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THE SEISMIC BEHAVIOUR OF REINFORCED CONCRETE BEAM-COLUMN KNEE JOINTS FOR BUILDINGS

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ABSTRACT

This paper describes the cyclic testing of two half-scale reinforced concrete beam-column building knee joints designed to the 1995 New Zealand Concrete Standard, NZS 3101 (1995). The two knee joints were identical, except that one had a standard hook detail for the beam bottom bars and column internal bars while the other unit had beam and column U-bars in the joint region. Both units approached their nominal strengths under both opening and closing bending moments. The hooked unit developed a joint shear failure at displacement ductilities greater than 4, while the U-bar unit was able to form a reversing beam plastic hinge with little joint deterioration, although some joint cover concrete was lost. The maximum levels of joint shear sustained in these two units approached 0.1 f'_c MPa, this being only half of the limiting joint shear stress specified in the NZ Concrete Standard.

KEYWORDS

Reinforced concrete; knee joints; beam-column joints; anchorage; joint shear; cyclic behaviour.

INTRODUCTION

Although researchers have studied the behaviour of reinforced concrete knee joints for buildings under monotonic loading (either closing or opening bending moments), very little research effort has considered the cyclic behaviour of knee joints under the expected seismic conditions. Cote and Wallace (1994), McConnell and Wallace (1994) and Mazzoni, Moehle and Thewalt (1991) have recently studied the cyclic response of reinforced concrete knee joints for buildings designed to U.S. Standards, and Ingham, Priestley and Seible (1994a,b) have investigated the inplane cyclic response of bridge knee joints. Cote and Wallace tested 406 x 229 mm (16" x 9") beams framing into 406 mm (16") square columns. The beam's bottom bars had a low reinforcement ratio of about 0.5% and the governing joint confinement regulations (ACI 318-1989) meant the volume of transverse ties were approximately double that now specified in the 1995 NZ Concrete Standard. Vertical joint shear reinforcement was provided by four U bars anchored into the column. Two of their units with standard 90-degree hook anchorage details exceeded the nominal opening moment by approximately 5%, but this only occurred at 4% drift. Maximum joint shear stresses during testing were of the order of 0.05 f'_c. The joints exhibited "stable" hysteretic behaviour up to ductility factors of 4 and 2 under opening and closing moments respectively.

McConnell and Wallace (1994) extended the testing programme discussed above by considering the use of T-headed bars for anchorage within the knee joint, as well as testing several specimens having conventional standard 90-degree hook anchorages. Some beams were widened to 279 mm (11") and the bottom beam reinforcing ratio was increased to about 0.8%. These units failed to reach their nominal moment strength in each direction (for example Joint KJ1 reached 88% under closing moment, 82% under opening moment) and diagonal tension splitting failures occurred in several joints, where an average joint shear stress of $0.6\sqrt{f'_c}$

MPa was reached. They concluded that the shear stress limit for knee joints should be about $0.5\sqrt{f'_c}$ MPa (that is half of the specified limit in ACI 352-91 (1991)).

Mazzoni et al (1991) tested two knee joints (305 by 254 mm beams framing into similar sized columns) with a larger beam steel ratio of 1.3%. Again standard 90-degree hook anchorages were used for both beam and column reinforcing. Two or four rectangular ties were used as horizontal joint shear and confining steel, while the only vertical shear steel in the joint was two intermediate column bars. Both units experienced loss of anchorage of the column hooked bars followed by disintegration of the joint core. Opening moment strengths reached only 54% and 60% of the nominal strength. The maximum joint shear stress sustained was $0.77\sqrt{f'_c}$ under closing moment conditions, although the units were designed to carry the limiting stress of $1.0\sqrt{f'_c}$. These three series of tests showed that joint shear failures occurred even when ACI Standard requirements for joint confinement were adhered to and formed the basis for the tests described in this paper.

DESIGN OF TEST UNITS

Although there have been many series of tests on interior and exterior beam-column joints in NZ over the last 25 years, no testing has been completed on the cyclic behaviour of knee joints.

