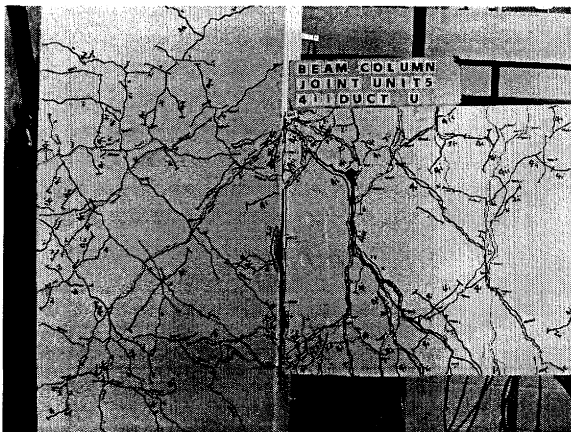


Figure 3. Unit 5 after 4% drift cycles.



Figure 4. Unit 5; Bar buckling during 6% drift cycles.

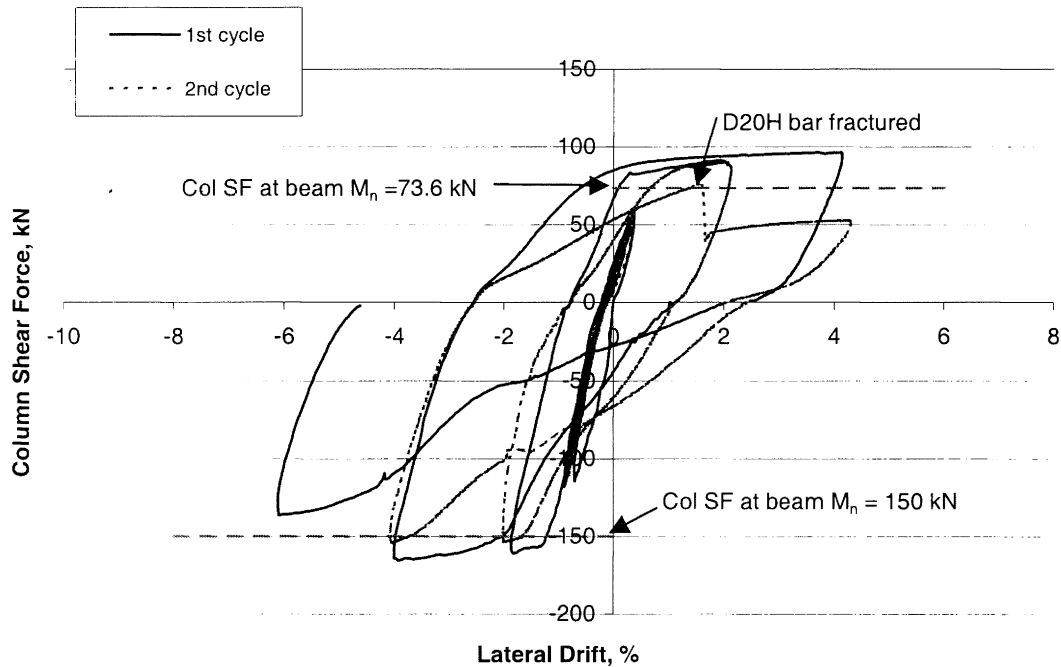


Figure 5. Unit 6: Story Shear (kN) – % Storey Drift Ratio hysteresis loops.

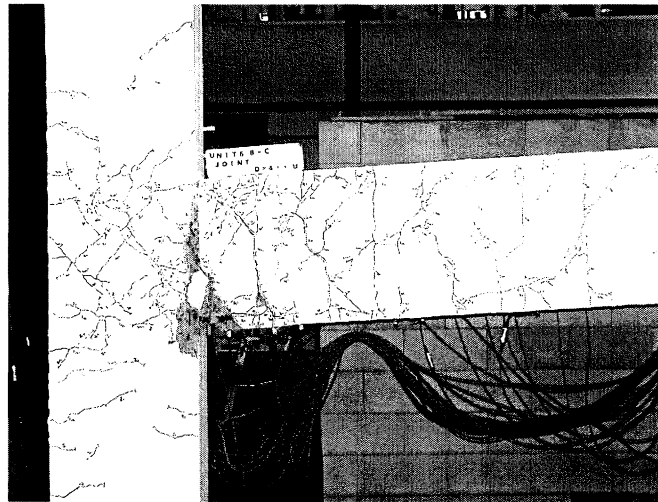


Figure 6. Unit 6 at completion of the 4% drift cycles.

