13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 44

BOND STRENGTH OF REINFORCED CONCRETE-BEAM COLUMN JOINTS INCORPORATING 500 MPA REINFORCEMENT

Nicholas BROOKE[1], Les MEGGET[2], Richard FENWICK[3] and Jason INGHAM[4]

SUMMARY

A database of beam-column joint test results has been assembled and analysed to determine appropriate design drift limits for the prevention of bond failure in reinforced concrete frames. In order to enhance the coverage of the database which predominantly contains units having small beam reinforcing bar sizes, further beam-column joints have been designed at the University of Auckland using 25 mm beam reinforcement. Results from the first two of these tests are reported. Despite the first unit not meeting the requirements of the recent amendment to NZS 3101:1995 with respect to column depth, the units did not exhibit a bond failure in the joint region.

INTRODUCTION

Since the introduction of grade 500 MPa reinforcing steel to the New Zealand market as a replacement for the previous grade 430 MPa reinforcement, concerns have been expressed concerning the validity of existing design guidelines with the new higher grade reinforcement. In particular, attention has been given to the increased likelihood of bond failure within interior beam-column joints.

In order to assess the influence of using grade 500 MPa reinforcing steel in beams, a database of test results for beam-column joint sub-assemblies was compiled. This database consisted of 59 tests. It included a database of 48 tests compiled by Lin [1] with additional tests reported by Blakeley et al. [2] and [3], Young [4], and Megget et al. [5]. This data has been analysed and suggestions are presented on how to control bond failure in joints zones.

Within the database there are few units incorporating reinforcement with a yield stress of 500 MPa or greater, and only one of these had beam reinforcement with bar diameters greater than 20 mm. To rectify this deficiency a further series of tests on four beam-column joints has been initiated at the University of Auckland. These tests use 25 mm grade 500E (HD25) reinforcing steel in the beams. The results of the first of these tests are discussed in this paper.

[1] Doctoral student, University of Auckland, Auckland, New Zealand. Email: edmund@ihug.co.nz
[2] Senior lecturer, University of Auckland, Auckland, New Zealand. Email: l.megget@auckland.ac.nz
[3] Visitor, University of Canterbury, Christchurch, New Zealand.
[4] Senior lecturer, University of Auckland, New Zealand. Email: j.ingham@auckland.ac.nz

BOND FAILURE IN BEAM COLUMN JOINTS

There are three failure modes for interior beam-column joints [6]. These are:

- Failure of the beam plastic hinge zones adjacent to the joint, through shear and flexure.
- Shear failure of the joint zone.
- Bond failure of the longitudinal beam reinforcement in the joint zone.

With respect to the stability of the whole structure, the least serious of these is bond failure. This failure mode results in a loss of strength and stiffness of the beams and hence leads to the preferred beam sway failure mode. In contrast the other two failure modes can reduce the strength of the columns and lead to the less ductile column sway (weak storey) failure mode [6]. Additionally, it is considered that some bond deterioration is inevitable in beam-column joints experiencing reversing inelastic demands [7]. For these reasons, it is logical to provide a lower level of safety against bond failure than against the other potential failure mechanisms.

In analysing the database of test results, it was necessary to identify those tests in which bond failure of the joint zone was the primary cause of failure. Examination of the test results showed that for joints containing at least 75% of the joint zone shear reinforcement required by NZS 3101:1995 a joint zone shear failure was unlikely to precede bond failure of the beam reinforcement [6]. This criterion reduced to 29 the number of tests, which could be used to assess bond performance.

Determination of drift level at which bond failure occurred

All units were subjected to cyclic loading histories. Bond failure was assumed to have occurred if the load sustained when the drift was half way between the target drift and a position of zero load was less than 25% of the maximum value in the previous cycle (see Figure 1). The failure was assumed to have taken place at the peak drift of the previous cycle. To recognize the superior performance of cases where a unit sustained a drift level several times before bond failure occurred, 0.25% was added to the failure drift (drift limit) for each successful half cycle to the same peak displacement before the onset of bond failure.

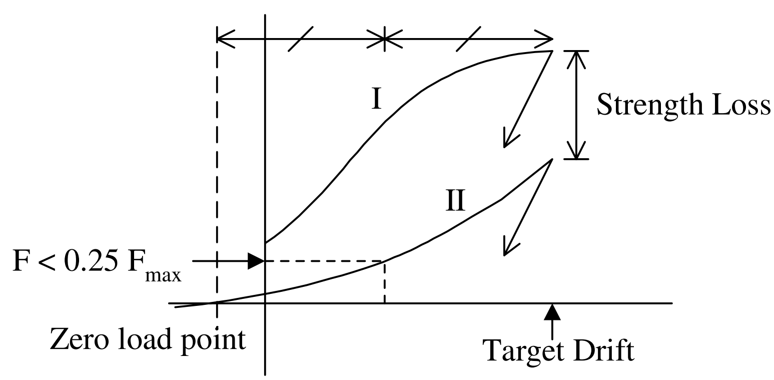

Figure 1 Criteria used to determine occurrence of bond failure.

