13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 3472

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

Leslie M. MEGGET[1], Meg B. BARTON[2] and Richard C. FENWICK[3]

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 ¾ 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 described in this paper followed 4 previous units tested at Auckland University which incorporated the then current high strength reinforcing steel with a 5 percentile characteristic yield strength of 430 MPa, Megget *et al*[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 1and 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 ¾ 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 ¾ of the beam's nominal moment, followed by displacement based cycles to displacement ductilities of ± 2, 4, 6 and sometimes 8. Failure was assumed

[1] Dept. of Civil & Environmental Engineering, University of Auckland, New Zealand. Email: l.megget@auckland.ac.nz
[2] Postgraduate Student, Dept. of Civil & environmental Engineering, University of Auckland, NZ.
[3] Visitor, Dept. of Civil Engineering, University of Canterbury, Christchurch, NZ.

when the load sustained 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 a hinge formed 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 prior to the 90-degree bend, Megget *et al*[1]. There was also bond loss along the bottom beam bars in the joint and this was associated with substantial joint degradation.

Unit 2 had an additional horizontal force applied on the beam's longitudinal axis (at the beam's free end) to resist the beam elongation, which occurs when beam plastic hinges develop (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 it didn't extend as far as the bend.

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 = 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 D20H in units 1-3).

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 ¾ column depth from the inner column face, the Standard requires that the bars' development length must be measured from a point, the lessor 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 40 MPa concrete. The length $8d_b+L_{dh}+h_c/4$ (160+303+125), 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 reinforcing and concrete strengths found for each unit. Figure 1 shows the reinforcing details of the units 5-8.

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 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 (as allowed when the leg length is greater than a third of the column depth; they were about $h_c/2$ here) then this joint was over designed by about 60%, but if the short legs were neglected the over design was only 8%.

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

Unit	Beam main Reinforcing (f_y in MPa)	Beam ties in PHZ	Column main reinforcing	Column shear ties in potential PHZ	Joint Horizontal Shear ties	Concrete strength after testing (MPa)
5	4-D20H Top, 4-D20H Btm. L-bars (572 MPa)	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	37.9
6	4-D20H Top, 2-D20H Btm. L-bars (542 MPa)	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	31.4
7	4-D20H U-bars (543 MPa)	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	40.7
8	6-D20H U-bars (541 MPa)	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	42.5

In Unit 7 the joint had only 4 sets of R8 ties although the Standard requires 5. This was to check any over conservatism in the Concrete Standard as it relates to joint shear ties. Lin & Restrepo [5] 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. The current overstrength factor for the 500E Grade beam yield stress of 1.35 was not used in these designs but a factor of 1.15 (to allow for strain-hardening) was incorporated into the design because the actual yield stress of the beam bars was known and used in the calculations.

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 0.2 f'_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. Assuming the area of the short legs was included then the 4 tie-sets provided about 90% of the Standard's [2] requirements.

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 segment experienced small axial tension and compression alternatively during every loading cycle. The specified 28-day concrete compressive strength for Units 5-8 was 40 MPa.

To measure displacements in the test unit 6mm 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