UNIT 7 RESULTS

This unit was identical to unit 5 except for the inclusion of U-bars instead of standard hooks and the use of 4 horizontal tie-sets instead of 5 within the joint zone. The excellent hysteretic behaviour, which exceeded that of the previous sub-assemblies, occurred (see Figure 7) up to and including the first \pm ductility 6 cycles (6% drift), in which the maximum strength reached $1.21M_n$ at ductility 6 downwards. In the second cycle to ductility 6 there was a decrease in strength to about 85% of maximum reached in the previous upward cycle, with the corresponding drop to 78% of the strength reached in downward ductility 6 cycle. The unit's

condition at the completion of the 6% drift cycles is shown in Figure 8.

The beam shear deformation contribution to the drift increased from about 13% at the end of the 2% drift cycles to approximately 27% in the second 4% drift cycle and it doubled again in the second cycle to 6% drift. This beam shear deformation increase is shown on Figure 9 where the contributions from beam, joint and column flexure and shear to the total beam deflection are shown at each cycle peak during testing. Beam bar buckling occurred in the plastic hinge zone during the 6% drift cycles.

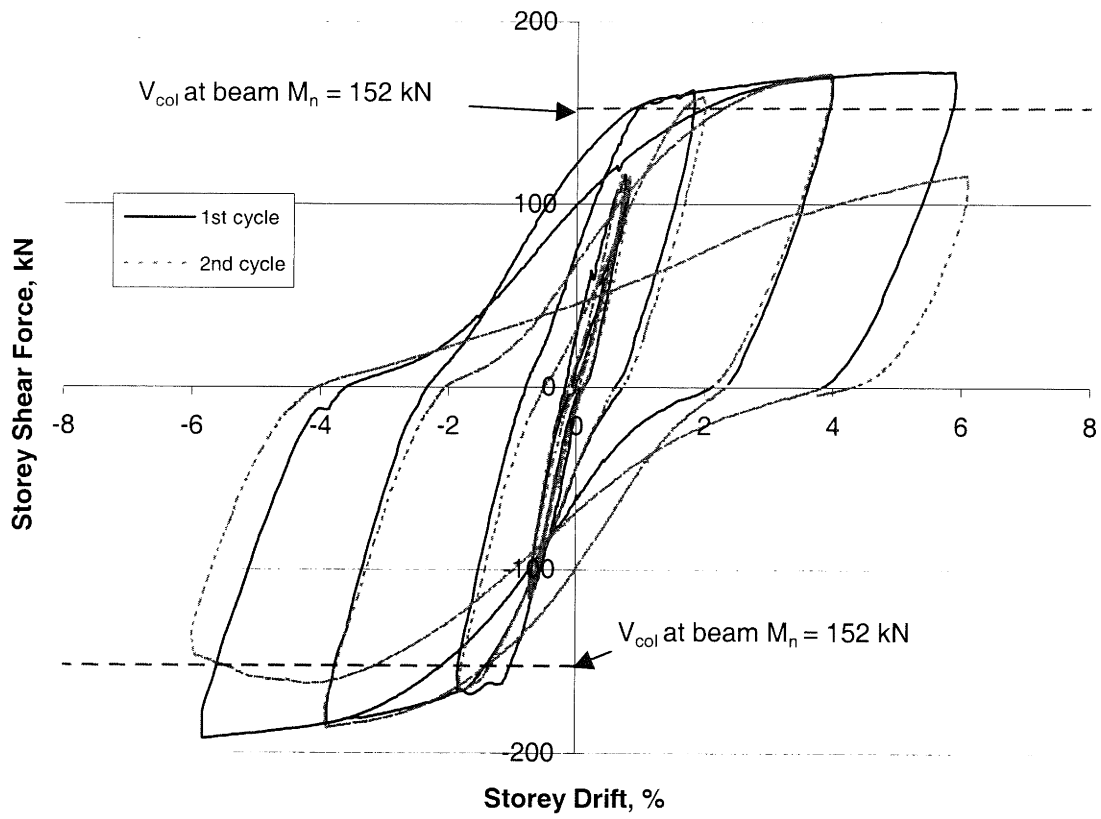


Figure 7. Unit 7 Storey Shear - Storey Drift (%)

