THE SEISMIC BEHAVIOUR OF SMALL REINFORCED CONCRETE BEAM-COLUMN KNEE JOINTS

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SUMMARY

The majority of research into beam-column knee joints has been conducted with monotonic loading. Many of these joints failed to reach their member moment capacity, especially under opening moments, while a few cyclic knee joint tests have been completed in the United States this decade. This paper describes the cyclic testing of 8 small knee joints designed to the 1995 New Zealand Concrete Standard. In addition two joints designed and detailed to the 1965 N.Z. Concrete Code were also tested. Joints with U-bar anchorages performed better than joints with standard 90 degree hook details on beam and column bars. The current Concrete Standard (NZS3101:1995) designs usually attained their nominal moment capacity in both directions up to and including ductility 4 displacements, but subsequently strengths fell off at higher ductilities. Joints with extra diagonal bars across the inner corner were able to sustain their nominal member strengths to higher ductility levels, especially under opening moments. A maximum horizontal joint shear stress of $0.12 f_c^{'}$ for knee joints, in ductile frame buildings is recommended, where this limit is 60% of the current NZS3101:1995 Standard recommendation. An approximate 25% degradation of the joint shear stress occurred as displacement ductility factors increased from 1 to 8. The 1960's designed joints behaved poorly, as expected, with joint shear and anchorage failures occurring, in both moment directions, at strength levels below the beam's nominal strength. A maximum joint shear stress of only $0.072 f_c^{'}$ was reached and this fell to about a third of that stress between displacement ductility factors of 1 and 4 under closing moments.

1. INTRODUCTION

1.1 Monotonic tests

Although considerable research effort has been concentrated on the seismic design of interior and exterior beam-column joints of ductile frames since the late 1960's, investigation into the seismic performance of knee joints, found at the top of multi-storey frames or in portal frames, has been minimal. During the late 1960's and 70's there was considerable research on small knee joints completed in Europe, but the majority of the testing was under monotonic loading (either opening or closing actions). Many differing anchorage and joint snear tie details were tested but generally the knee joints behaved poorly, especially under opening (positive) bending moments. In many cases the joints failed to reach their nominal member strengths before failing in the joint region, due either to shear (diagonal tension) or loss of anchorage to the beam and/or column bars. Loss of cover from the outside of the corner was often a pre-requisite to anchorage loss and subsequent joint failure. Many of these joints contained little or no transverse joint ties, either horizontally or vertically (Nilsson [1], Mayfield [2, 3] and Skettrup [4]). The addition of joint ties usually resulted in an improvement in joint strength but did not guarantee enough concrete joint confinement or shear strength to allow nominal beam or column strengths to be attained. This was especially so for beams or columns with large reinforcement ratios. The author studied these early opening moment tests [5] and concluded that if attainment of the nominal member strength was required then the beam or column reinforcement ratio, $p = A_s / bd$, where A_s is the tensile reinforcing area, b and d were the beam's width and effective depth, would need to be less than about $0.5\sqrt{f_c^{'}} / f_y$, where $f_c^{'}$ and f_y are the reinforcing yield and 28-day concrete compressive cylinder strengths, respectively.

In fact, this is the same "conservative" limit recommended in the 1982 New Zealand Concrete Code Commentary (NZS3101:1982) [6], for small knee joints under closing moments with no transverse joint ties, for the concrete to

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resist the diagonal tension forces. The suggested detailing for opening joints was to provide radial hoops to resist the whole of the diagonal tension across the corner.

Bari [7] and Fenwick tested nine knee joints under opening moments, which were not shear reinforced with ties in the joints. They found that by adding diagonal bars across the joint's inner corner, within a small fillet, the beam's nominal strength could be reached. This was due mainly to the critical section being moved away from the column face to the section at the end of the fillet, as well as reducing the shear stress across the joint, due to the larger effective joint cross-section.

The other conclusion reached from all the previous opening moment knee joints was that the maximum sustainable diagonal tension stress in unreinforced (no transverse bars and/or ties) knee joints was about $0.4\sqrt{f_c'}$ (MPa) or about $0.07 f_c'$, a limit suggested independently by Priestley [8].