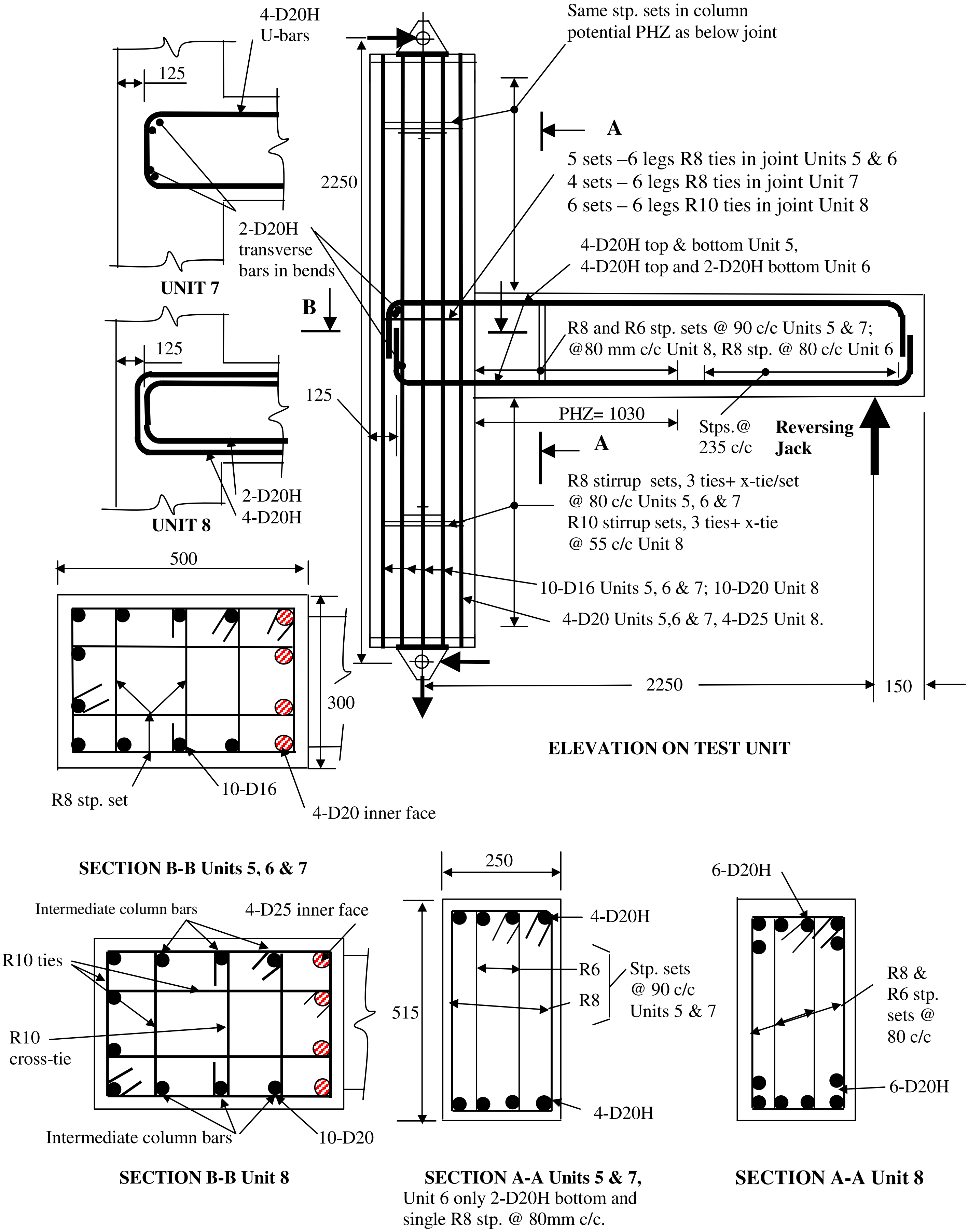

Figure 1. Details of test units 5-8.

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) 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, 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 the 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 hysteresis 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 29% 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.

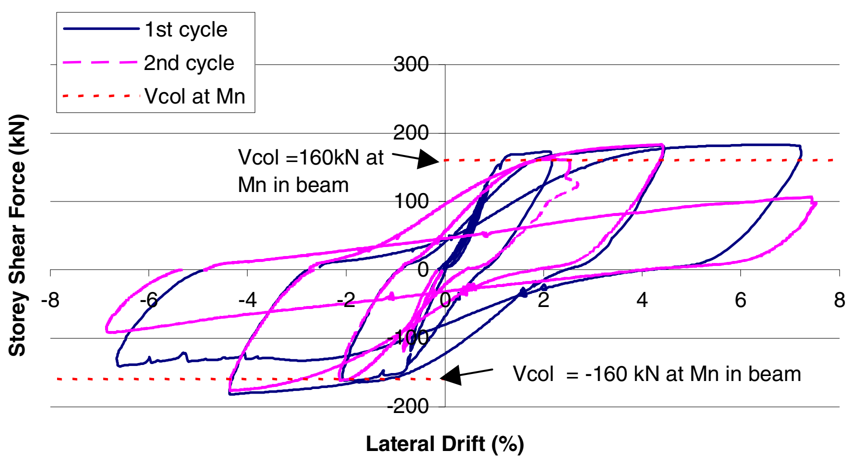

Figure 2. Unit 5 Column Shear - % Storey Drift Ratio hysteresis loops.

