BOND STRENGTH IN INTERNAL BEAM-COLUMN JOINTS INCORPORATING HIGH STRENGTH REINFORCEMENT

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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 an existing data base of internal beam column joint tests there was a lack of test results of beams reinforced with high strength reinforcing bar diameters greater than 16mm. To enhance this data base and improve design criteria for bond in internal beam column joint zones a series of tests of beam-column sub-assemblies was planned at the University of Auckland. The results of three of these tests are described in the paper. Bond failure occurred in one of these tests with bar buckling limiting the capacity of two of the tests. There is some indication that the quantity of intermediate column bars in the joint zone influences the bond resistance. The results confirm previous observations that the flexibility of beams constructed using high-grade reinforcement, such as Grade 500E, severely reduces the structural ductility factor that should be used in seismic design.

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 when applied using 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, 3], Young [4], and Megget et al. [5]. This data has been analysed and suggestions are presented here on how to control bond failure in joint zones [6].

Within the database there were 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. All four units have been constructed, and the test results of the first three units are discussed in this paper.

BOND FAILURE IN BEAM-COLUMN JOINTS

There are three failure modes for interior beam-column sub-assemblies [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 overall stability of the structure, the least serious of these is bond failure. This failure results in a loss of stiffness and strength of the beams. Consequently this form of failure may lead to a premature beam sway failure mechanism in a multi-storey moment resisting frame. However, this failure mode is preferable to those associated with a weak storey, which may result in the formation of a column sway mode. This would lack ductility and be very sensitive to P-delta actions [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 allow a higher

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probability of bond failure occurring than of the other potential failure mechanisms occurring.

Design for the prevention of bond failure in NZS 3101:1995

The occurrence of bond failure between concrete and steel reinforcement is complicated. It depends on many parameters, including concrete strength and confinement, reinforcement yield strength and bar diameter, and the length of reinforcement over which bond can develop. There are two equations for establishing the maximum ratio of reinforcing bar diameter to column depth in NZS 3101:1995 [8]. The less conservative of these is equation 7-14 from clause 7.5.2.5 [8], reproduced below:

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

In the above equation 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. The α factors account for whether the joint is part of a one- or two-way frame (α_f), the overstrength factor of the reinforcing steel (α_o, 1.4 for grade 500E reinforcing steel [9]), the depth of fresh concrete cast beneath a given bar (α_t), axial load (α_p) and the ratio of the areas of top and bottom steel in the beam (α_s). Clause 7.5.2.5 is modified in amendment three to NZS 3101:1995 [8] to allow for the more severe bond demands that are placed on the concrete in the joint region by high strength reinforcement. The amendment requires that the maximum bar diameter allowed shall be 70% of the value given by equation (1) above, unless one or more of the given conditions is satisfied. These conditions are;

- Grade 300 reinforcement shall be used for longitudinal beam steel reinforcement through the joint;
- Inter-storey displacements are calculated using the time history method and satisfy the limits in NZS 4203:1992 [10] (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.

Determination of drift level at which bond failure occurred

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 indicated that for joints containing at least 75% of the joint zone shear reinforcement required by NZS 3101:1995 [8] 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 that could be used to assess bond performance.

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 strength developed in the direction of loading (see Figure 1). The failure was assumed to have taken place at the previous peak displacement. To recognise 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.

It was evident from tests instrumented to measure longitudinal reinforcement slip in the joint region that bond failure occurred at an earlier stage than

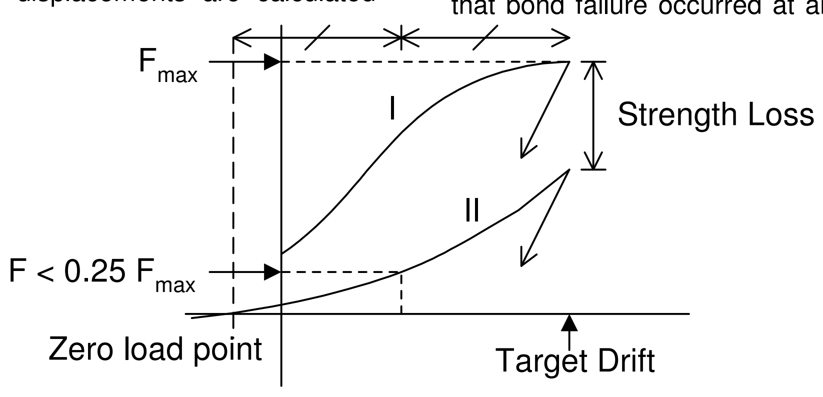

Figure 1 Criteria used to determine occurrence of bond failure.