It was thought likely that bond failure occurred at an earlier stage than that where strength loss was noted. This assumption was accounted for by defining a "modified drift limit" as equal to the drift limit from above divided by one plus the proportion of strength lost at the target drift. Use of the modified drift limit reduced the scatter of plots relating drift limits to other variables.

Analysis of test results

It is evident from Figure 2 that there is a strong correlation between the modified drift limit and the yield stress. It is noted that for the tests using higher grade reinforcement the ratio of bar size used in the test to the maximum bar size allowed by NZS 3101:1995 (numbers by data points) increased with reinforcement grade. This can be attributed to the relaxation of the bond criterion in the standard around the time grade 430 reinforcing steel was introduced.

By plotting modified drift at bond failure multiplied by the square root of the ratio of actual bar size to allowable bar size against yield stress (see Figure 3) design values of allowable drift can be assessed. If the maximum permitted bar diameter to be used (i.e. d_{ba}/d_{bc} = 1.0) the average value of modified drift at bond failure can be determined. For grade 500 reinforcing ($f_{y\ average}$ ~ 550 MPa) the value is 3.1%, with a standard deviation of 0.47%. Similarly, for grade 300 steel ($f_{y\ average}$ = 320 MPa) the value is 4.7%, standard deviation 0.55%. The data from the unit described in this paper is shown on both Figure 2 & Figure 3, but was not included in the analyses based on these figures.

For a 90% probability that bond failure will not occur in the ultimate limit state, the drift limits for the two grades of reinforcing are reduced to [6]:

- 3.5% drift for Grade 300 reinforcement
- 2.5% drift for Grade 500 reinforcement.

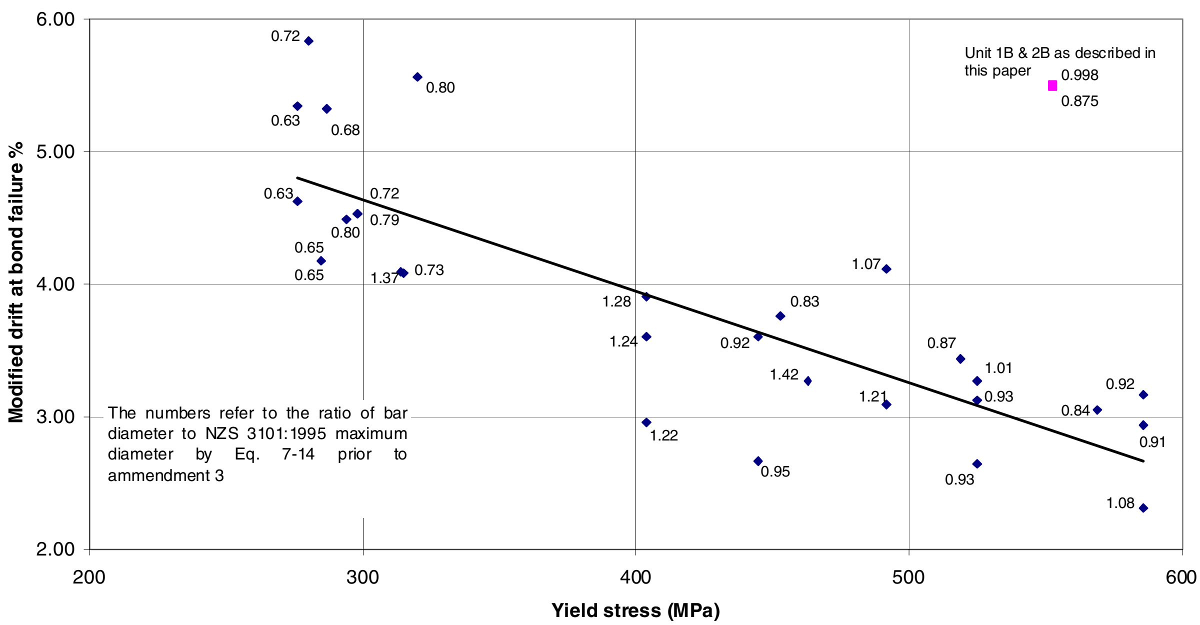

Figure 2 Modified drift at bond failure versus yield stress of reinforcement.