The new NZ Concrete Standard, NZS 3101 (1995) allows a greater than 50% reduction in horizontal joint ties (A_{jh}) required over that specified in the previous Standard (NZS 3101:1982). This reduction applied to joints of 1-way frames with low column axial loads and horizontal joint shear stresses, v_{jh}, less than $0.14\ f'_c$. These conditions apply to the knee joints tested in the current series. However the new Standard specifies an increase in vertical joint shear reinforcement of 20% over the previous standard. Usually this vertical joint steel is provided by intermediate column bars along the two sides of the joint in the plane of the beam(s) under consideration. In the two tests described here the vertical joint reinforcement A_{jv} was provided by a single D10 (deformed 10 mm diameter) U-bar anchored into the column, see Fig. 1. Six 4ϕ tie-sets provided the Standard's requirements for the horizontal joint ties. These drawn wire ties were heat-treated to give a yield stress of about 300 MPa; 4 mm mild steel rod being unavailable.

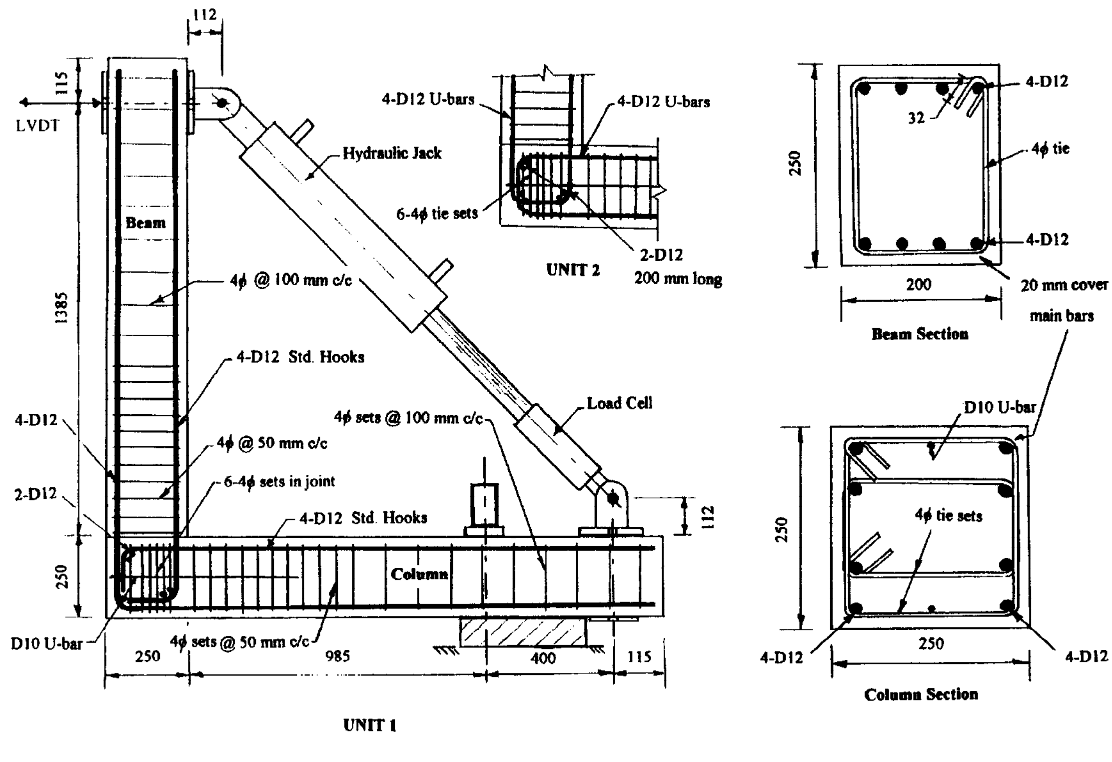


Fig. 1 Knee Joints 1 and 2 showing loading set up.

The joints were designed as half-scale units and 4-D12 bars with a specified yield stress of 300 MPa were used as both the top and bottom beam reinforcing. The 250 mm square column had the same principal reinforcement as the beam, as shown in Fig. 1. The only difference between the two units was the anchorage detail of the principal beam and column bars. In the first unit standard 90-degree hooks with a straight anchorage length of 12 d_b were used, while in the second unit the beam and column bars were U-bars, the beam bars positioned inside the column bars.