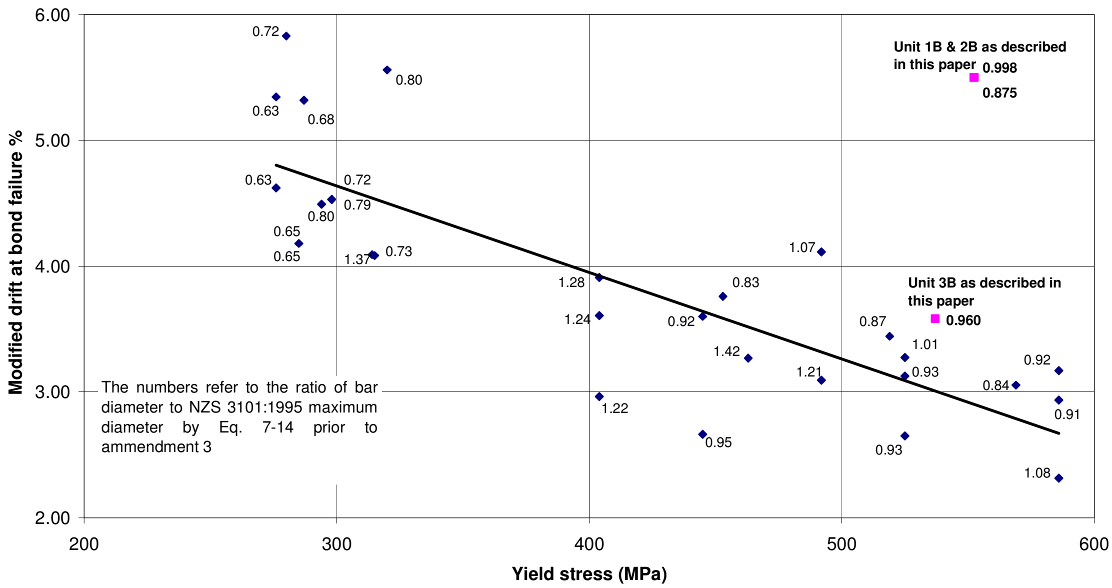

Figure 2 Modified drift at bond failure versus yield stress of reinforcement.

that where strength loss was noted. This 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

Figure 2 shows a plot of the modified drift at failure versus the yield stress of beam reinforcement. It shows that there is some correlation between the modified drift limit and the measured reinforcement 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 [8] (numbers adjacent to 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.