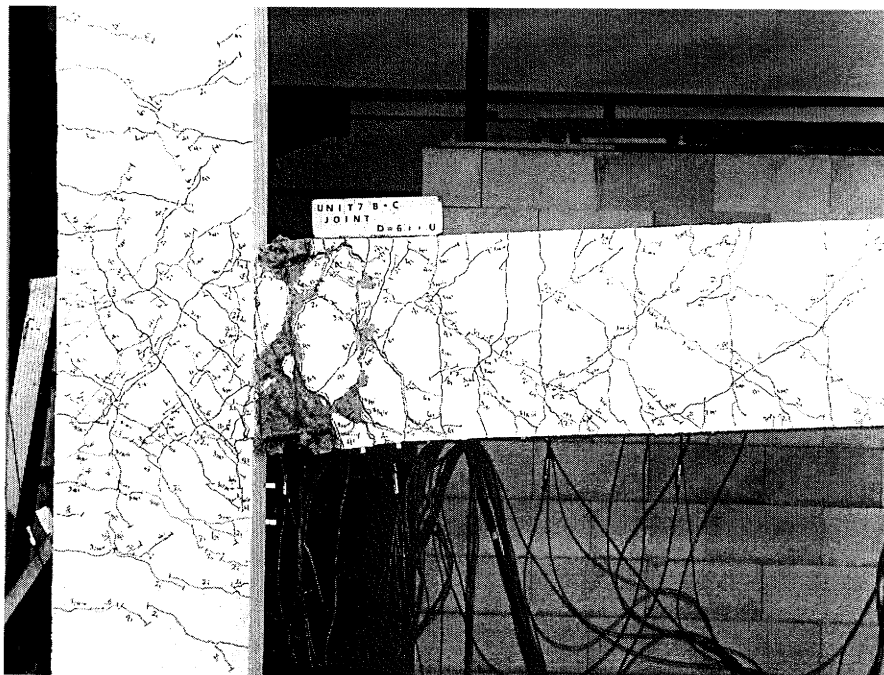


Figure 8. Unit 7 after completion of the 6% drift cycles.