While many anchorage arrangements have been tested monotonically, the predominant two details examined have been U-bars, in which the tension steel becomes the compression steel when it exits the joint, and the other incorporating 90 degree hooks *out* of the joint zone. This latter detail has not been acceptable for seismically loaded joints for three decades in New Zealand, as the bend out of the joint does not contribute to the development of the concrete compression strut needed to resist the high joint shears. The U-bar arrangement allowed larger strengths to be reached in the joint under opening moments, when compared with the other tested details, many of which were impractical to construct [5].

1.2 Previous Cyclic Knee Joint Tests

The first "modern" cyclic tests on small scale building knee joints were completed by Mazzoni, Moehle and Thewalt [9]. Two knee joints were tested and subsequently the second joint was retrofitted and retested. Both beam and column sections were 305 mm deep by 254 mm wide with 3-No 6 bars (19 mm diameter) top and bottom in the beam, ($p = p' =$ 1.33%). The first unit had only 2 - 9.5 mm diameter horizontal ties within the joint region, (twice the recommended quantity), while unit 2 had four such ties. These ties were equivalent to 52 and 104% of the current N.Z. Concrete Standard, *NZS3101 (1995)* [10] horizontal joint tie requirements. The vertical joint steel was 2 - 19 mm diameter column bars through the centre of the joints (1 bar per in-plane column side). It was assumed that these bars had a standard hook at the top of the column anchored in the cover concrete above the beam bars and were therefore ineffective as vertical joint steel (no details given). The beam and column bars were anchored with standard 90-degree hooks within the joint. Ties had the conventional American Concrete Institute (ACI) anchorage detail of a 135-degree hook one end and a 90-degree hook at the other. This tie anchorage arrangement is not permissible in the N.Z. Standard [10].

Both joints failed to reach their theoretical beam strength; maximum strength ratios (test moment/nominal beam moment, M_{test}/M_n) of 60 and 79% being sustained under opening and closing actions, respectively for the 4-hoop joint. The 2 hoop joint's strength ratios were less, 54% under opening and 78% under closing moment. Failure occurred in the joint zone in both tests, due mainly to splitting of the joint concrete on the outer faces which "resulted in the loss of effective joint and beam cross-sections as well as deterioration in the anchorage condition for the column and beam reinforcement." The absence of any transverse joint ties across the top and down the sides of the column would have exacerbated the joint's failure. The continuing drop off in strength sustained at higher ductility factors was more predominant under closing actions, due to the anchorage loss of the top beam bars and the outer column bars. The beam top and bottom covers were large at 41 mm (1.625 inch) for these small beams and the loss of cover would have decreased the section capacity considerably.

The retrofit to the 4-hoop joint included inserting 2-No 3 (9.5 mm diameter) U-bars vertically into the joint with 305 mm development lengths into the column and the addition of 3-No 4 (12.7 mm) diagonal bars with 180 degree hooks positioned across the re-entrant corner. The reason for the diagonal bars was to improve the tensile transfer across the joint under opening actions. The amount of cross-sectional area was based on the recommendations of Nilsson & Losberg [11], that the area of diagonal bars be about a half of the beam's tension steel area. These bars were unable to be positioned at the optimal 45-degrees, due to construction difficulties, and were fixed at about 30-degrees to the beam's axis. The concrete within the joint and for a length of 300 mm along the beam and column was removed and recast.

The retrofitted joint was tested to the same loading programme as previously and formed a plastic hinge in the beam with little damage to the joint. The moment strength ratio (M_{test}/M_n) increased markedly to 1.12 and 0.98 under opening and closing moments, respectively and the observed maximum joint shears were greater than the expected maximums. The effect of the new diagonal bars on the beam strength at the column face section was not included in the nominal moment calculations. The joint continued to sustain moments larger than nominal for several reversing cycles up to a displacement ductility of about 5 in both directions.

Cote and Wallace [12] tested four half-scale knee joints having 406 mm deep by 229 mm wide beams with 406 mm square columns. All 4 joints had the same principal beam and column reinforcing, namely 4-No 5 top bars and 2-No 5 (15.9 mm diameter) bottom bars in the beam and 4-No 6 bars (1 in each corner) and 4-No 5 bars (1 at each mid-side) in the columns. These bars were anchored with standard 90 degree hooks in the joint. The difference between the tested units was in the transverse joint steel fitted.