For exterior joints the NZ Standard specifies measuring the hook development length (l_{dh}) from a point 8 beam bar diameters in from the beam-column face. (The other specified possibility of beginning l_{dh} at the column centreline position was not critical here). This requirement allows for the observed behaviour of yield penetration into the joint when the reversing beam hinge forms near the column face. The design formula gave l_{dh} = 137 mm in this case but a minimum value of 150 mm dictated here. Thus the horizontal length required before the 90 degree bend was 246 mm, greater than the space provided in the 250 mm deep column. However the Standard allows a 20% reduction in l_{dh} when two transverse bars of at least the same diameter as the beam bars are provided inside the 90-degree bends, although only 1 transverse bar could be placed in the first unit's outside corner due to congestion in the joint.

The beam was designed to hinge near the column face with a hinge length of two beam depths, and the transverse reinforcement was designed accordingly. The column region near the joint was designed to the current confinement and shear requirements of a "potential plastic hinge". Both units were cast on their sides with 25 MPa concrete having a maximum aggregate of 10 mm.

TEST SETUP and RESULTS

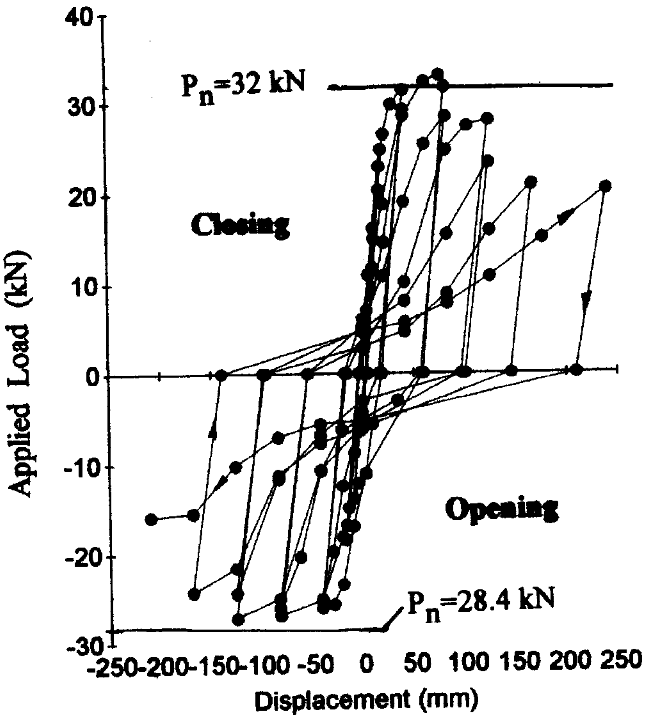
Figure 1 shows the test setup, which incorporated a hydraulic jack at 45 degrees allowing a small reversing axial load in both the beam and column. The column end was stressed down to the test floor. Deformations along the units were measured using portal frame gauges attached to 6 mm ϕ studs welded to the principal reinforcing bars. These studs had a gap around them through the concrete cover of 5 mm.

As is usual in NZ sub-assemblage tests, the units were loaded through two "elastic" cycles to three quarter yield level and then two cycles to a displacement ductility of ±2 (42 mm beam tip deflection) in both directions. Subsequent double cycles to ductilities of ±4, ±6 and ±8 were performed, with the final cycle being at the limit of the jack's extension (about a displacement ductility of 10). The first yield displacement at the beam's load application position (21 mm) was estimated by extrapolating the actual displacement at the ¾ yield force level.

Load Deflection Plots

Although both units began developing plastic beam hinges, in subsequent cycles the standard hook unit failed in the joint while the U-bar unit formed a substantial beam hinge with some loss of joint cover concrete. Figures 2 and 3 show the applied load plotted against the beam deflection measured by LVDT at the position of applied load for Units 1 and 2 respectively.