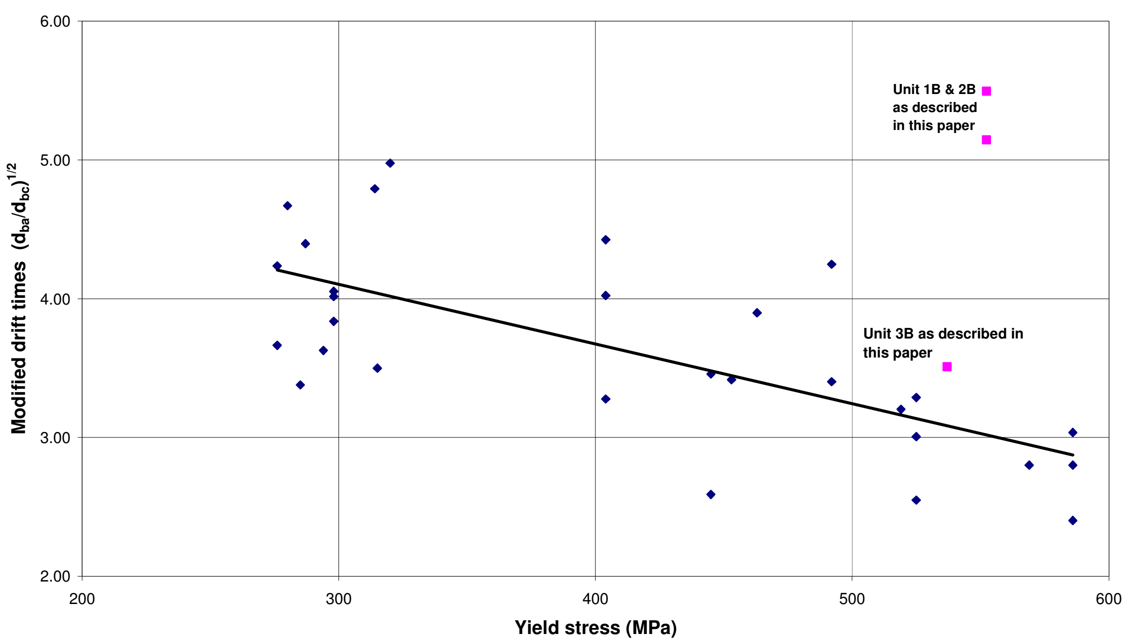

Figure 3 Modified drift at bond failure * $(d_{ba}/d_{bc})^{1/2}$ versus yield stress.

By plotting modified drift at bond failure multiplied by the square root of the ratio of actual bar size to allowable bar size ($(d_{ba}/d_{bc})^{1/2}$) against yield stress (see Figure 3) design values of allowable drift can be assessed. If the maximum permitted bar diameter is used (i.e. $d_{ba}/d_{bc} = 1.0$) then the average value of modified drift at bond failure can be determined. For grade 500 steel reinforcement ($f_{y\ average} \sim 550$ MPa) the value is 3.1%, with a standard deviation of 0.47%. Similarly, for grade 300 steel reinforcement ($f_{y\ average} = 320$ MPa) the value is 4.2%, with a standard deviation of 0.55%. The data from the units 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 at the ultimate limit state, the drift limits for the two grades of reinforcement are reduced to [6]:

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

It has been shown [11] that elastic methods of analysis can significantly underestimate drift values compared to those calculated by inelastic time history analysis. This is recognised by the New Zealand loading standard NZS 4203:1992 [10] 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 [10]. It is also necessary to account for the S_p factor incorporated in NZS 4203:1992. To do this, the ultimate limit state drift should be multiplied by S_p, i.e. 0.67. It is reasonable to increase this value somewhat to allow for the fact that in an earthquake displacements do not cycle between the extremes of displacement as occurs in most laboratory tests. Therefore, a value of 0.8 has been used to give the recommended design drift limits to prevent premature 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 beam-column joint test units

To extend the existing data base on internal beam column joint tests 4 sub-assemblies are being built and tested at the University of Auckland. In these tests high strength 25 mm steel bars are used as reinforcement in the beams. For all four units the beam longitudinal reinforcement was three HD25 bars top and bottom, while the target compressive strength of the concrete selected was 35 MPa for unit one, 50 MPa for units two and three and 40 MPa for unit four. Except where it was impractical to do so, the units were designed to comply with the New Zealand concrete design standard [8], including amendments up to