Units KJ1, KJ2 and KJ4 all had 4-No 3 (9.5 mm) ties horizontally and 4-No 3 U-bar stirrups vertically in the joint region. However from the sketches in Cote & Wallace, it appears that only 3 horizontal ties were positioned between the top and bottom beam bars. The legs of the vertical ties in KJ1 and 2 extended a development length, L_d into the column ending with 135 degree hooks. In the other two joints the end of vertical U bar's tails extended only $1.5L_d$ beyond the beam centreline, with no hooks. Unit KJ2 had an additional 2-No 3 diagonal bars across the re-entrant corner. Joint KJ3 was identical to KJ4 except that only 2 horizontal ties and 2 vertical U-bars were provided in the joint.

The column and joint ties were anchored with conventional 135 degree hooks. However the beam ties comprised U-ties with a 90 degree hook one end and a 135 degree hook the other, with a short top cross-tie with similar hooks. This detail, although popular in the US, for reasons of ease of construction, was prohibited in the 1982 NZ Standard [6]. The designs, except KJ3, fully complied with the then current American Concrete Institute (ACI) Concrete Code (1991) [13]. When compared to similar sized joints designed to NZS3101:1995, KJ1, 2 and 4 had 1.33 times the required amount of horizontal joint ties (A_{jh}) and about 4.2 times the required amount of vertical joint steel (A_{jv}).

All four joints were able to reach the beam's nominal strength in both directions but this only occurred at about 4% lateral drift (displacement ductility, μ, approximately 4), when strain-hardening in the beam bars occurred. Although the authors comment that joint KJ2 only reached a strength 3.3% greater than the beam's nominal moment, this was calculated assuming the diagonal bars contributed to the beam's bottom reinforcement. If the diagonals are neglected, which is more realistic, the joint's efficiency, M_{test}/M_n increased to 1.24 under opening actions. At 2% lateral drift ($\mu \cong 2$) the average joint efficiencies for the 4 joints were 92 and 96% under opening and closing moments, respectively. By ignoring the cover concrete at the beam-column intersection, which had almost completely spalled at $\mu >2$, the average opening efficiencies increased by about 7% at 2% lateral drift.

The vertical joint U-stirrups improved the strength ratio, especially under closing moments by carrying the diagonal tension forces across the joint. The average joint shear stresses, v_{jh}, attained in this series of tests were 20% and 55% of the maximum stress of $1.0\sqrt{f_c^{'}}$ (MPa) specified in ACI 352 Committee (1991) [14] for opening and closing moments, respectively. However the test shear stresses seem to have been calculated using the design concrete strength ($f_c^{'} = 27.6$ MPa) instead of the actual strength at testing of 45.7 MPa. Using the actual $f_c^{'}$ values decreased the maximum shear stress ratios to $0.155\sqrt{f_c^{'}}$ (MPa) and $0.43\sqrt{f_c^{'}}$ (MPa) under opening and closing moments, respectively.

This series of knee joints was continued by McConnell and Wallace [15, 16] and Wallace, McConnell and Gupta [17], and included conventional reinforcement details and T-headed bars used on the principal beam and column bars, instead of standard hooks. The aim of the conventional joints was to have enough principal reinforcement to allow the joint shear stresses to reach the maximum specified, $1.0\sqrt{f_c^{'}}$ (MPa) in the ACI Code. To fit more beam steel in, the beam was made 50 mm wider than the earlier joints.

Joint KJ7, with a top beam steel ratio of 1.39% and a bottom steel ratio of 0.83% failed to reach full strength in both directions and only sustained joint shear stresses of $0.261\sqrt{f_c^{'}}$ and $0.604\sqrt{f_c^{'}}$ (MPa) under opening and closing moments, respectively. This joint had the same transverse joint tie arrangements, as KJ1, 2, and 4.

Wallace *et al* [17] concluded that the limiting joint shear stress should be $0.67\sqrt{f_c^{'}}$ (MPa) for knee joints without transverse beams and that the $1.0\sqrt{f_c^{'}}$ (MPa) limit specified in the ACI Committee 352 (1991) recommendations for corner columns was unconservative. Table 1 gives a summary of the conventionally reinforced US knee joints tested since 1990, with separate maximum values of the joint shear stress shown for opening and closing actions.