In Unit 1 (standard hooks) the maximum moment exceeded the nominal beam moment by 3% under closing moment at the first cycle to ductility factor, μ=-4 and the applied moment only reached about 95% of the nominal opening moment in the first cycle at μ=+6. As testing continued the sustained moments progressively reduced until during the first cycle at μ=-8 the closing and opening moments had reduced to about 57% and 75% respectively of their nominal values. During the ductility 4 cycles much of the joint cover concrete was loose and the cover on the back of the joint had fallen off. This loss reduced the effective lever arm by up to 15% for the section, especially under closing moments, and it is this reduction that explains why M_n was never reached under opening moments.



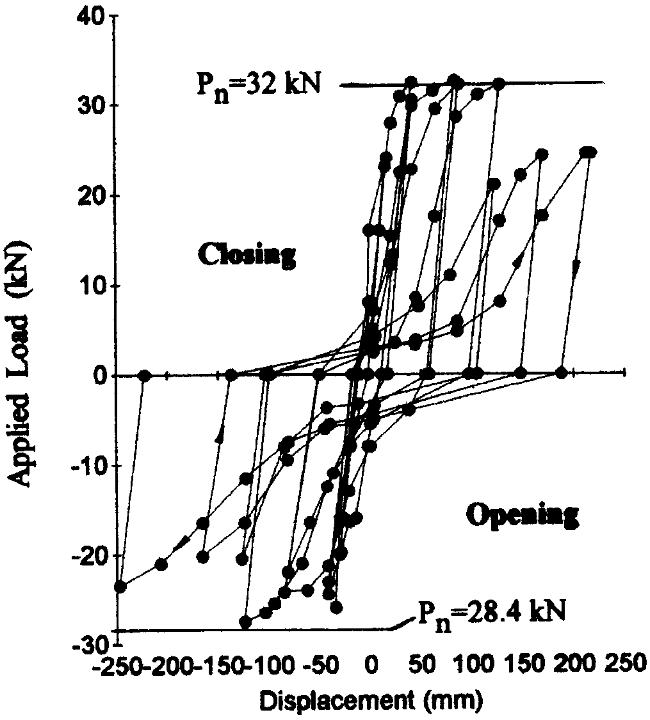


Fig. 2 Load Displacement, Knee Joint 1

Fig. 3. Load Displacement, Knee Joint 2

Unit 2 with U-bars attained a closing moment nearly 2% larger than the beam's nominal moment under the corresponding low axial compression, at the first cycle at μ=-6. Strength retention was better with this unit than the unit with standard hooks, but there were substantial strength drop offs to 66% and 72% of the closing and opening M_n values, respectively, in the second cycle to ductility factor 6. It was in this cycle that a second series of cracks opened in the joint and shear deformations increased relative to flexural beam deflections (see following Section). In the later cycles the concrete cover at the beam-joint interface had fallen off and the beam's effective depth had reduced to a length of about 175 mm; less than the distance between the centroids of the top and bottom beam bars.

Flexure and Shear Deformations

From the portal gauges along the principal beam and column bars it was possible to calculate the flexural and axial deformations developed during testing. Likewise the diagonal gauges allowed the calculation of the shear distortions in the joint, beam hinge zone and column zones adjacent to the joint. Figs. 4 and 5 show the accumulated flexural and shear deflections in Units 1 and 2 respectively, measured at cycle peaks. These have been calculated as horizontal deflections at the point where the beam load was applied. Also plotted is the actual beam deflection at the same position measured by the LVDT at each cycle peak. The striking difference between the two Figures is that the shear deformations increase steadily during the Unit 1 test and become a larger proportion of the total deformation, but in Unit 2 the shear deformations remain relatively constant while the flexural deflections, due mainly to the beam hinge, continue to increase at the larger ductilities. This difference in deformation modes can be seen in Figures 6 and 7, which show both units at ductility factor 10 at test completion with the portal gauges removed.