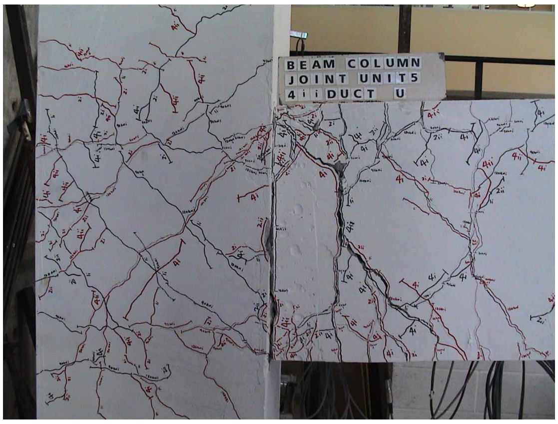

Figure 3. Unit 5 after 4% drift cycles.

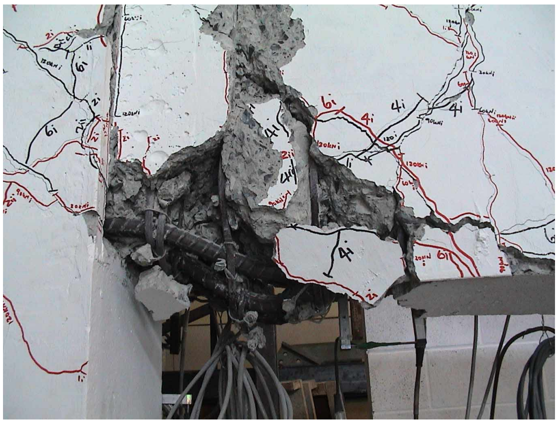

Figure 4. Bar buckling during 6% drift cycles.

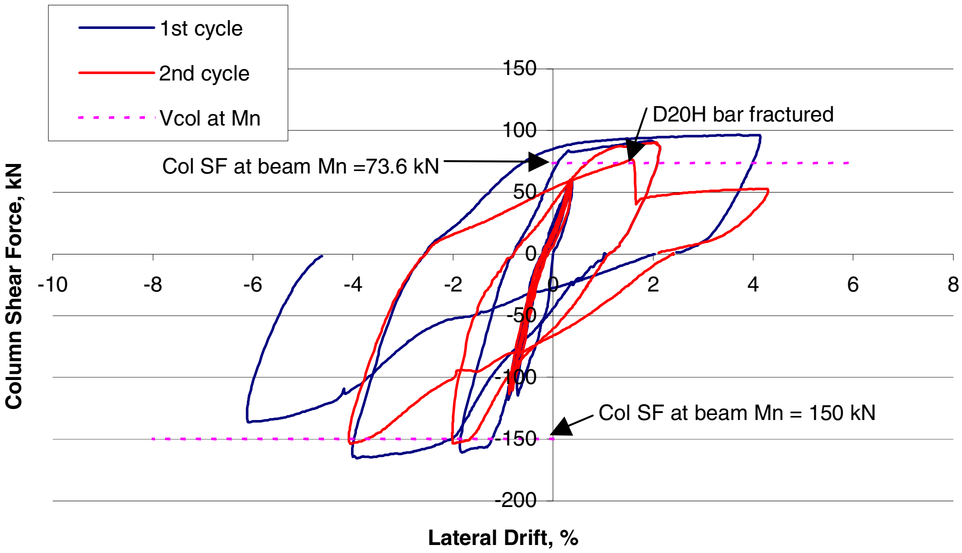

Figure 5. Unit 6: Story Shear (kN) – % Storey Drift Ratio hysteresis loops.

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 ± 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.