2. DESIGN OF TEST SPECIMENS

2.1 Reinforced knee joints designed to 1995 NZ Concrete Standard

All the test units were designed to the current Concrete Standard (NZS3101:1995) [10] except for the two beam-column joints designed to the 1964 Model Building Bylaw (NZSS1900 Chap. 9.3, 1964) [18], described in Section 2.2. The aim was to test approximately half-scale knee joints using small bars to facilitate fabrication and testing in the University of Auckland's Test Hall. All units had beams which were 250 mm deep by 200 mm wide while the columns were 250 mm square. The lever arm from the applied load point to the column face was about 1385 mm, but this varied slightly from test to test. The total column length, including the joint zone was 1750 mm and the applied load points represented the approximate positions of the points of contraflexure under lateral seismic force conditions.

Beam-column knee joints designed in NZ, which may experience seismic loading, are treated in the same way as exterior beam-column joints in the 1995 Concrete Standard, where the column continues above the joint. The design equations for exterior joints in the 1995 version of the Concrete Standard (NZS3101:1995) are amendments to the 1984 Code equations using the combined diagonal strut and joint truss models developed by Park and Paulay [19]. The equation for the horizontal joint reinforcement is

$$A_{jh} = \frac{6v_{jh}}{f_c^{'}}\beta\left(0.7 - \frac{C_j N^*}{f_c^{'} A_g}\right)\frac{f_y}{f_{yh}} A_s \quad (1)$$

where v_{jh} is the joint horizontal shear stress, β is the ratio of compression beam reinforcement area to that of the tension steel *but* ≤ 1, C_j is the ratio considering bi-axial joint shear stresses = 1 here, N^* is the column axial force (negative if tensile), A_g is the gross column area, f_{yh} is the horizontal ties' yield stress and A_s is the area of beam tension reinforcement at the column face. The $6v_{jh} / f_c^{'}$ factor was added to the final version of the Standard after the first two knee joints had been designed. This factor has a

TABLE 1 OTHER RESEARCHER'S CYCLIC KNEE JOINT TESTS

	f_c' (MPa)	f_y (MPa)	Beam A_s, A_s'	Joint Ties	$\frac{M_{test}}{M_n}$	$\frac{v_{jh}}{f_c'}$	$\frac{v_{jh}}{\sqrt{f_c'}}$	$\rho f_y / \sqrt{f_c'}$
Mazzoni 1	**CLOSE** $f_c' = 42.1$ **OPEN**	503	3 #6 top & bottom	2 #3 ties horizontal	0.779 0.537	0.101 0.053	0.655 0.345	1.027
Mazzoni 2	**CLOSE** $f_c' = 42.1$ **OPEN**	503	3 #6 top & bottom	4 #3 ties horizontal	0.788 0.602	0.102 0.059	0.664 0.381	1.027
Mazzoni Retrofit	**CLOSE** $f_c' = 50.3$ **OPEN**	503	3 #6 top & bottom	4 #3ties horiz. +2 #3U vertical +3 #4 diagonal	0.98 1.12	0.170 0.145	0.79 0.67	0.940
McConnell Wallace KJ7	**CLOSE** $f_c' = 32.85$ **OPEN**	455	5 #6 top 3 #6 bottom	4 #3 ties horizontal 4 #3U vertical	0.879 0.816	0.105 0.045	0.604 0.261	1.094 0.656
Cote Wallace KJ1	**CLOSE** $f_c' = 45.7$ **OPEN**	448	4 #5 top 2 #5 bottom	3 #3 ties horizontal 4 #3U vertical	1.027 1.038	0.048 0.013	0.327 0.085	0.621 0.310
Cote Wallace KJ2	**CLOSE** $f_c' = 49.7$ **OPEN**	448	4 #5 top 2 #5 bottom	3 #3 ties horiz. 4 #3 U-bars vert. 2 #3 diagonals	1.048 1.24 (ignores diagonal bars)	0.046 0.014	0.320 0.097	0.595 0.298
Cote Wallace KJ3	**CLOSE** $f_c' = 45.0$ **OPEN**	448	4 #5 top 2 #5 bottom	2 #3 ties horiz. 2 #3 U-bars vert.	1.011 1.009	0.048 0.013	0.324 0.084	0.626 0.313
Cote Wallace KJ4	**CLOSE** $f_c' = 45.6$ **OPEN**	448	4 #5 top 2#5 bottom	4 #3 ties horiz. Only 3 in joint? 4#3 U-bars vert.	1.054 1.075	0.050 0.013	0.336 0.089	0.622 0.311

minimum value of 0.85 for joints with low shear stresses and an upper limit of 1.20 for highly stressed joints.