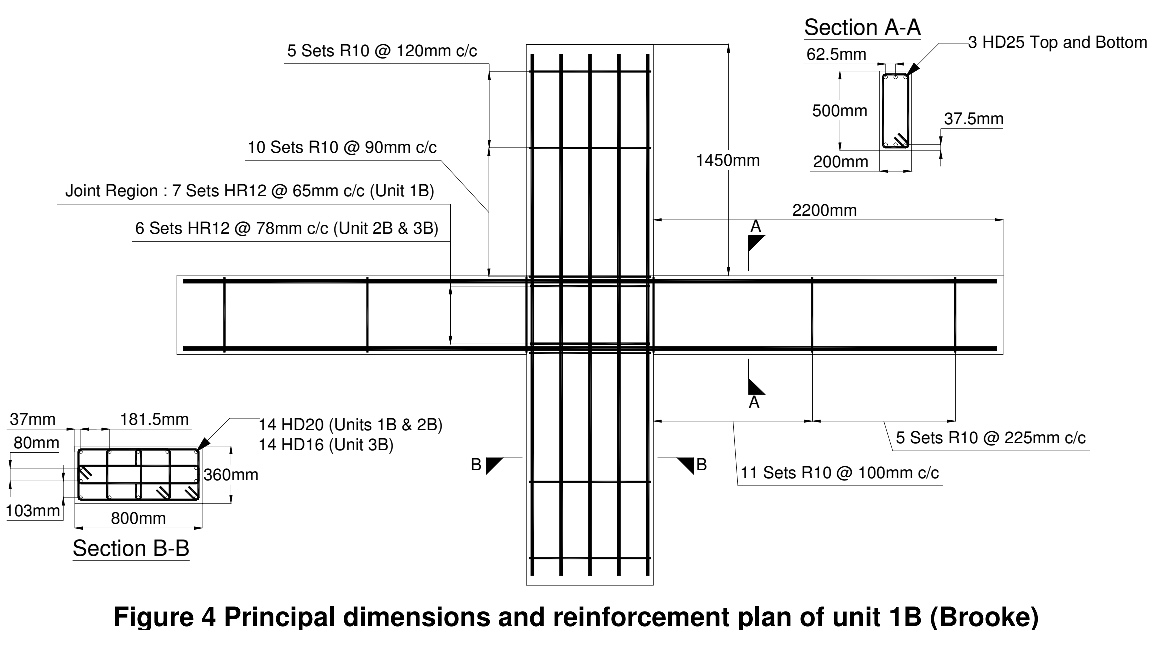

Figure 4 Principal dimensions and reinforcement plan of unit 1B (Brooke)

Table 2. Bond strength related design details of University of Auckland test units

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	-
Unit 3B	675	44.8	537	25	26.1	1.59	5.0

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

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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 cover measured to the stirrups or ties was limited to 15mm to maximise the reinforcement quantity that could be used. This cover was generally less than prescribed by NZS 3101 [8], but was considered acceptable given the short life span of the units and the protected covered environment they were built and tested in.

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 [8]. For the other units, the higher target concrete strengths allow higher reinforcement ratios (2%, 2% and 1.67% for units 2B, 3B and 4B 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. Unit 3B was similar in reinforcement layout, but had a reduced column depth of 675 mm. Design details of units 1-3B 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.

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 [8]. Units 1B-3B designed at the University of Auckland did not fulfill any of the five “amendment three” conditions outlined previously, resulting in large column sizes being required, especially for unit 1 due to the low target concrete strength used.

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

Unit	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)
1B	35	31.2	25	500	552.4	1.4*	1.0	1.0	1.55	1.0	955	918
2B	50	40.6	25	500	552.4	1.4*	1.0	1.0	1.55	1.0	799	804
3B	50	44.8	25	500	537	1.4*	1.0	1.0	1.55	1.0	799	745

* 1.15 for h_c act. calculation.

The design of the column depth of units 1B, 2B and 3B is summarised in Table 3. The required column depth was calculated twice, firstly using the design strengths of the materials and secondly using the measured material strengths. With the design yield stress of the reinforcement an over-strength factor of 1.4 was used to allow for the likely yield strength and strain hardening [9]. However, with the measured yield stress a value of 1.15 was used as in this case only allowance for strain hardening was required.. Note that the units were cast on their side so no allowance was required for fresh concrete depth beneath the reinforcement, and that the units were designed using a draft of amendment in which the maximum bar diameter allowed was 80% of that given in equation 7-14 (not 70% as is the case in the final amendment). For reasons of practicality the column depth of unit 1 was reduced to 800 mm. This value is close to what would have been required before amendment 3 to NZS 3101:1995 [8] was issued.

Loading sequence
Units 1-3B were tested in the University of Auckland Civil Engineering test hall. The units were instrumented extensively with portal displacement gauges. Of special note are gauges A, B, C and D as marked in Figure 5. These measured movement of the beam longitudinal reinforcement relative to the column reinforcement, which provides a measure of beam reinforcement slip within the joint zone. Displacement at the load points was measured using turnpot gauges. Cyclic loading was applied by two double acting hydraulic actuators mounted at the beam ends, and the units were restrained by steel frames and single acting hydraulic actuators at the column ends. The gauge setout and loading details can be seen in Figure 5. 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 half-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 6) was applied as planned for units 2B and 3B.

TEST OBSERVATIONS AND RESULTS

Stiffness of beam-column joint units
From the first half-cycles of the tests it was determined that the lateral drift of the units at first yield 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 [12], as did the drift at first yield of unit 3B, which was 1.44%. Using standard moment-area theory, and assuming a linear force-displacement relationship up to first