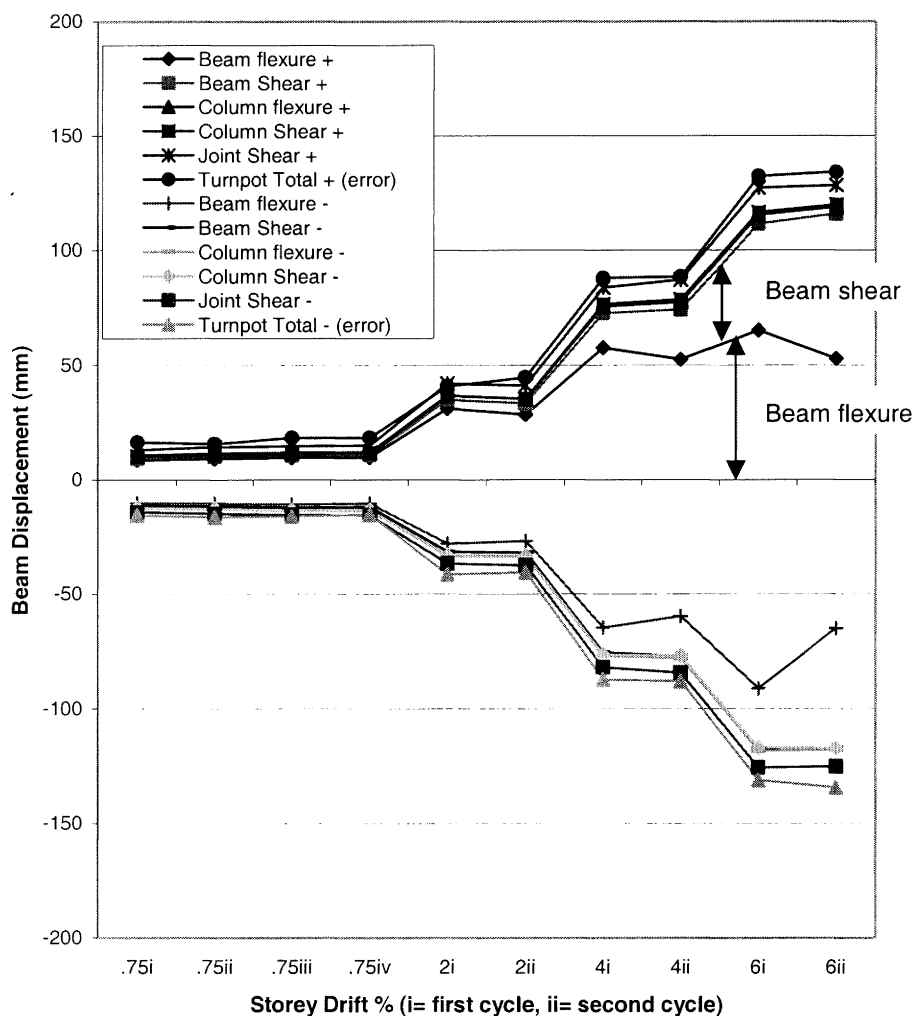


Figure 9. Unit 7 Beam components of displacement during testing.

There was only minor cracking in the joint zone but there were a greater number of fine cracks when compared to Unit 6. During the ductility 4 cycles the diagonal joint cracks were less than 0.2 mm wide except for the main diagonal crack, which was 1.5 mm and 0.9 mm wide during downward and upward loading, respectively. Again there was no apparent slip of the beam bars in the joint and there was no change in the joint cracking pattern between the 4 and 6% drift cycles. The reduction in the amount of recommended joint zone ties had no detrimental effect on the unit's behaviour, thus indicating that a reduction in the horizontal joint shear requirements may be possible for low axially loaded columns.

UNIT 8 RESULTS

This beam had a flexural tension reinforcement proportion of 1.63% with 6-D20H bars top and bottom in the form of U-bars. The maximum permissible [2] reinforcement ratio is 1.67%. The beam's nominal flexural strength (431 kNm)

was exceeded in both directions during the ductility 2 cycles and a plastic hinge formed in the beam adjacent to the column face. The ductility 1 drift corresponded to a drift of 1.35%, which based on a permissible value of 2.5% [8] would limit the structural ductility factor to about 2.

Good energy absorbing column shear-drift loops were produced for the ductility 2 and 4 cycles with a maximum flexural strength of $1.10M_n$ being reached at an inter-storey drift of 5.4%, see Figure 10. During the first ductility 2 cycle the test had to be stopped when 2 bolts sheared in the column base plate connection (first positive moment loop). The pivot bracket was subsequently welded to the column base plate and the test continued as planned. In the second negative half cycle to ductility 4 the load decreased by 14% from the first cycle, while the corresponding decrease was 11% in the upward direction. In this unit the decrease in strength occurred earlier than in the other units. In addition there was a greater proportion of deformation occurring due to shear deformation in the beams in the ductility cycles to 4 and 6

than in the other units. The component of beam shear deformation was greater than that due to beam flexure (56% compared with 41%) as shown in Figure 11. Extensive yielding of the stirrups within 200 mm of the column face led

to disintegration of the beam core concrete and buckling of the beam reinforcement, see Figure 12.