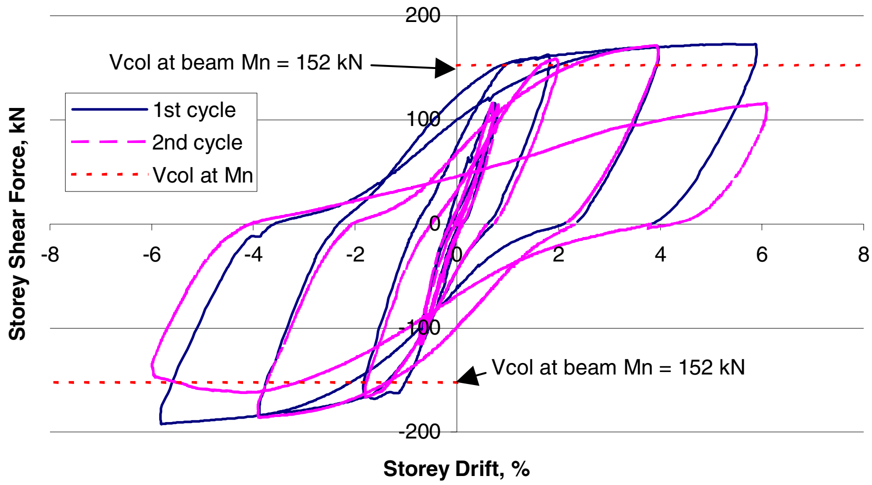

Figure 7. Unit 7 Storey Shear - Storey Drift (%)

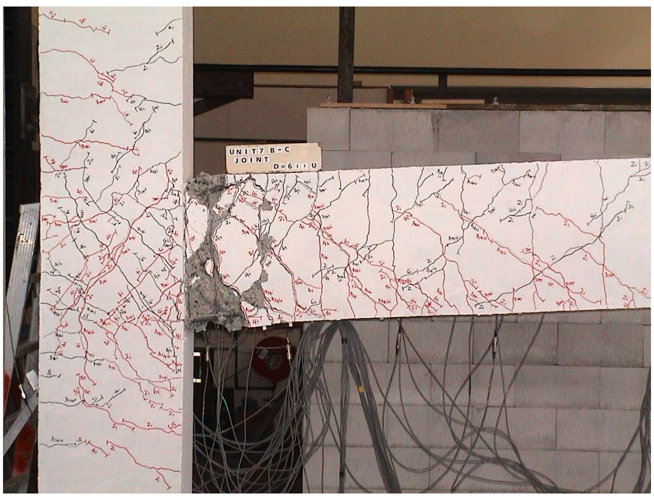

Figure 8. Unit 7 after completion of the 6% drift cycles.