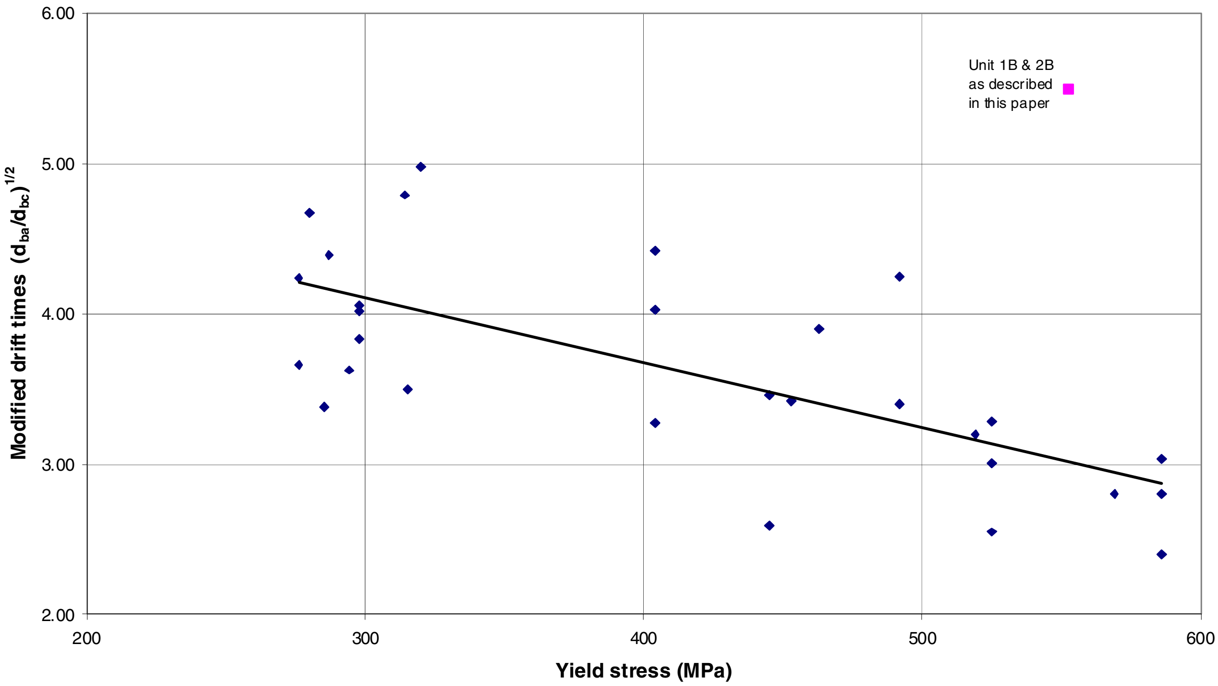

Figure 3 Modified drift at bond failure * $(db_a/db_c)^{1/2}$ versus yield stress.

It has been shown [8] that elastic methods of analysis can significantly underestimate drift values compared to those calculated by inelastic time history analysis. This is recognized by the New Zealand loading standard [9] through the inclusion of a factor to allow for the underestimation. This factor varies with building height from 1.25 for a building of height less than 15m to 1.67 for buildings taller than 30 m [9]. It is also necessary to account for the S_p factor incorporated in NZS 4203. To do this, the ultimate limit state drift should be divided by $1/S_p$, i.e. 1.5. It is reasonable to reduce this value somewhat to allow for the fact that the earthquake does not cycle between drift extremes as occurs in laboratory testing. Therefore, a value of 1.25 has been used to produce design drift limits to prevent bond failure (see Table 1).

Table 1. Design drift limits to prevent bond failure

Building height	Grade 300E	Grade 500E
(m)	(MPa)	(MPa)
<15	2.24%	1.60%
>30	1.68%	1.20%

ADDITIONAL INTERNAL BEAM-COLUMN JOINT TESTS

Design of Units

In order to fill the gaps in the database of beam-column test results, four further tests are being undertaken at the University of Auckland. Three of these units have been designed, and the first two have been built and tested. The beam longitudinal reinforcement for the first three units is to be kept constant as 3 HD25 bars top and bottom, while the target compressive strength of the concrete selected was 35, 50 and 70

MPa for units one, two and three respectively. Except where it was impractical to do so, the units are designed to comply with the New Zealand concrete design standard [10], including amendments up to February 2004.

Where possible dimensions were kept the same as those used by Young [4] and Megget et al. [5]. The beam dimensions were 500 mm deep and 200 mm wide. The columns were 360 mm wide, and the column depths were determined by bond strength requirements. In all units the covering concrete provided was reduced to 15 mm to maximise the reinforcement quantity that could be used. 15 mm was generally less than proscribed by NZS 3101 [10], but was considered acceptable given the short life span of the units and the protected covered environment they were built and tested in.