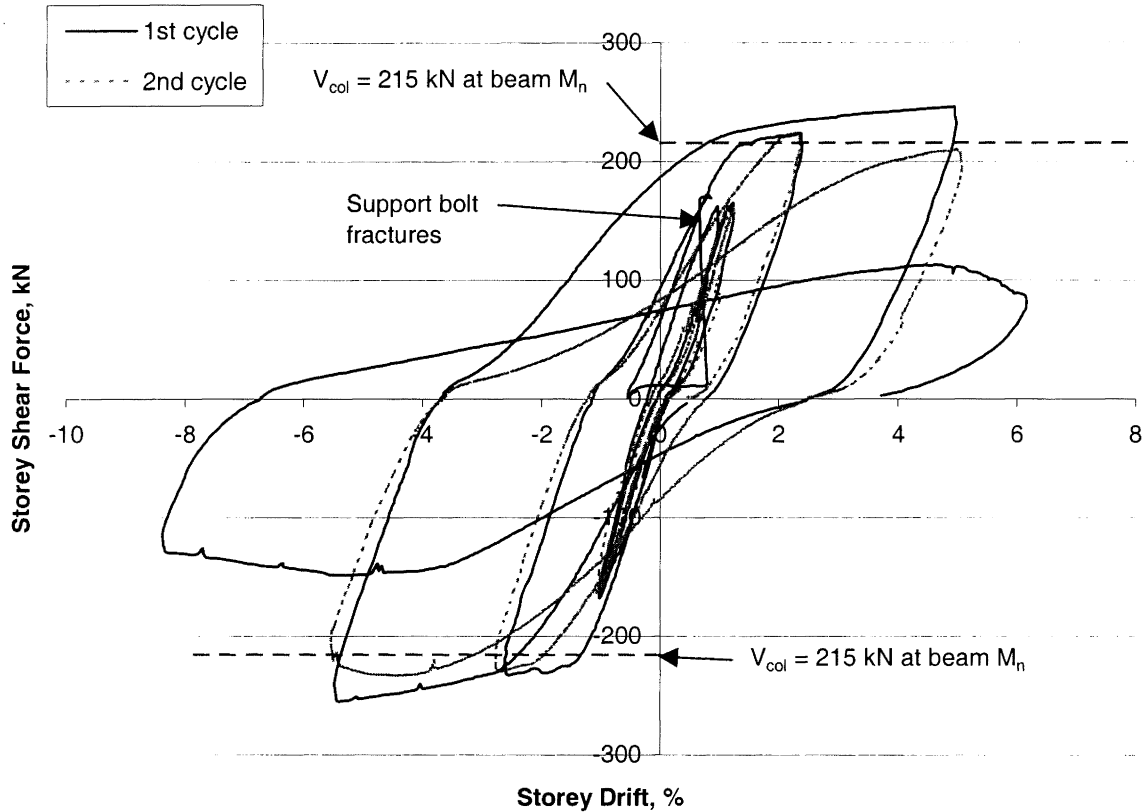


Figure 10. Unit 8 Storey Shear (kN)- Storey Drift (%)

BEAM ELONGATION

The beam elongations were calculated from the portal gauge readings on the beam bars over the 4 beam segments instrumented on each unit. Figure 13 plots the elongations at each cycle peak (storey drift), the downward loading cycles being separated from the upward ones. Note that the first and second cycles to the same drift are separated by 0.5% drift on the plot for clarity, whereas in reality both cycle peaks had very similar drifts. The maximum elongations range from about 14 to 19.4 mm (2.7 to 3.8% of the beam depth), whereas in units 1 to 4 the range was from 8 to 16.5 mm (Megget and Fenwick [1]). This increase is due mainly to the

lack of anchorage loss (bar slip) in the joint region when compared to units 1 and 2, which had substantial bar slip in the joint, thus reducing elongation. The extreme measured elongation of 23 mm in Unit 6 is not representative as it occurred after the bottom reinforcing bar had fractured.

The sudden drop in elongation recorded in units 5 and 7 was associated with the bar buckling that occurred usually in the second 6% drift cycles. The elongation in Unit 8 was considerably smaller at equivalent drifts than the other 3 units, but it must be remembered that Unit 8 with 5-D25H bars top and bottom first yielded at a drift of about 1.35% whereas units 5-7 yielded at about 1% drift.