Beam Bar Anchorage

The standard hook detail was not very satisfactory during testing and after the joint cover fell off the 90-degree bend and tail tended to open, thus eliminating the bond along the $12d_b$ length. On the other hand the U-bar detail behaved much better and was not able to bend out during testing. The D12 transverse bars positioned in the 90-degree bends performed admirably and were being pushed diagonally inwards by the bend in the beam and column bars allowing the diagonal joint strut to form after many cycles in Unit 2.

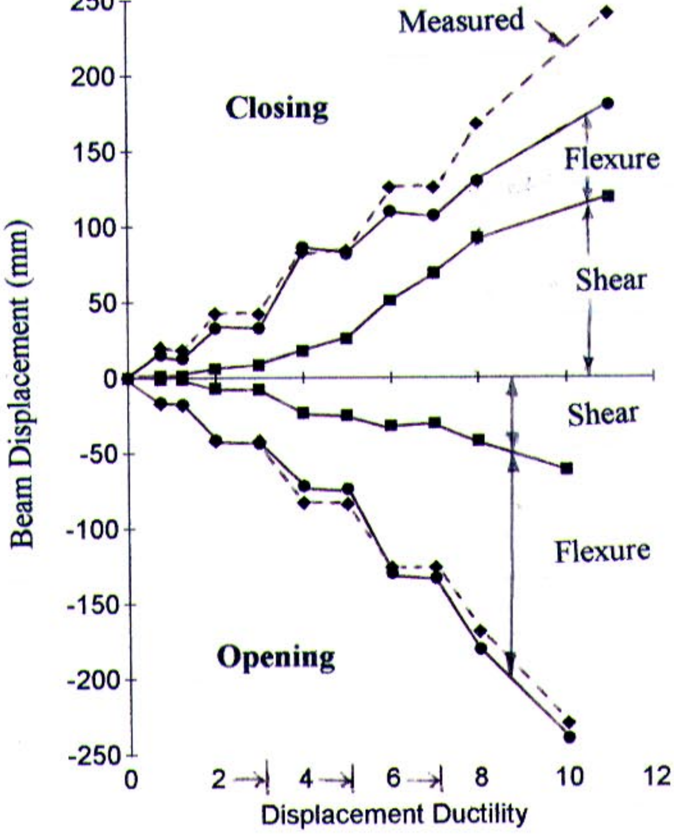