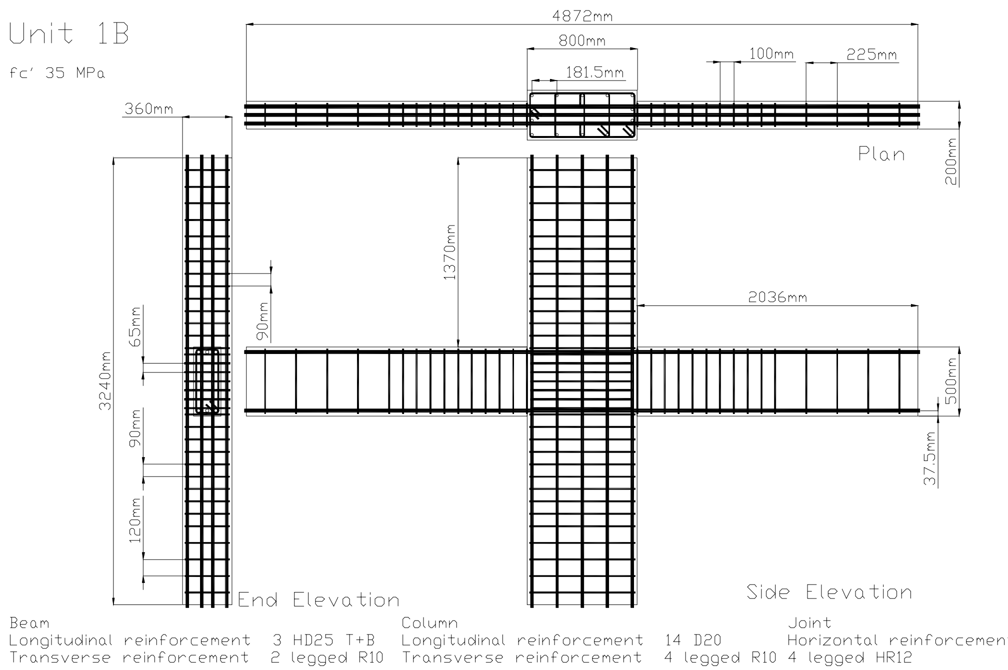

Figure 4 Principal dimensions and reinforcement plan of unit 1B (Brooke)

Using 3 HD25 reinforcing bars gave a reinforcement ratio (A_s/bd) of 1.6 %. For unit 1B this was greater than the maximum ratio of 1.5% allowed by NZS 3101:1995 [10]. For units two and three, the higher target concrete strengths allow higher reinforcement ratios (2% and 2.67% respectively). Details of unit 1B can be seen in Figure 4. The reinforcement layout and dimensions of Unit 2B were identical to unit 1B except that the higher concrete strength allowed the removal of one set of stirrups from the joint region. Design details of units 1B, 2B and the units tested by Young [4] and Megget et al. [5] are summarised in Table 2. Note that bond failure occurred in the units tested by Young and Megget et al. at 4.5% drift or less.

There are two equations for establishing the maximum ratio of reinforcing bar diameter to column depth in NZS 3101. The column depth required to allow the use of 25 mm reinforcing bar was determined using the less conservative equation 7-14 from clause 7.5.2.5 [10],

$$\frac{d_b}{h_c} \leq 6\left(\frac{\alpha_t \alpha_p}{\alpha_s}\right)\alpha_f \frac{\sqrt{f'_c}}{\alpha_o f_y} \qquad (1)$$

In equation 7-14 d_b is the bar diameter, h_c is the column depth, f'_c is concrete strength and f_y is the nominal yield stress of the reinforcement used. The α factors account for whether the joint is part of a one- or two-way frame, the overstrength factor of the reinforcing steel (1.4 for grade 500E reinforcing steel), the depth of fresh concrete cast beneath a given bar, axial load and the ratio of the areas of top and bottom steel in the beam. Clause 7.5.2.5 is modified in amendment three to NZS 3101:1995 [10] to allow for the more severe bond demands placed on the joint by high strength reinforcement. The amendment requires that the maximum bar diameter allowed shall be 70% of the value given by equation 7-14 (in the draft amendment used for the design of units 1B and 2B the maximum bar diameter allowed was 80% of that given in equation 7-14), unless one or more of the given conditions is satisfied. These conditions are;

- Grade 300 reinforcement shall be used for longitudinal beam steel through the joint;
- Inter-storey deflections are calculated using the time history method and satisfy the limits in NZS4203 (clause 2.5.4.5);
- The storey drifts at the ultimate limit state do not exceed 1.2% when calculated using the equivalent static or modal response spectrum methods;
- The beam column joint is protected from plastic hinge formation at the faces of the column;
- The plastic hinge rotation at either face of the column does not exceed 0.006 radians.