The amended equations allow a reduction in A_{jh} of 48% and 67% of the NZS3101 1982 Code [6] requirements for knee joints with low column axial forces and joint shear stresses, $v_{jh} \leq 0.167 f_c'$ and for the maximum recommendation of $0.2 f_c'$, respectively.

The vertical joint shear design equation was also amended during the design period. The final version published was

$$A_{jv} = \frac{0.7}{1 + \frac{N^*}{f_c' A_g}} \left(\frac{h_b}{h_c}\right) A_{jh} \frac{f_{yh}}{f_{yv}} \qquad (2)$$

where h_b and h_c are the total depths of the beam and column, respectively and f_{yv} is the vertical joint reinforcing yield stress. This equation, for knee joints with low column axial force, gives a vertical joint shear reinforcement, A_{jv} requirement between 84 and 118% of the previous Code requirements for the v_{jh} levels above, when the column depth is equal to the beam depth.

The testing of New Zealand designed *exterior* beam-column joints to the current Standard's levels of joint shear reinforcement were studied by Cheung et al [20], but there has been no examination of *knee joints* designed to either the 1982 or 1995 Standards under seismic conditions.

The nominal joint shear stress limit was 1.5 $\sqrt{f_c'}$ (MPa) in the 1982 Code but became $0.2 f_c'$ in the 1995 edition. This represents a drop of about 40% for f_c' strengths of 20 MPa and a 6% decrease for 50 MPa concrete. Where unreinforced joints are considered the maximum joint shear strength is related to the diagonal tensile strength, which is proportional to $\sqrt{f_c'}$, but for joints adequately shear reinforced with horizontal and vertical ties, the shear strength is more dependent on the diagonal compression strut and is better measured in terms of f_c'.