Fig. 4 Flexure and Shear Deflections
Knee Joint 1

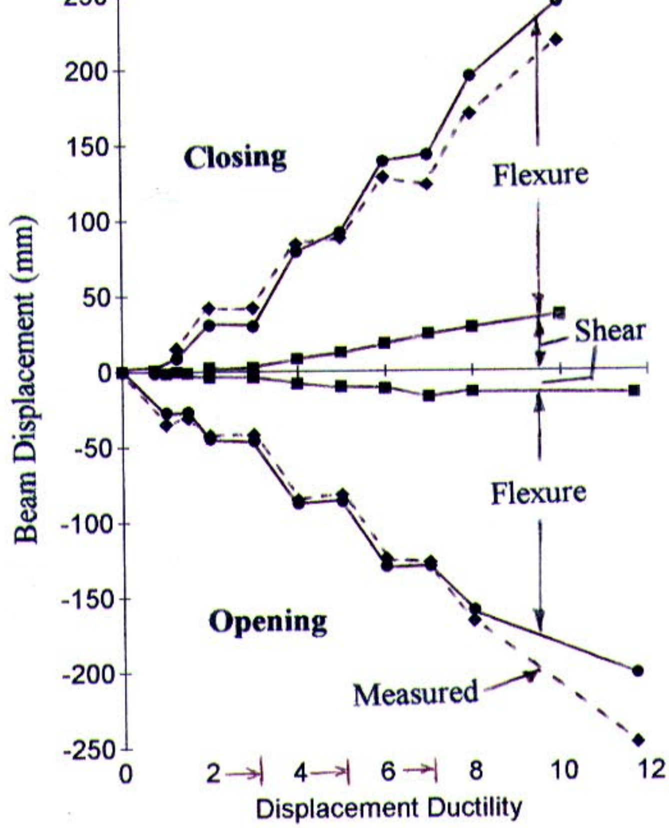


Fig. 5 Flexure and Shear Deflections
Knee Joint 2

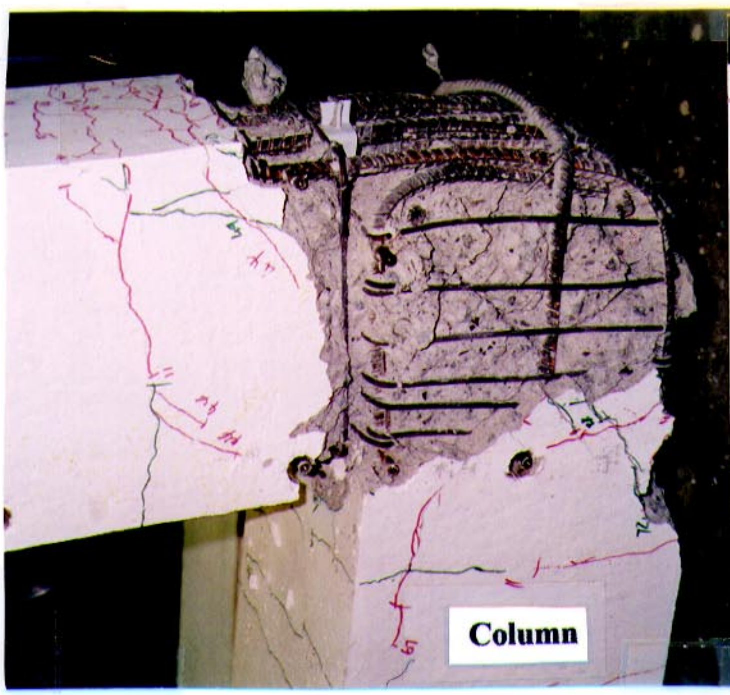


Fig. 6 Knee Joint 1 with Standard
hooks at ductility 10

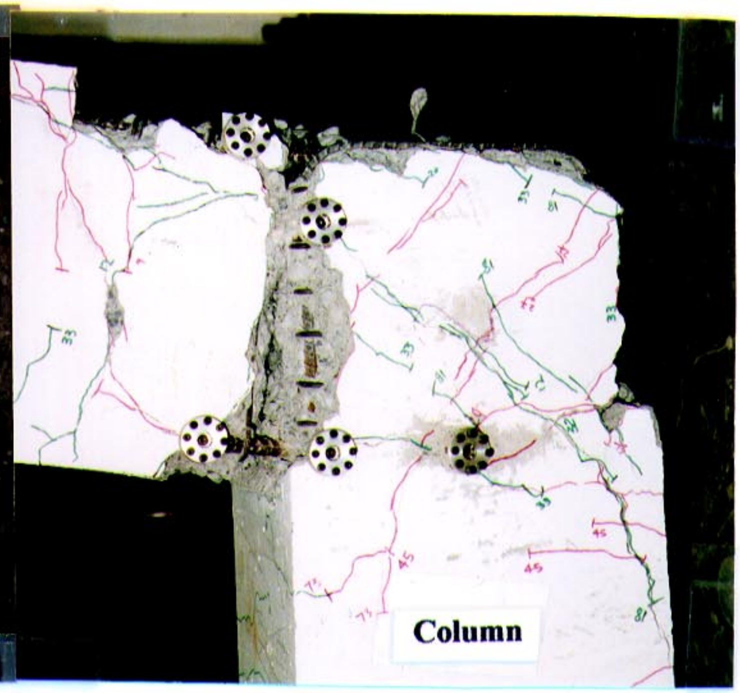


Fig. 7 Knee Joint 2 with U-bars
at ductility 10

Although the transverse bars were attempting to initiate the strut formation in Unit 1 at high ductility factors, a substantial strut was not possible due to the highly cracked and ruptured joint core concrete.