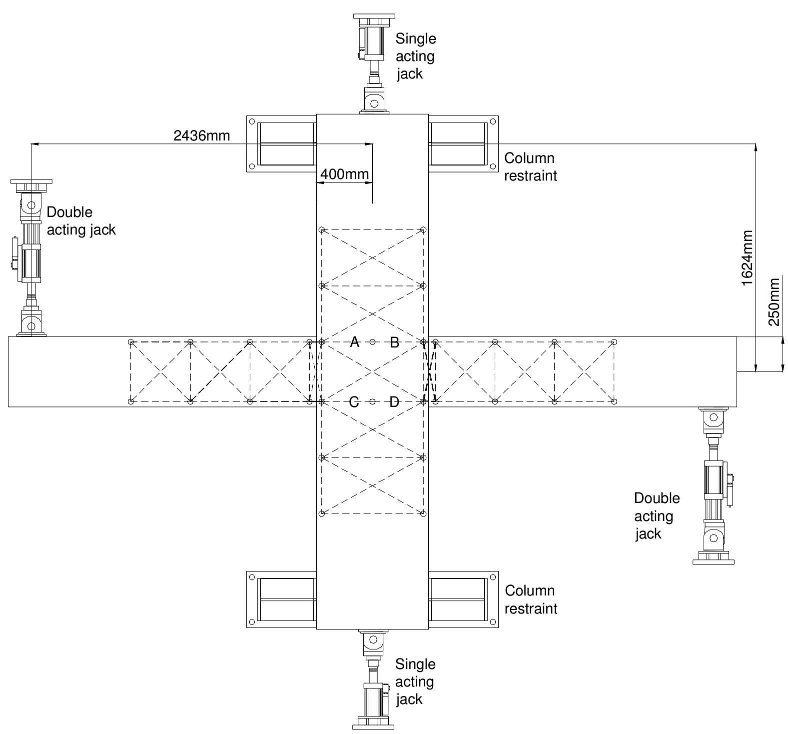

Figure 5 Portal gauge layout and loading arrangement for unit 1-3B.

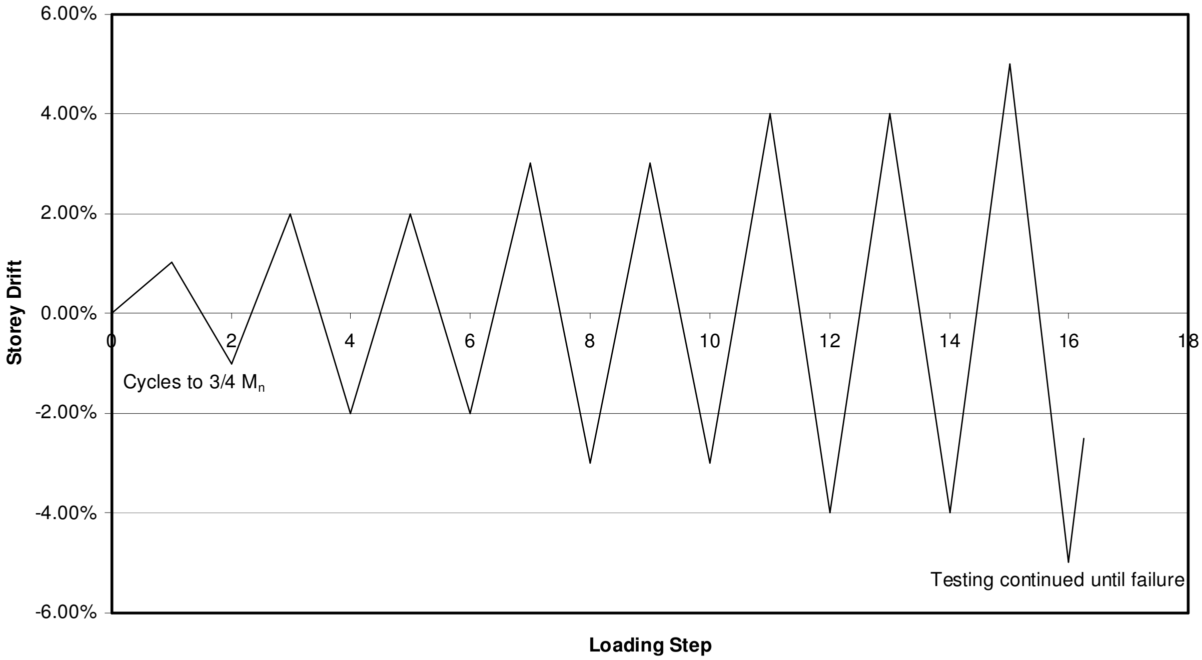

Figure 6 Planned loading cycle for units 1-3B

yield, it is straightforward to evaluate the effective moment of inertia of the beams based on the force and displacement at yield (due to beam shear and flexure only). The ratio of effective to gross moment of inertia is:

$$\frac{I_e}{I_g} = \frac{FL^3}{3E_c I_g} \cdot \frac{1}{\delta} \quad (2)$$

where *F* is the force, *L* is the length from the force application point 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 [8], and I_g is the gross section moment of inertia. This calculation is summarised in Table 4. The effective moment of inertia measured for the units is notably higher than the value of $0.32I_g$ suggested in the amendment 3 of NZS 3101:1995. However the NZS 3101:1995 amendment 3 effective moment of inertia values are for beams with more prototypical lower reinforcement ratios (p~0.7%) than that of units 1-3B (p=1.6%).