Table 2. Bond strength details of University of Auckland tests

Unit	Column depth	f'_c actual	f_y actual	d_b	d_b allowed*	P_w beam	Bond failure drift
	(mm)	(MPa)	(MPa)	(mm)	(mm)	(%)	(%)
Young	520	49.2	519	16	24.7	1.13	4.5
Megget et al. 1	520	29.3	588	16	16.8	0.64	1.7
Megget et al. 2	520	40.4	588	16	19.8	1.31	3.4
Megget et al. 3	520	40.9	588	16	19.9	0.64	2.8
Unit 1B	800	31.2	552	25	27.2	1.59	-
Unit 2B	800	40.6	552	25	28.6	1.59	-

*Maximum bar size allowed by NZS 3101:1995 excluding amendment 3. Maximum allowed including amendment 3 is 70% of presented value.

Units 1B-3B designed at the University of Auckland did not fulfill any of the five conditions, resulting in large column sizes being required, especially for unit 1 due to the low target concrete strength used. The design of the column depth of units 1B and 2B is summarised in Table 3. The column depth was calculated using the specified and actual material properties. When the actual yield stress was used the overstrength factor was taken as 1.15 to allow for the strain-hardening only. Note that the units were cast on their side so no allowance was required for fresh concrete depth beneath the reinforcement. For reasons of practicality the column depth of unit 1 was reduced to 800 mm. This value is close to what would have been designed without amendment three to NZS 3101:1995 [10].

Loading Sequence

Unit 1B and 2B were tested in the University of Auckland Civil Engineering test hall, lying parallel to the floor. It was intended that an elastic load cycle to 75 % of the nominal yield strength would be completed in both loading directions. Problems were encountered with the test setup during the first semi cycle of unit 1B. These caused the applied load to exceed the yield load, and a decision was made to load the unit in the other direction to a displacement equal to that reached in the first direction. Following this "elastic" cycle, double reversing cycles to 1.5%, 2%, 3% and 4% lateral drift were applied, continuing until a significant drop in strength occurred. The loading cycle (see Figure 5) was applied as planned for unit 2B. The units were instrumented extensively with portal displacement gauges as shown in Figure 6. Displacement at the load points was measured using turnpot gauges.

Table 3. Design of column depth for units 1B and 2B

f'_c nom.	f'_c act.	d_b	f_y nom.	f_y act.	α_o	α_t	α_p	α_s	α_f	h_c nom.	h_c act.
(MPa)	(MPa)	(mm)	(MPa)	(MPa)						(mm)	(mm)
35	31.2	25	500	552.4	1.4*	1.0	1.0	1.55	1.0	955	918
50	40.6	25	500	552.4	1.4*	1.0	1.0	1.55	1.0	799	804

* 1.15 for h_c act. calculation.