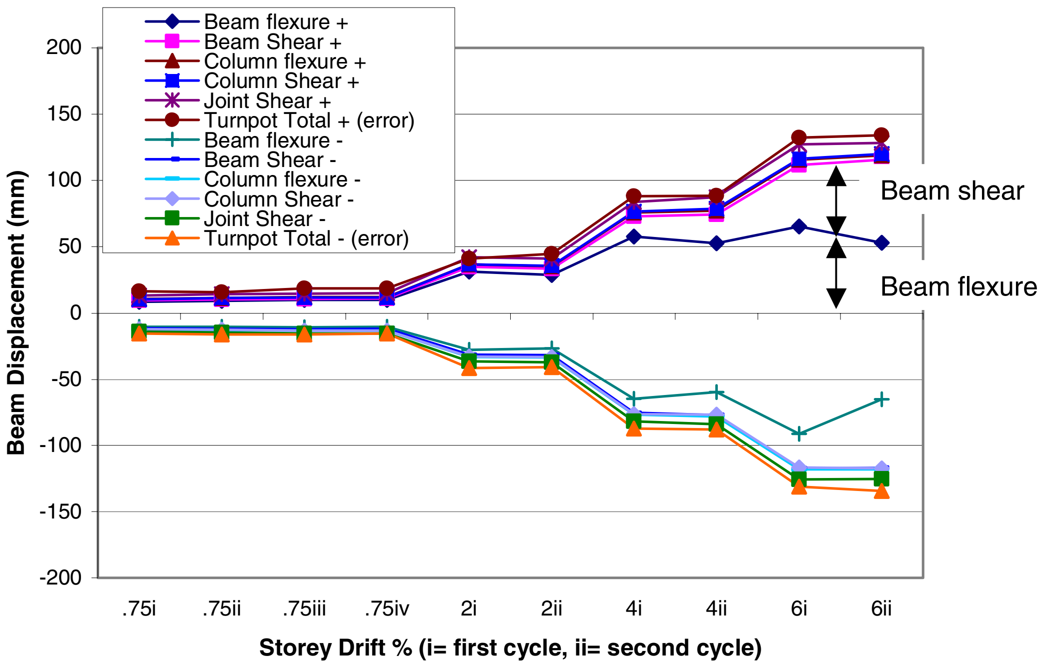

Figure 9. Unit 7 Beam components of displacement during testing.

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 ductility 4 and 6 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% [6] would limit the structural ductility factor to 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. 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 200mm of the column face led to disintegration of the beam core concrete and buckling of the beam reinforcement, see Figure 12.

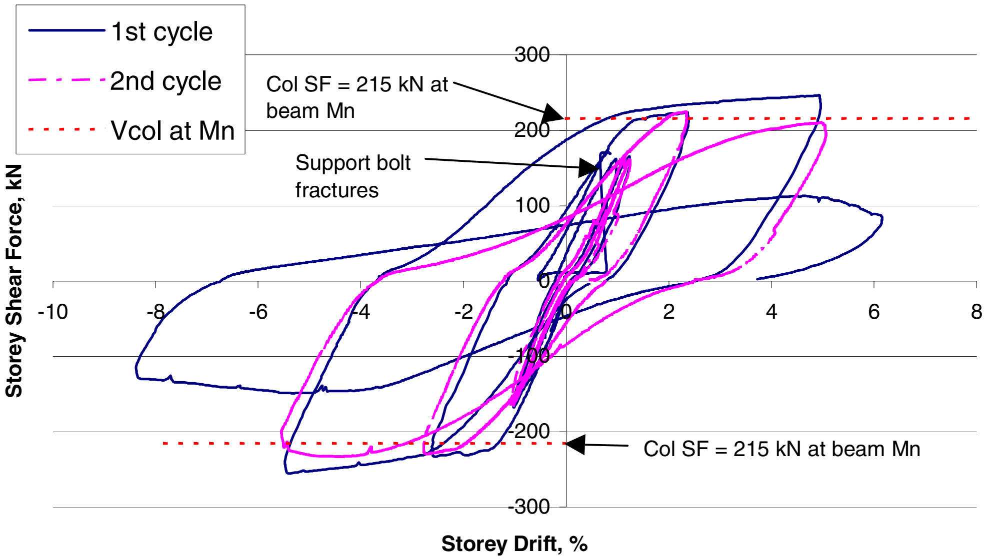

Figure 10. Unit 8 Storey Shear (kN)- Storey Drift (%)