TABLE 1

Test		$p = \frac{A_s}{bd}\%$	$\frac{M_{test}}{M_n}$	$\frac{A_{jh}\text{ (actual)}}{A_{jh}\text{ (code)}}$ [1]	$\frac{A_{jv}\text{ (actual)}}{A_{jv}\text{ (code)}}$ [2]	$\frac{pf_y}{\sqrt{f'_c}}$	Max joint shear, v_{jh}
Megget	closing	0.99	1.03	1.01	0.91	0.67	$0.094 f'_c$
Unit 1 std. hooks	opening	0.99	0.95	0.97	0.86	0.67	$0.076 f'_c$
Megget	closing	0.99	1.02	1.01	0.91	0.67	$0.093 f'_c$
Unit 2 U-bars	opening	0.99	0.97	0.97	0.86	0.67	$0.077 f'_c$
Mazzoni 1	closing	1.325	0.78	0.58	1.75^3	1.03	$0.101 f'_c$
std. hooks 2-9.5ϕ ties	opening	1.325	0.54	0.52	1.47^3	1.03	$0.053 f'_c$
Mazzoni 2	closing	1.325	0.79	1.16	1.75^3	1.03	$0.102 f'_c$
std. hooks 4-9.5ϕ ties	opening	1.325	0.60	1.04	1.47^3	1.03	$0.059 f'_c$
Cote & Wallace	closing	0.912	1.03	2.04	4.00	0.61	$0.049 f'_c$
KJ1 3-9.5ϕ ties	opening	0.456	1.04	1.90	3.56	0.30	$0.011 f'_c$
McConnell & Wallace	closing	1.388	0.88	1.25	1.83	1.10	$0.125 f'_c$
KJ7	opening	0.833	0.82	1.18	1.66	0.66	$0.046 f'_c$

Notes: 1. A_{jh} (code) calculated by NZS 3101:95
2. A_{jv} (code) calculated by NZS 3101:95
3. A_{jv} are intermediate column bars not anchored at top of joint.

DISCUSSION

Table 1 presents the relevant ratios of test efficiency (M_{test}/M_n), actual to code amounts of horizontal and vertical joint ties calculated using the latest NZ Standard (1995), the material ratios ($\rho f_y/\sqrt{f'_c}$) and the maximum joint shear stresses measured during the recent tests completed in the US and NZ. Although closing nominal beam strength was reached in three of the tests, full opening strength was only reached in 1 test (Cote & Wallace) and that was in a joint with considerably more joint shear ties than specified in NZS 3101 (1995). The two tests described here were close to the nominal moment under the maximum opening force, but both had large strength degradation in later cycles due mainly to joint deterioration. The author has suggested in an earlier paper (Megget 1994) that the material ratio is a good measure of how strength efficient a knee joint is likely to be, especially under opening moments. The NZ Standard (1995) recommends that $\rho f_y/\sqrt{f'_c} \leq 0.5$ if full moment strength is required and these tests enhance that view, see Table 1. Both Mazzoni's tests had material ratios twice the recommended limit and both failed to reach their nominal moment by large margins. The other tests, with material ratios of about 0.6, got close to full moment strength (as detailed in Table 1). Hakuto, Park and Tanaka (1995) have suggested that the maximum joint shear for unreinforced exterior beam-column joints, with little or no joint ties, be about 0.17 f'_c or 1.0 $\sqrt{f'_c}$ MPa (the maximum value for v_{jh} in NZS 3101 is 0.2 f'_c). However, the tests described here, and those reported by other authors and described by Megget (1994), show that for building knee joints a limit of 0.1 f'_c or 0.5 $\sqrt{f'c}$ MPa seems more appropriate. Figure 8 shows the maximum horizontal joint shear stress sustained against f'_c for all the knee joint tests known to the authors with the proposed equations for v_{jh} plotted. Note that many of the earlier tests had monotonically applied opening bending moments, rather than cyclic loading.