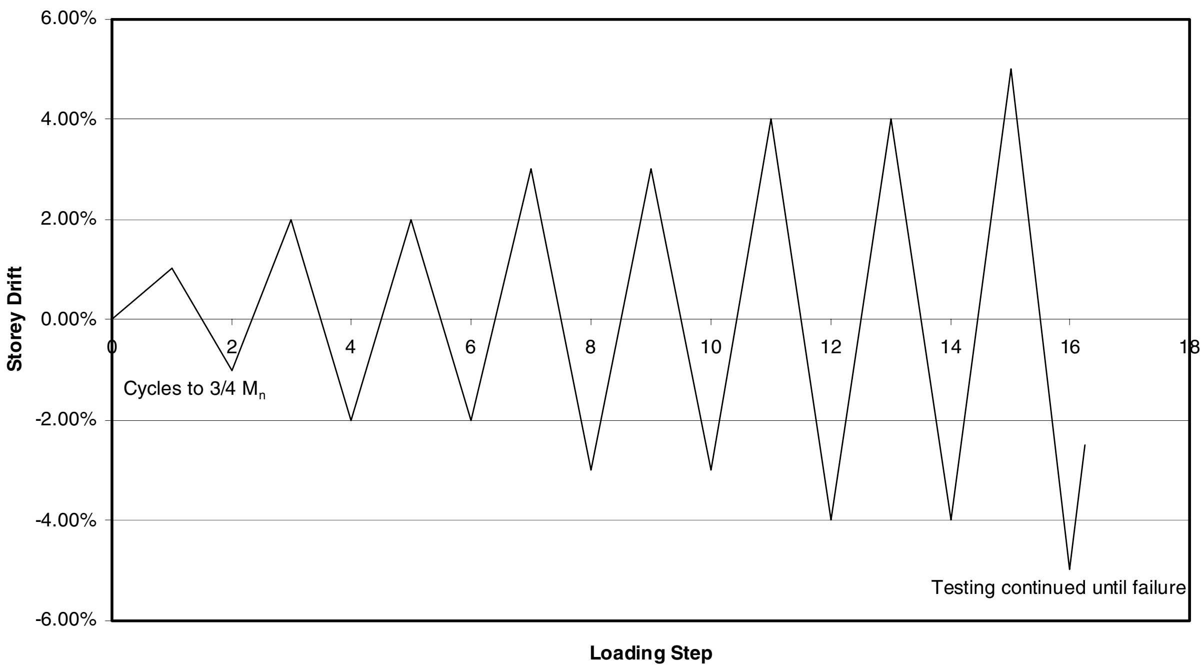

Figure 5 Planned loading cycle for units 1B and 2B

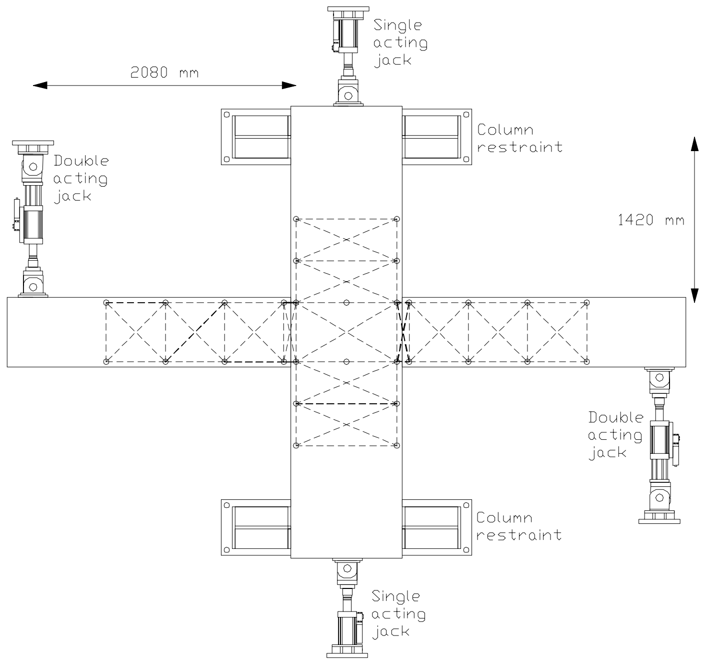

Figure 6 Portal gauge layout and loading arrangement for unit 1B and 2B.

Test Observations and Results

From the first semi-cycles of the tests it was determined that the yield lateral drift of the unit was approximately 1.32% for unit 1B and 1.22% for unit 2B. This value was approximately in agreement with the value of 1.35% predicted using methods presented by Priestley [11]. Using standard moment-area theory and the deformations measured on the beam alone it is straightforward to evaluate the effective moment of inertia of the beams based on the force and displacement at yield, and assuming a linear force-displacement relationship up to first yield. The ratio of effective to gross moment of area is

$$\frac{I_e}{I_g} = \frac{FL^3}{3\delta E_c I_g} \qquad (2)$$

Where F is the force, L is the length from the application point of the force to the column face, δ is the displacement of the load point due to beam shear and flexure only, E_c is the elastic modulus of concrete as determined in NZS 3101:1995, and I_g is the gross section moment of inertia. This calculation is summarised in Table 4. The effective moment of inertia of both units is notably higher than the value of $0.32I_g$ suggested in the amendment 3 of NZS 3101:1995. However the NZS3101 amendment 3 effective moment of inertia values are for beams with more prototypical lower reinforcement ratios (p~0.7%) than that of unit 1B and 2B (p=1.6%).

When compared to the nominal column shear force, i.e. the column shear force calculated to occur when the beams develop their nominal moment capacity, the yield strength of unit 1B was reasonably close (approximately 7.5% greater in the first semi-cycle) to that calculated using actual material properties (see Figure 7). This may be a result of the problems encountered during the testing of unit 1B which lead to the unit yielding during the first "elastic" cycle. The actual yield strength of unit 2B was more closely matched to the predicted strength, being only 4.5% greater during the first semi-cycle (see Figure 8).