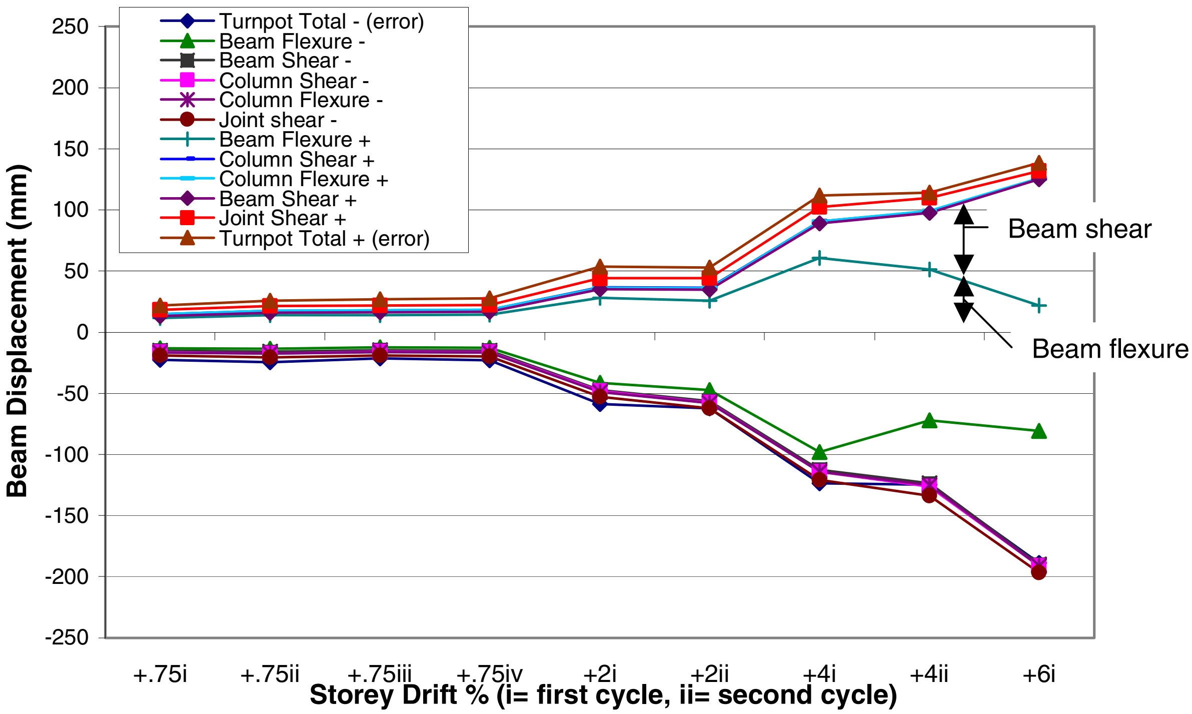

Figure 11. Unit 8 contributions to beam deflection during testing.

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 *et al*[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.

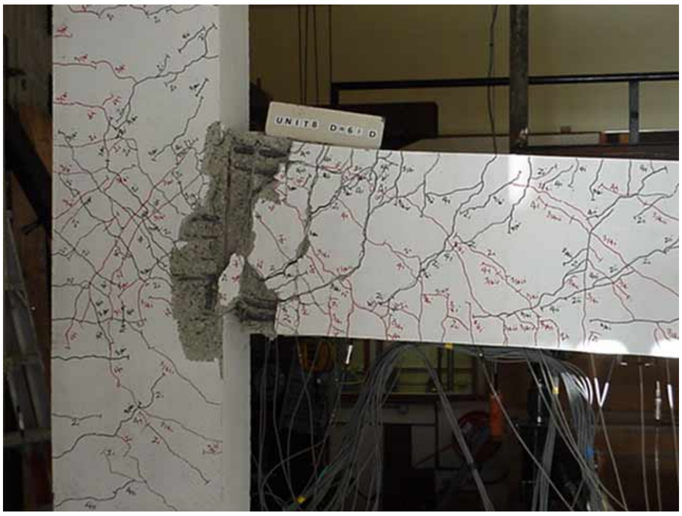

Figure 12. Unit 8 during the 7.5% drift cycles. Note shear deformations in PHZ.