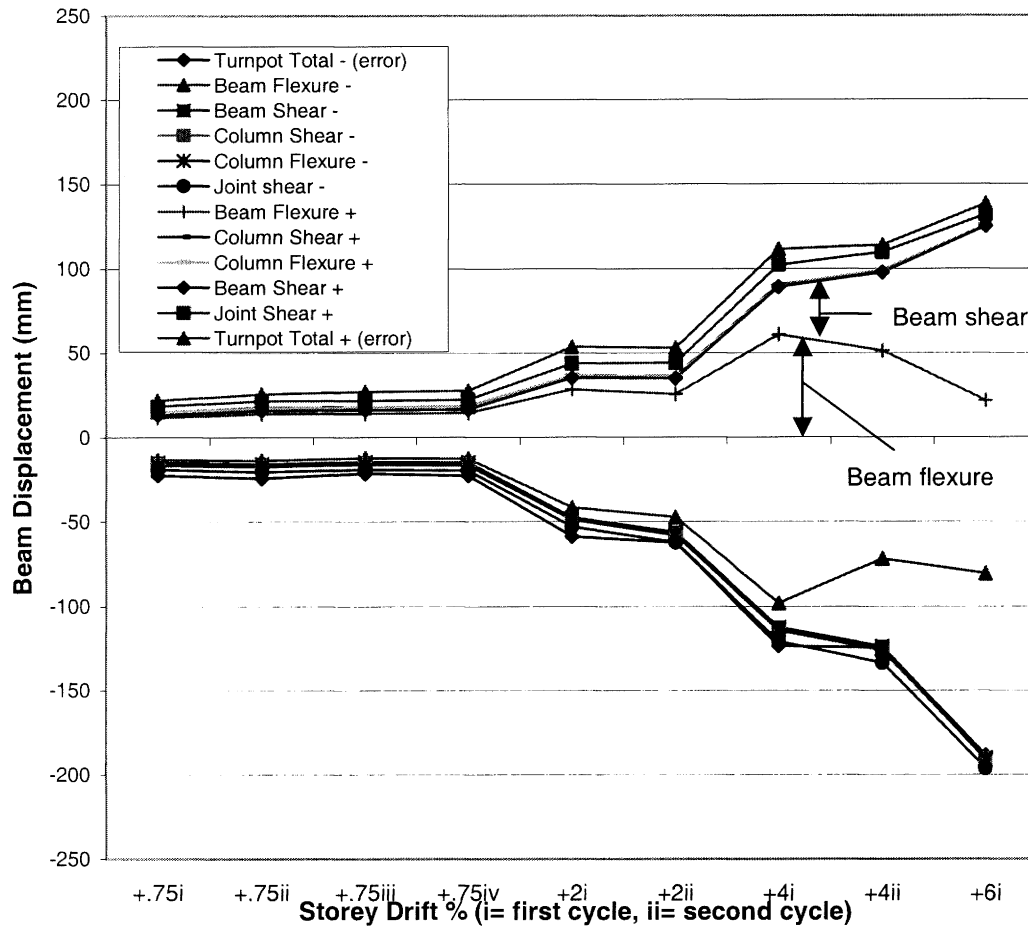


Figure 11. Unit 8 contributions to beam deflection during testing.

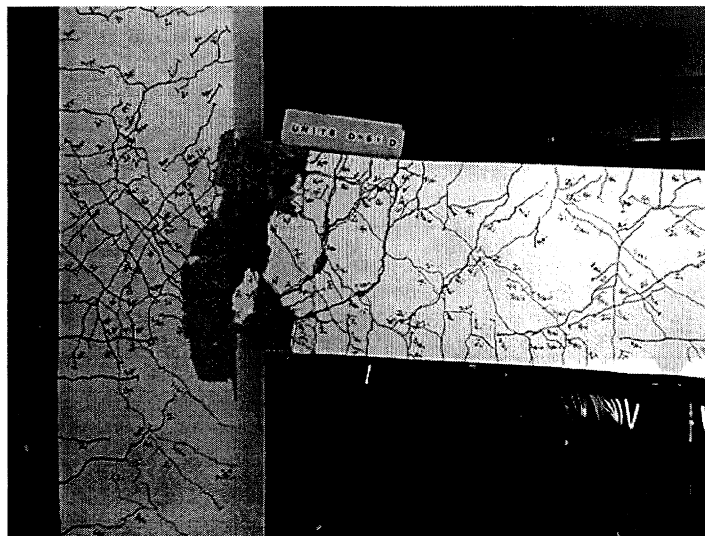


Figure 12. Unit 8 during the 7.5% drift cycles. Note shear deformations in PHZ.