Table 4 Calculation of effective moment of inertia for beams in Unit 1B and 2B

Unit	Beam	Yield Force	Beam Length	Yield Displacement*	f'$_c$	E_c	I_g Beam	Ratio I_e/I_g
		(kN)	(mm)	(mm)	(MPa)	(MPa)	(mm^4)	
1B	Left	166.9	2085.25	19.7	31.2	25445	2.083E+09	0.48
1B	Right	165.4	2078.25	19.9	31.2	25445	2.083E+09	0.47
2B	Left	172.8	2031	19.5	40.6	28054	2.083E+09	0.42
2B	Right	160.2	2052	20.6	40.6	28054	2.083E+09	0.38

*Beam shear and flexural displacement components only

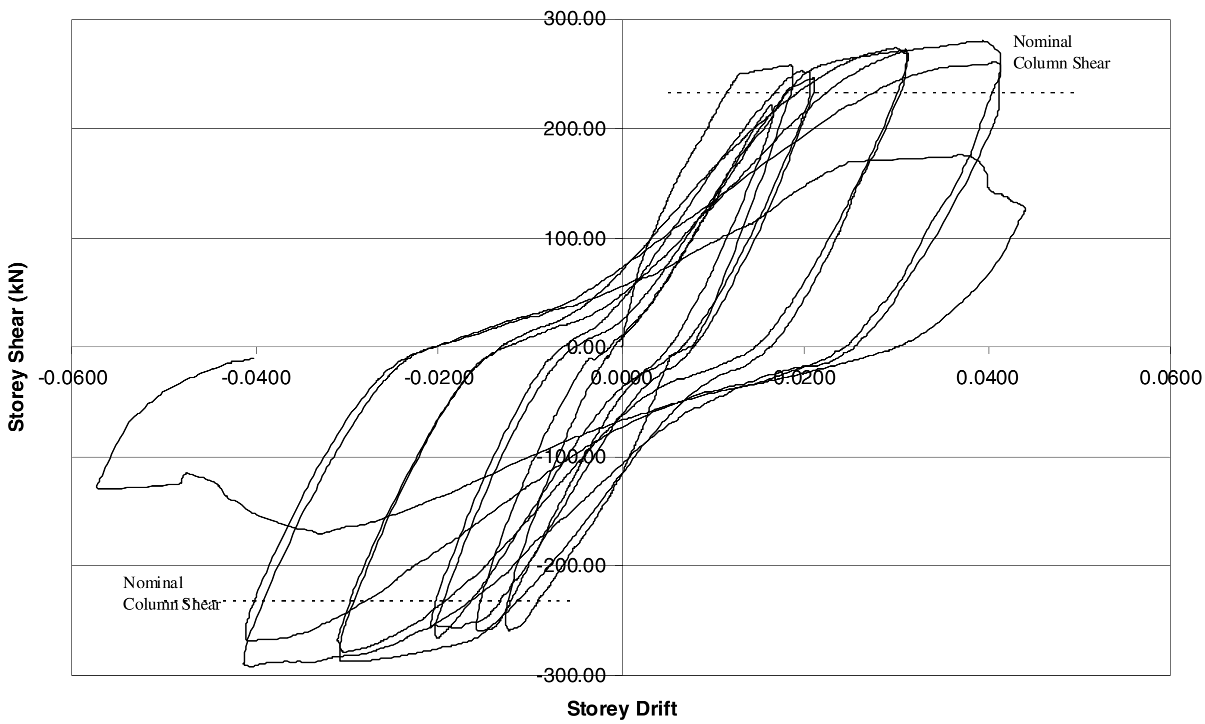

Figure 7 Unit 1B storey column shear versus storey lateral drift.

The damage sustained by units 1B and 2B during testing followed expected patterns. Shear and flexural cracks formed in both beams, eventually linking across the full depth of the beams. Concurrently, fine flexural cracks developed in the column, and shear cracking occurred in the joint zone. These cracks in the column and joint did not open beyond approximately 0.5 mm, indicating that the column reinforcement remained within its elastic strain range. For the cycles to 2% drift or more, almost all damage occurred in the beam plastic hinge zones, with cracks adjacent to the column on both sides opening to approximately 7 mm when the drift was 3%.