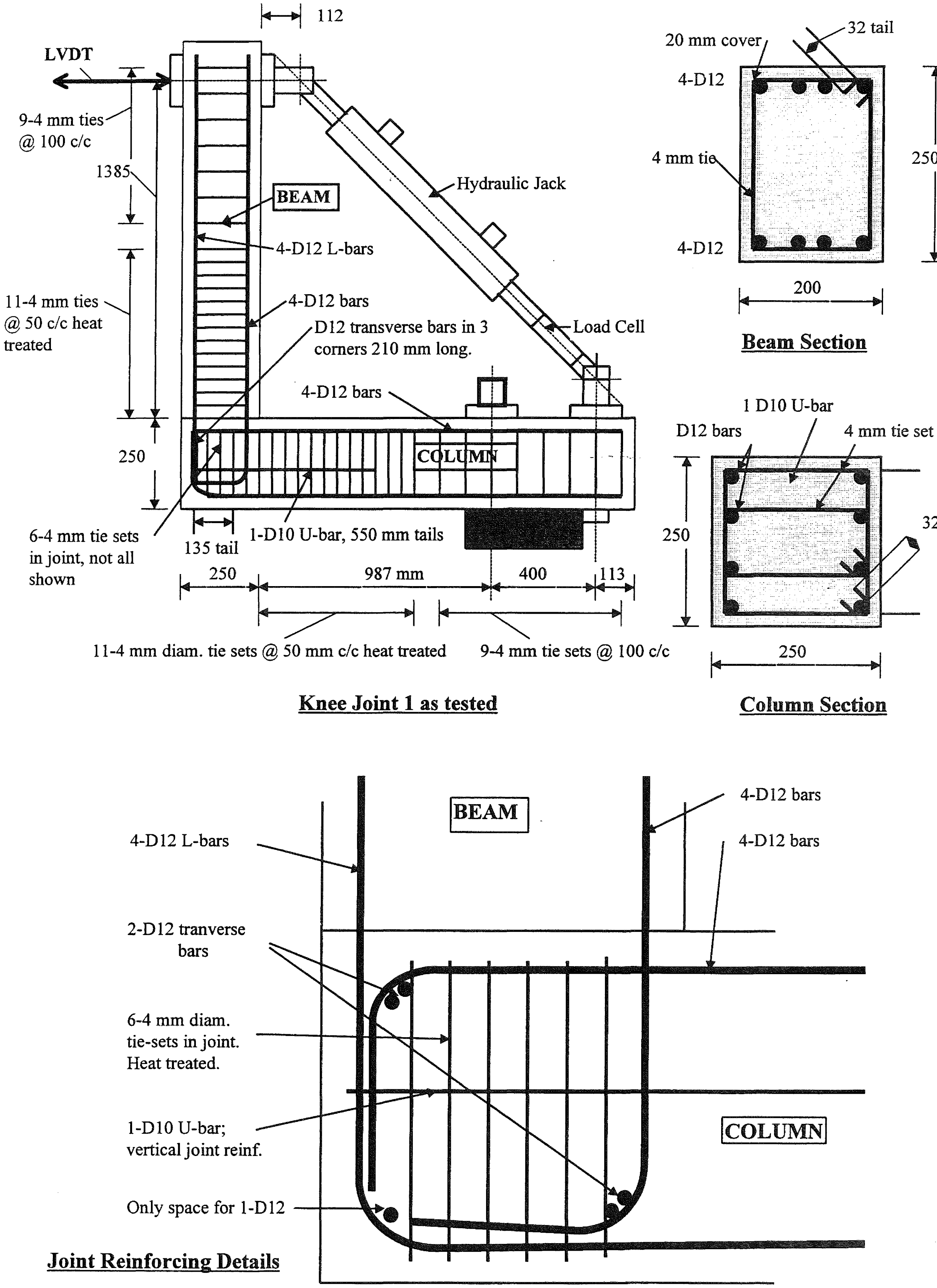

Figure 1: Knee Joint 1 designed to 1995 Standard with small diameter bars (D12 standard hooks on inner bars)

From the previous monotonic knee joint work and the few cyclic knee joints tested in the United States of America, it was envisaged that the $0.2 f_c^{'}$ limit would be unattainable and impracticable in the 1995 Concrete Standard designed knee joints. This is also due to the small joint dimensions considered here and the fact that only one beam enters the joint, approximately halving the joint shear stress magnitude when compared with a similar *interior* joint.

The objectives of this testing programme were to check the suitability of the 1995 Concrete Standard's (NZS3101) requirements for knee joints designed for seismic loading and also to ascertain the strength and ductility capabilities of reinforced concrete knee joints designed to the 1960's Code of Practice.

Knee joints 1 and 2 were designed with a medium amount of beam reinforcing (4-D12 bars top and bottom, $p = p' = 1.01\%$) and their only difference was in the beam and column bar anchorage detail. Knee 1 incorporated 90-degree standard hooks on the bottom beam and inner column bars with continuous L-bars for the top beam and outer column bars. The column principal reinforcement was also 4-D12 bars on the outer and inner faces. Figures 1 and 2 detail the reinforcement in knee joints 1 and 2, respectively. Knee 2 used continuous U-bars as the beam and column main bars. The internal bend radius used throughout was the minimum specified, $2.5d_b$ = 30 mm. A $12d_b$ tail = 144 mm was specified for the standard hook. The anchorage of this principal reinforcement complied with all aspects of the 1995 Standard's requirements, but it was necessary to invoke the requirement that two extra transverse bars, of at least the same diameter as the bars being anchored, be positioned in the 90-degree bend to reduce the hook's minimum development length, (L_{dh} = 150 mm) by 20%, measured from a point eight beam bar diameters in from the inner column face. The length of 8 bar diameters is to allow for the probable yield propagation into the joint under the cyclic reversals of moment.