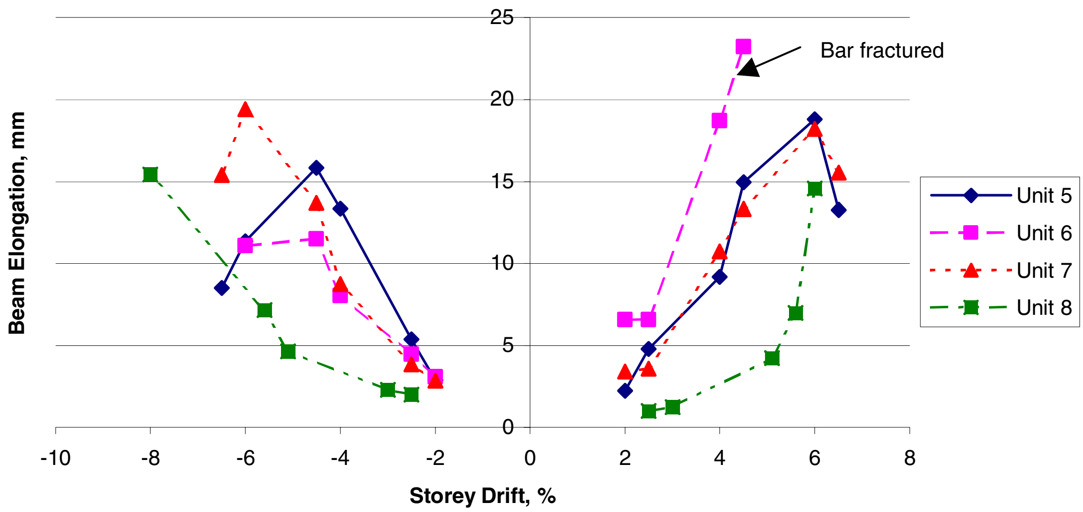

Figure 13. Beam elongation at cycle peaks for Units 5-8.

Beam stiffness

Table 2 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.

Table 2. 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

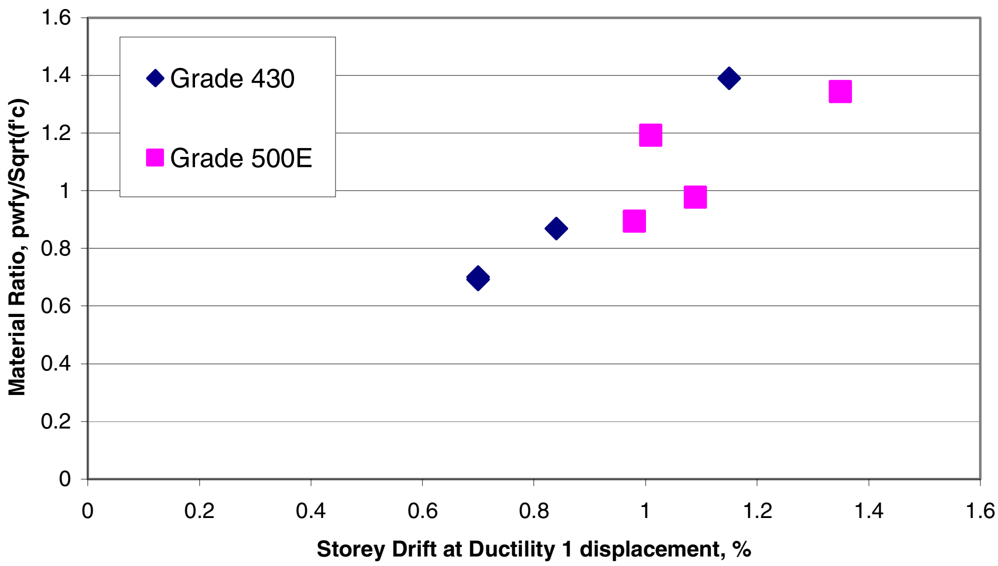

Figure 14. Ductility 1 Storey Drift versus Material Ratio, p_w ($f_y/\sqrt{f'_c}$)

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 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 [7].

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 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 (NZS3101:1995) [2] requirements for horizontal joint zone ties but still showed no joint distress. It may be possible to reduce the joint shear requirements in external beam-column joints with low column axial loads.

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 [6] buckling appears to occur at lower drift ratios when the beams incorporate 500 MPa reinforcing. This "global" type of buckling occurs 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 ductility. 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 90-degree hook + 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 ductility one 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 reinforcement is used.

ACKNOWLEDGEMENTS

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

REFERENCES

1. Megget L. and Fenwick R. "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, "Concrete Structures Standard, Part 1- The design of Concrete Structures." NZS 3101:1995.

3. Fenwick RC. and Megget LM. "Elongation and load deflection characteristics of reinforced concrete members containing plastic hinges." Bulletin NZSEE; 26(1): 28-41.

4. Megget LM. "The seismic design and performance of reinforced concrete beam-column knee joints in buildings." Earthquake Spectra; 19(4): 863-895.

5. Lin C-M. and Restrepo JJ. "Seismic behaviour and design of reinforced concrete interior beam-column joints." Bulletin NZSEE; 35(2) 108-128.

6. Standards New Zealand, "General structural design and design loadings for buildings." NZS 4203:1992.

7. Fenwick, R. and Megget L. "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 2003.