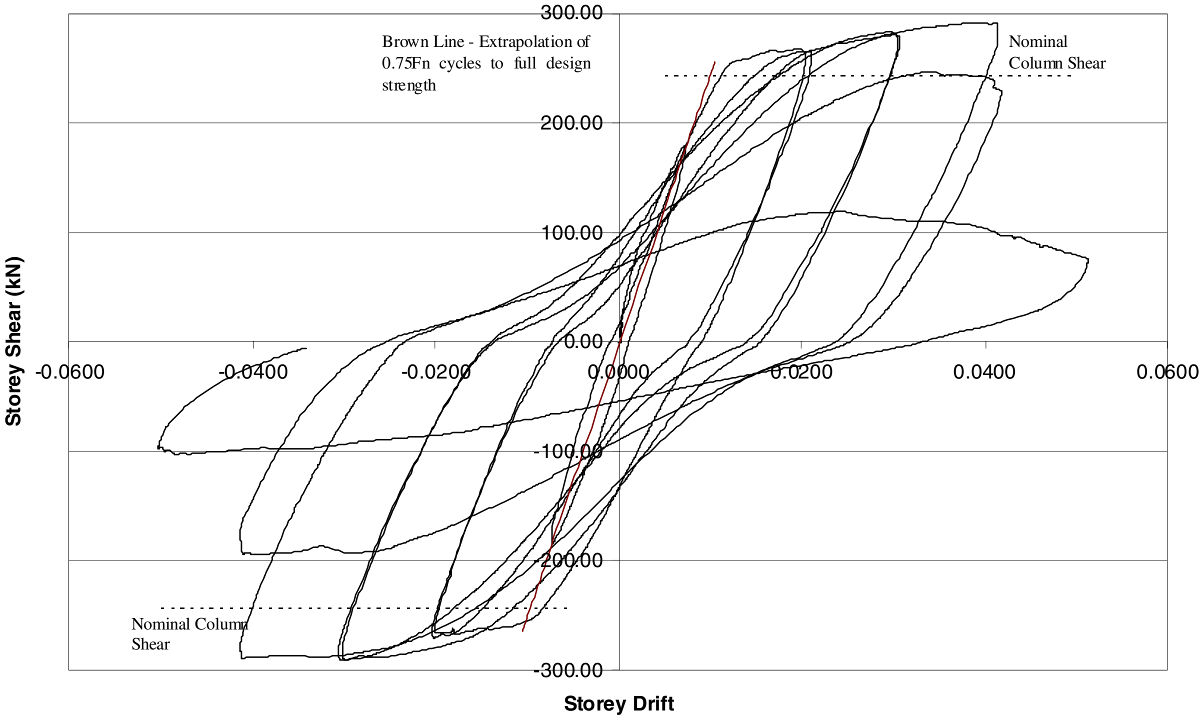

Figure 8 Unit 2B storey column shear versus storey lateral drift.

In contrast to the earlier tests at the University of Auckland neither unit 1B nor 2B experienced bond failure, despite neither conforming with the most recent amendment to NZS 3101:1995 [10] when actual material properties were used in calculations (see Table 2). For unit 1B the beam reinforcement slipped no more than 2.2 mm through the joint zone at a lateral drift of 5%, while for unit 2B slip did not exceed 0.5 mm. Moreover the stiffness of the both units at low load levels remained similar through out the test (see Figure 7 and Figure 8). It is possible the improved response is related to the large quantity of vertical reinforcement in the column region, which is thought to improve bond performance. Equation 11-7 of NZS 3101:1995 [10] required vertical joint shear strengths of 577 kN and 440 kN for units 1B and 2B respectively, while the six column interior HD20 reinforcing bars provided a total nominal strength of 942 kN. The quantity of vertical joint reinforcement provided more closely matched that required for the two further units tested to complete this series of tests.

Testing of the first unit was halted when the primary longitudinal reinforcement of both beams buckled during the first complete cycle to 5% lateral drift, leading to severe torsional distortion of the beams. Therefore, the maximum drift achieved before strength dropped below 80% of the previous maximum was 4%, and the ductility achieved was $\mu = 3.04$ based on a yield drift of 1.32%. It is noted that the final drift level considerably exceeds the maximum allowable drift in NZS 4203 [9], AS/NZS 1170 [12] and over-seas codes of practice [13]. For unit 2B the strength achieved during the fourth semi-cycle to 4% (in the negative direction) failed to achieve 80% of the strength developed during the second semi-cycle (also in the negative direction), meaning that failure occurred during this semi-cycle. Despite this failure, testing was continued, leading to both beams twisting during the first complete cycle to 5% drift as in the first test. It was noted that during the second semi-cycle to 5% drift the left beam exhibited a large shear distortion in the plastic hinge zone instead of twisting. This occurred due to large quantities of core concrete breaking up and falling from the hinge zone allowing the reinforcement to deform freely.

CONCLUSIONS

- Based on the analysis of a database of previous test results design drift limits for reinforced concrete frames should be between 1.20% and 2.24 % depending on building height and reinforcement grade used.
- In previous beam-column joint tests with Grade 500 beam reinforcement, bond failure occurred between storey drifts of 1.7 and 4.5%. The longitudinal reinforcement of the first two units with D25H bars did not slip substantially before buckling of the plastic hinge zones prevented further testing, despite having column depths that did not comply with the amended Standard.
- The unexpectedly good bond performance of the test units may have occurred because of the large quantity of vertical reinforcement in the joint region. Further tests are to be conducted to investigate this possibility.

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