Accuracy of strength prediction

The measured strength of unit 1B prior to strain hardening was 7.5 percent greater than the value calculated using the measured material properties. (see Figure 7). This may be a result of the problems encountered during the testing of unit 1B

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	2044	19.7	31.2	25445	2.083E+09	0.45
	Right	165.4	2037	19.9	31.2	25445	2.083E+09	0.44
2B	Left	172.8	2031	19.5	40.6	28054	2.083E+09	0.42
	Right	160.2	2052	20.6	40.6	28054	2.083E+09	0.38
3B	Left	161.7	2100	24.8	44.8	29109	2.083E+09	0.33
	Right	166.7	2100	19.8	44.8	29109	2.083E+09	0.43

*Beam shear and flexural displacement components only

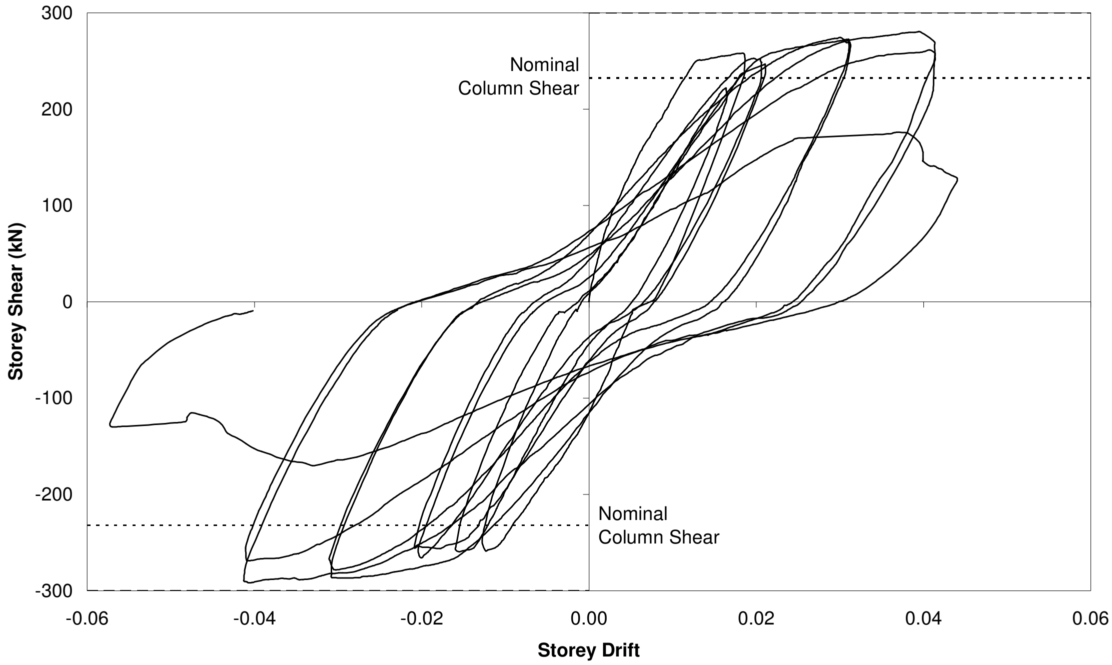

Figure 7 Unit 1B storey column shear versus storey lateral drift.

which lead to the unit yielding during the first "elastic" cycle. The actual yield strengths of unit 2B and 3B were more closely matched to the predicted strength, being less than 5% greater during the first half-cycle (see Figure 8 and Figure 9).