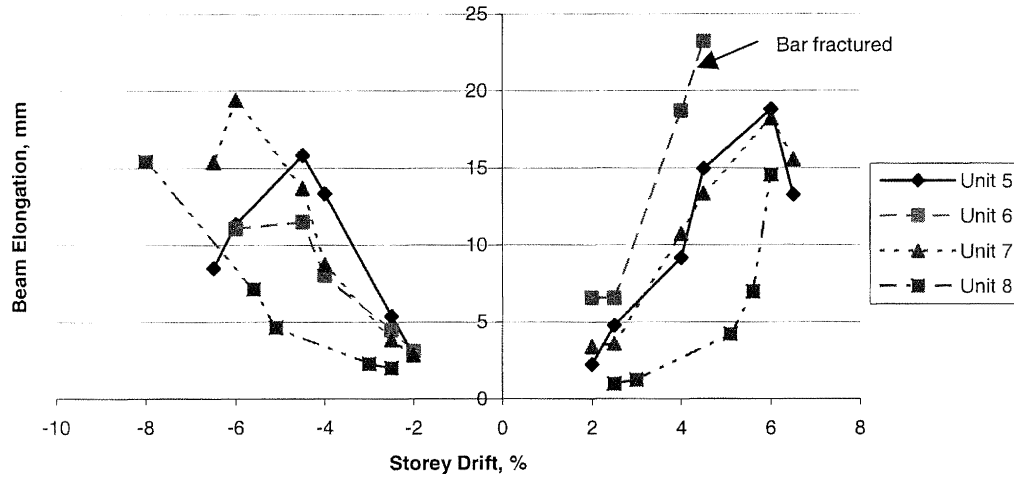


Figure 13. Beam elongation at cycle peaks for Units 5-8.

Table 4. Ductility One Storey Drift of all test units.

Unit	Ductility One Storey Drift (%)	Beam Reinforcement Ratio (%)	Beam yield stress, f_y (MPa)	Concrete Compressive Strength, f'_c (MPa)
1	0.70	1.06	452	46.7
2	0.70	1.06	452	48.0
3	0.84	1.05	482	33.9
4	1.15	1.74	465	33.9
5	1.09	1.05	572	37.9
6	1.01	1.05 top 0.53 bottom	542	31.4
7	0.98	1.05	543	40.7
8	1.35	1.62	541	42.5

BEAM STIFFNESS

Table 4 gives the ductility one displacements of the 8 external joints tested, together with their beam reinforcing ratio, the reinforcing yield strength and the concrete compressive strength at testing.

Figure 14 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 is 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 either reinforcement grade the ductility one drifts are high. Working back from permissible drift limits in Standards of practice would give ductility values considerably less than are commonly assumed in design [8].

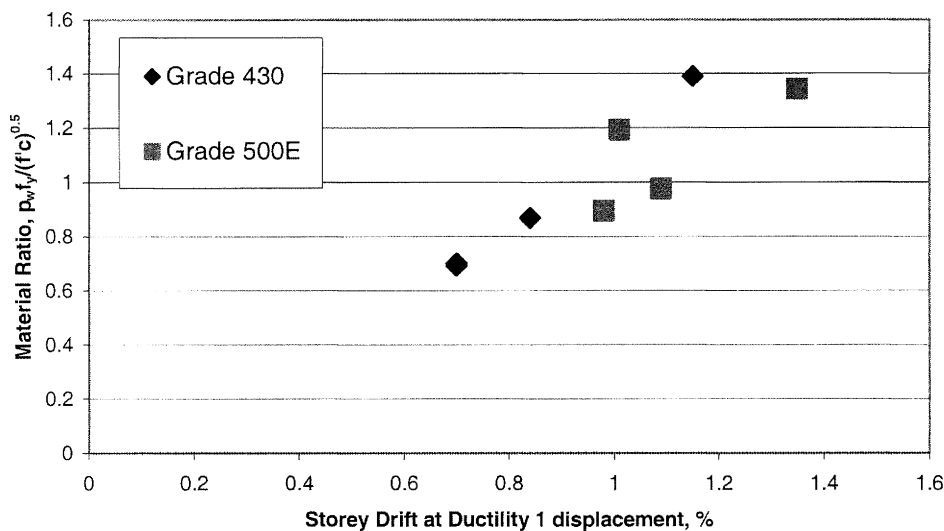


Figure 14. Ductility 1 Storey Drift versus Material Ratio, $p_w (f_y/\sqrt{f'_c})$