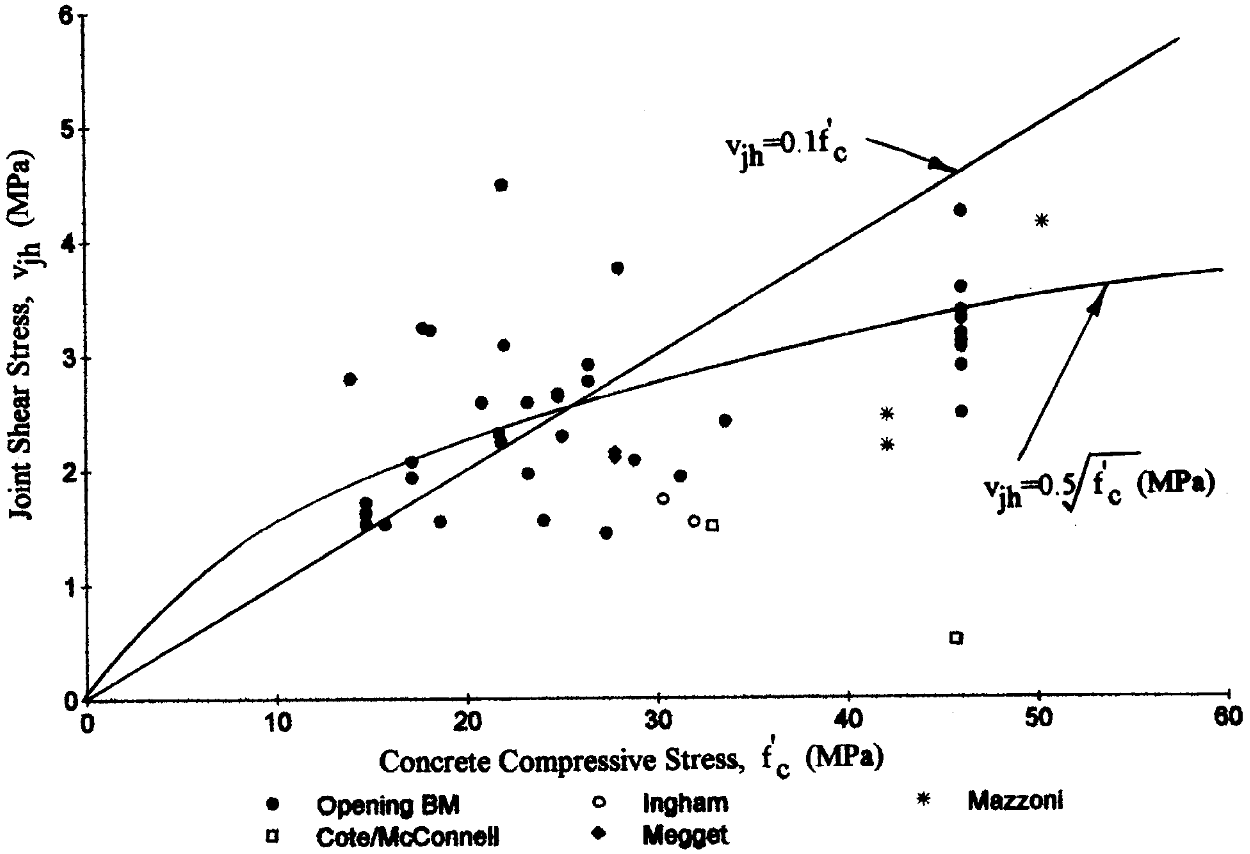


Fig. 8 Maximum Horizontal Joint Shear Stress Versus Concrete Compressive Strength for Knee Joint Tests (Opening and Cyclic tests)

CONCLUSIONS

1. Small knee joints have a better cyclic behaviour when the beam and column bar anchorage is by means of U-bars rather than standard 90-degree hooks.
2. Even with substantial horizontal and vertical joint ties designed to the latest standards, joint shear failures can still occur.
3. A maximum joint shear stress of 0.1 f'_c or 0.5 $\sqrt{f'c}$ MPa is recommended for knee joints designed for buildings.
4. If full beam or column strength is a design requirement then the material ratio ($\rho f_y/\sqrt{f'_c}$) should not exceed a value of 0.5. For members of knee joints with higher values, joint shear failures are almost inevitable.
5. Transverse bars within the 90-degree anchorage bends in joints help considerably in the formation and continuance of a diagonal joint strut.

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