Damage patterns

The damage sustained by units 1-3B 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 diagonal 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

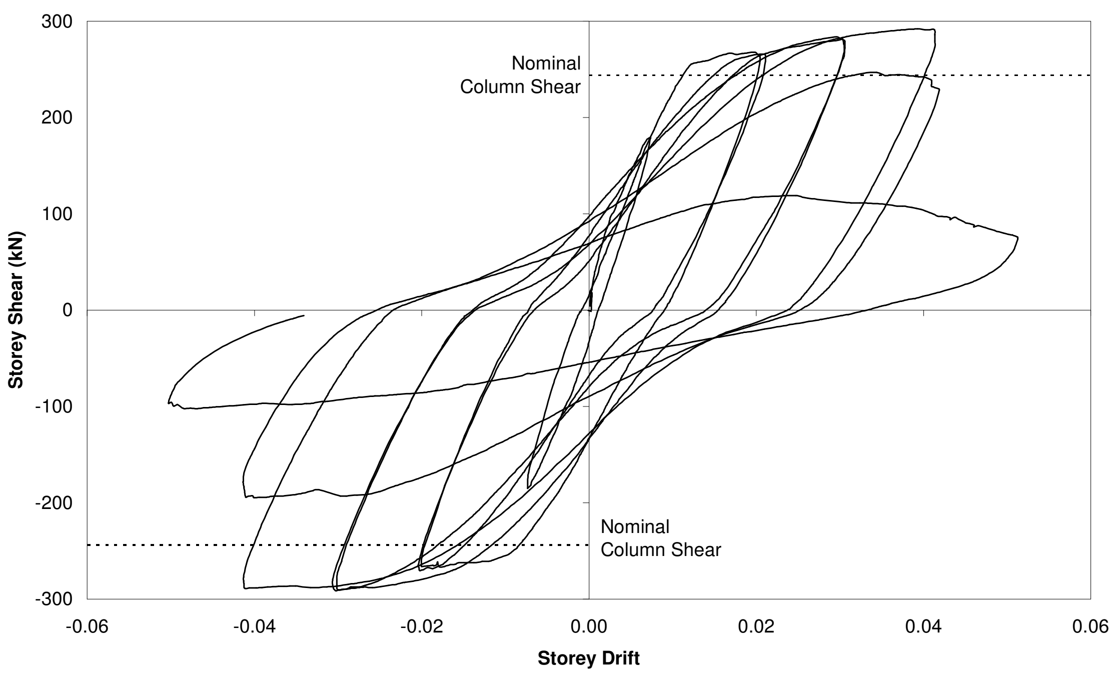

Figure 8 Unit 2B storey column shear versus storey lateral drift.

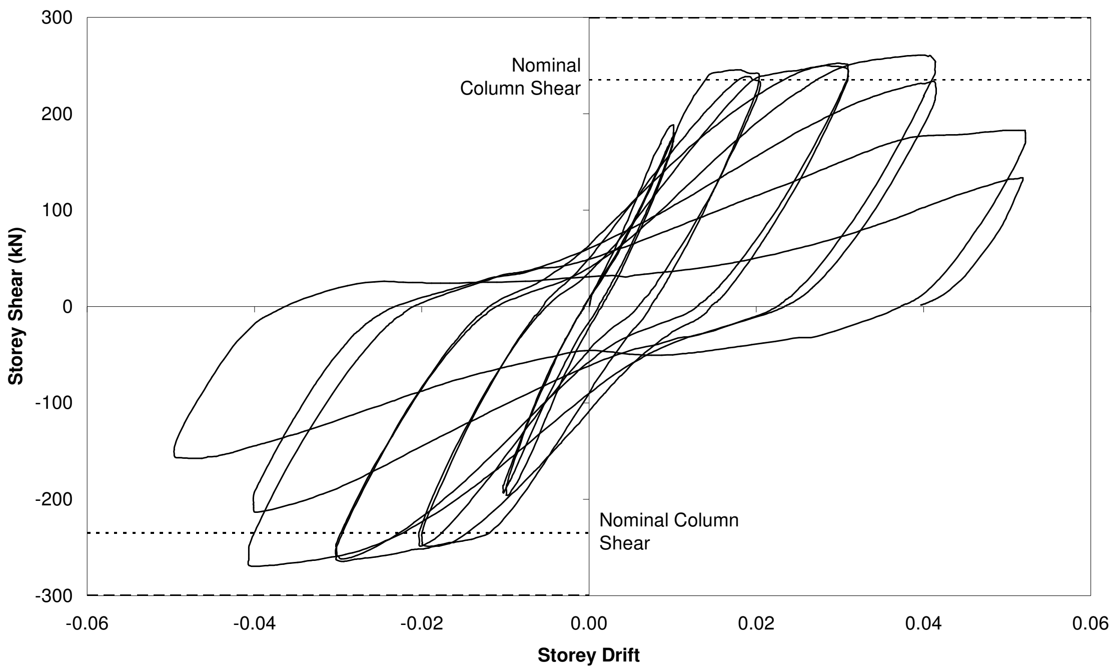

Figure 9 Unit 3B storey column shear versus storey lateral drift.

the drift was 3%. Unit 3B was the exception to this statement. Damage continued to increase in the joint region until the conclusion of the test. This was clearly associated with bond failure occurring within the joint.