CONCLUSIONS

- Anchoring the beam bars in external beam-column joints short of the outside column bars was found to locally reduce the flexural strength of the column at the face of the joint zone when the inside bars are subjected to tension. In Unit 1, in the first series of tests, this led to appreciable yielding of the column bars. In Units 5-8 described in this paper the 4 inner column bars were increased by one bar size to compensate for this loss of strength. This additional reinforcement gave a theoretical increase in strength of about 10% when the inner column bars were in tension, and this was sufficient to prevent column bars from yielding in these units.
- All the sub-assemblies reinforced with Grade 500E reinforcing showed no anchorage loss or shear-degradation in the joint zone for cyclic displacements up to 6% drift. Unit 7 had only about 65% of the New Zealand Concrete Standard's (NZS 3101:1995) [2] requirements for horizontal joint ties (assuming the short tie legs are ignored) but still showed no joint distress. The minor reduction in joint shear requirements due to ignoring the short-legged ties which do not cross the main joint diagonal crack, as specified in the 2004 Draft Concrete Standard [5], seems appropriate.
- The mode of failure in the units incorporating 500 MPa reinforcing was buckling of the beam reinforcement, usually during the 6% drift cycles. Although the drifts when buckling occurred were higher than the drift limits in the NZ Loadings Standard [7] buckling appears to occur at lower drift ratios when the beams incorporate Grade 500E reinforcing. This "global" type of buckling occurred between 3 or 4 tie sets after substantial beam shear deformation had developed in the plastic hinge zone.
- Maximum beam elongations between 2.7 and 3.8% of the beam depth were measured in all the units tested with 500E Grade beam reinforcing, about 35% greater than those measured for the same sized beams with Grade 430 reinforcing at the same level of lateral drift. This increase in elongation may further disrupt the precast slab/beam continuity assumed in multistorey ductile frame performance in major earthquakes.
- In the units tested there seemed to be little difference in performance between the joints with continuous U-bar anchorage and the more conventional "standard hook"; 90-degree bend + $12d_b$ tail anchorage. The U-bar detail has a major advantage as it reduces the complexity of reinforcing in the joint zone, allowing easier placement and compaction of the concrete. The added transverse bars within the 90-degree bends to allow a reduction in the development length appear to work well as no beam bar slip was apparent.
- The inter-storey drift corresponding to first yield of the beam bars was found to increase with the quantity of reinforcement in the beam. The magnitude of these inter-storey drifts indicates that only low displacement ductility values should be assumed in design where high strength main reinforcement is used.

ACKNOWLEDGEMENTS

The testing described here was completed in the University of Auckland, Department of Civil & Environmental Engineering Test Hall and was sponsored by the NZ Earthquake Commission.

REFERENCES

1. Megget, L. and Fenwick, R. (2003). "Seismic performance of external reinforced concrete beam-column joints." *Bulletin of the NZ Society for Earthquake Engineering*; **36**(4): 223-232.
2. Standards New Zealand, (1995). "Concrete Structures Standard, Part 1- The Design of Concrete Structures." NZS 3101:1995.
3. Fenwick, R.C. and Megget, L.M. (1993). "Elongation and load deflection characteristics of reinforced concrete members containing plastic hinges." *Bulletin NZSEE*; **26**(1): 28-41.
4. Megget, L.M. (2003). "The seismic design and performance of reinforced concrete beam-column knee joints in buildings." *Earthquake Spectra*; **19**(4): 863-895.
5. Standards New Zealand, (2004). "Concrete Structures Standard," DZ 3101.1 Public Comment Draft 15 September 2004, 274p.
6. Barton, Meg B. (2003). "Seismic behaviour of external reinforced beam-column joints using Grade 500E steel", M.E. thesis, *The University of Auckland, Dept. of Civil & Environmental Engineering*, July, 226p.
7. Lin, C-M. and Restrepo, J.J. (2002). "Seismic behaviour and design of reinforced concrete interior beam-column joints." *Bulletin NZSEE*; **35**(2) 108-128.
8. Standards New Zealand, (1992). "General structural design and design loadings for buildings." NZS 4203:1992.
9. Fenwick, R. and Megget, L. (2003). "The influence of using Grade 500 reinforcement in beam-column joint zones and on the stiffness of reinforced concrete structures." *Auckland Uniservices Report No. 9089*, July.