The horizontal joint shear ties comprised 6 sets of 4 mm diameter wire, each set comprising 2 rectangular ties with standard 135-degree anchorages. The 4 mm hard-drawn wire was heat-treated to reduce its yield stress to about 300 MPa and restore its ductile stress-strain characteristics. When these first two units were designed the $6v_{jh} / f_c^{'}$ factor, which now appears as a multiplying factor in formula (1) for calculating the amount of effective horizontal joint shear reinforcement, A_{jh}, was not included and the ties were designed to the Draft Code as it then existed. This meant that the ties were theoretically over designed by the 1/0.85 factor ($\cong$ 18%), because $6v_{jh} / f_c^{'} \leq 0.85$ here. However, using the final NZS3101:1995 design formula with the $6v_{jh} / f_c^{'}$ factor included and with f_y = 300 MPa, $f_c^{'}$ = 30 MPa, the actual f_{yh} = 266 MPa and 20 kN axial tension on the column gave an A_{jh} value of 308 mm². The actual amount provided was 24 legs of 4 mm diameter ties = 301 mm²; an under design of only 2%.

The vertical joint shear reinforcement requirement, A_{jv} was 182 mm² using the actual transverse steel yield stress, f_{yv} = 318 MPa, in the Concrete Standards' final design equation (also modified during the draft discussion period). The actual A_{jv} used was a single D10 U-bar (A_{jv} = 157 mm²) positioned outside the top beam bars and the tails of the inner column bars at the joint's top but anchored into the column below within the 4 ϕ column ties, see Figure 1. A full development length was provided beyond the bottom of the joint zone. In Unit 1 the U-bar was positioned outside all the column bars but in the later units space restrictions meant that the U-bar was placed <u>inside</u> the outer column U-bars, thus reducing the confining effect on the column bar anchorage.

The provision of a "weak beam-strong column" approach is not necessary at the top of ductile structures where the column axial force is usually small and the formation of column plastic hinges under the roof beams is unlikely to harm/worsen the performance of the frame during a major earthquake. Therefore there was no attempt to make the column measurably stronger than the beam. The beam and column potential plastic hinge zones were detailed as per the current Concrete Standard (1995), with 4 ϕ ties @ 50 mm c/c in the beam plastic hinge and double ties of the same size and spacing in the column plastic hinge, as shown in Figures 1 and 2.

In knee joints 1 and 2 it was expected that the horizontal shear stress reached in the joint would be approximately 2.4 MPa or $0.08 f_c^{'}$, which was only 40% of the maximum allowed in NZS3101: 1995 [10].

Knee joints 3, 4 and 6 were designed so that the expected joint shear stress would be higher than the earlier units, at about 3.6 MPa or $0.12 f_c^{'}$. The principal beam reinforcement was increased to 3-D16 U-bars in knee 3 ($p = p' = 1.36\%$), while knees 4 and 6 had 3-D16 bars in the top of the beam and 2-D16 bars in the bottom, ($p = 1.36\%$, $p' = 0.91\%$). These comprised 2-U bars and 1 L-bar, as shown in Figures 3 and 4. Knee 6 was different from knee 4 only in that 2-D12 diagonal bars were added across the inner corner to improve the opening moment performance. These diagonals were anchored in the top and outer faces of the beam and column, respectively. Figure 5 shows the details for knee 6. The amount of extra cross-sectional area of diagonal bars was found by using the recommendation of Nilsson and Losberg [11], that the area be between 33 and 50% of the area of main beam tension reinforcing (56% in knee 6).

The number of 6 mm diameter, 3 legged ties placed horizontally in the joint were five in knees 3 and 6 and four in knee 4, which had the smaller bottom beam reinforcement ratio. The design formula (1) gave an A_{jh} amount of 290 mm² for knee 3 assuming f_y = 300 MPa and f_{yh} (actual) = 378 MPa., while in knees 4 and 6 the A_{jh} required was 192 mm² and 199 mm², respectively. These smaller amounts of horizontal joint shear reinforcement are due to the opening moment action being critical in the design of most exterior joints. This is because the column axial force will always be less than for the closing moment condition, due to overturning frame action and assuming vertical earthquake affects are ignored. Also the A_{jh} formula is directly proportional to the β value (the ratio of the area of compression beam reinforcement to the area of tension beam reinforcement). Therefore when the top steel area is 50% larger than the bottom steel area, as in knees 4 and 6, β was equal to 2/3 under closing conditions and 1 (the maximum value) under opening moments. That is, the area of horizontal ties in an external joint is proportional to the bottom beam steel area, not the often larger top beam steel area. This is only true when $6v_{jh} / f_c' < 0.85$, because when the ratio is greater than 0.85 at higher joint shear stresses under closing moments will produce a larger A_{jh} value than the lower shear stress conditions under opening conditions. For a full explanation and derivation of the NZS3101:1995 design equations the reader is directed to Paulay and Priestley [21]. The design formulae also have an over strength factor included for the yield strength of the beam bars (1.25) and it is for this reason that the design requirements for A_{jh} mentioned here used the minimum specified f_y value of 300 MPa and not the actual yield stress found from tensile tests of the main reinforcement.