Bond Performance

In contrast to the earlier tests at the University of Auckland neither unit 1B nor 2B showed signs of bond failure in the joint region, despite not conforming with the most recent amendment to NZS 3101:1995 [8] when actual material properties were used in the calculations (see Table 3). 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 units at low load levels remained similar throughout the test (see Figure 7 and Figure 8).

Unit 3B showed signs of bond failure during the cycles to 2% drift, with horizontal splitting cracks forming in the joint region. The data recorded from the gauges measuring bar slip during the test indicate that significant slip first occurred during the cycles to 4% drift, and that the maximum slip measured before the gauge mounting points were broken off the reinforcing bars was approximately 7.5 mm. Determination of bond failure using the criteria discussed in this paper (see Figure 1) indicates bond failure had occurred by the second half-cycle to 5% drift. This reinforces the view that bond failure occurs prior to significant strength loss. The strength loss due to bond failure was approximately 40% in the cycle following bar slip.

Failure Modes

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 [10], AS/NZS 1170 [13] and overseas codes of practice [14].

The strength achieved by unit 2B during the fourth half-cycle to 4% (in the negative direction) did not equal 80% of that developed during the second half-cycle (also in the negative direction), meaning that, as generally defined in New Zealand, failure occurred during this half-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 half-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.

As mentioned previously, bond failure occurred in unit 3B during the cycles to 4% drift. However, strength degradation did not exceed 20% until the first half-cycle to 5% drift. Given that the yield drift of unit 3B was 1.44%, the unit developed a ductility of 2.8. In contrast to the first two units no buckling of reinforcement was evident in unit 3B due to the reinforcement being free to slip through the joint zone. Testing was halted following the third half-cycle to 5% drift as it was evident that no further damage could be inflicted due to the low load sustained by the sub-assembly.

THE INFLUENCE OF VERTICAL JOINT SHEAR REINFORCEMENT ON BOND STRENGTH

It is thought that the improved response of units one and two was due to the large quantity of vertical shear reinforcement in the joint region, which appears to improve bond performance. Equation 11-7 of NZS 3101:1995 [8] 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 – respectively 63% and 114% more than required. The quantity of vertical joint reinforcement provided in unit three more closely matched that required by NZS 3101:1995 (603 kN strength provided compared to 462 kN strength required, 30% more than required).

The vertical joint shear reinforcement of eighteen of the beam-column joint tests included in the previously mentioned database has been analysed. Only four of these units contained sufficient vertical joint shear reinforcement according to NZS 3101:1995. The other fourteen joints lacked between 5% and 50% of the required joint shear reinforcement. Of the beam-column joints containing sufficient vertical shear reinforcement of the joint, only one showed signs of bond failure. This unit contained insufficient horizontal shear reinforcement of the joint. It seems that the shear reinforcement of the joint region plays a significant role in preventing bond failure. The full influence of vertical joint shear reinforcement on bond performance is currently being investigated.

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 [6].
- In previous beam-column joint tests with Grade 500 beam reinforcement, bond failure occurred between storey drifts of 1.7 and 4.5%. Testing of two units with HD25 reinforcement did not cause bond failure before buckling of the plastic hinge zones prevented further testing, despite having column depths that did not comply with the amended Standard.
- Bond failure and significant reinforcement slippage did occur during testing of the third unit, which contained less vertical shear reinforcement relative to New Zealand standard requirements.
- The ductility 1 inter-storey drift for all the sub-assembly units tested with Grade 500 reinforcement were in the range of 1.2% - 1.45% This indicates that to comply with limiting inter-storey drift limits when this reinforcement is used the structural ductility factor needs to be much smaller than that commonly assumed in design. The existing values were developed for lower grade reinforcement.
- The unexpectedly good bond performance of the first two test units is thought to have occurred because of the large quantity of vertical reinforcement in the joint region. This aspect of bond strength needs investigating in more detail.

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