The actual amounts of A_{jh} provided were 424 mm^2 in knees 3 and 6 and 339 mm^2 in knee 4. The extra amount provided in knee 6 was due to the added joint shear stress possibly accruing from the two extra diagonal bars at the critical column face section. Usually the additional moment strength and joint shear stress would be neglected in design and this was done in a later unit, knee 9. Thus the horizontal ties were over designed by margins of 46% for knee 3, 76% for knee 4 and 114% for knee 6. However the Standard stipulates that the joint tie-sets adjacent to the top and bottom beam bars in exterior or interior beam-column joints should not be included in the tie-sets making up A_{jh}, due to their ineffective shear carrying capacity. In these knee joints only the bottom tie-set lay next to the beam bars and should be excluded, thus reducing the over design of the ties to 11, 32 and 71% for knees 3, 4 and 6, respectively.

The vertical joint shear reinforcement was 2-D10 U-bars in each of these three joints, A_{jv} = 314 mm^2. Using the same approach as above, with the actual f_{yh} and f_{yv} values, knees 3, 4 and 6 are over designed by 39, 110 and 107%, respectively when considering vertical joint steel.

Knee joint 7 was an attempt to get the maximum feasible amount of reinforcement into this small beam section. 3-D20 U-bars were provided ($p = p' = 2.14\%$). Theoretically this would have given a maximum joint shear stress of about 5.65 MPa or $0.19 f_c'$, for a concrete compressive strength of 30 MPa, this being close to the 1995 Standard's $0.20 f_c'$ limit for v_{jh}. However even though the specified concrete strength was 30 MPa, the actual f_c' value at testing was 50 MPa, which somewhat destroyed the aim of the test. 8-6 mm diameter tie sets (3 legs per set) were positioned in the joint, with difficulty, giving an actual A_{jh} of 679 mm^2. The design formula gave a value of 593 mm^2 using f_c' = 30 MPa producing an over design of about 14% (1% if 1 tie-set excluded). In this case the closing condition is critical due to the larger v_{jh} stress producing a larger $6v_{jh} / f_c'$ factor. If the actual f_c' of 50 MPa was used the A_{jh} amount decreased to 453 mm^2 (due to the lower $6v_{jh} / f_c'$ factor) and the horizontal joint ties were now over designed by 31%, when 1 tie-set is ignored.

Three D10 U-bars were provided for vertical joint shear reinforcement, A_{jv} = 471 mm^2, producing an over design of 65%. (For the actual f_c' = 50 MPa and f_{jv} = 337 MPa, A_{jv} required was 286 mm^2 under opening conditions with the column tension force of 37 kN). The joint's reinforcing details are shown in Figure 6.

Knee 9 was a refined version of Knee 6 with 2-D12 diagonal bars and the same 3-D16 top and 2-D16 bottom beam bars. However there were only 3 sets of 6 ϕ ties horizontally in the joint zone, giving a theoretical over strength of about 27% for horizontal joint shear. The required A_{jv} amount was 151 mm^2 and 1-D10 U-bar was provided with a cross-sectional area of 157 mm^2 (2 legs), a 4% over design. There were no transverse bars positioned in the 90-degree bends of the column and beam bars and this meant that only 70% of the L_{dh} requirement was provided, assuming that the 150 mm minimum length is appropriate with these small columns. Figure 7 gives the joint's reinforcing details.

Knee 10 was identical to knee 9 except all the principal beam and column reinforcing was anchored in the joint with standard 90-degree hooks. The tails on these hooks were only $9.4 d_b$ long (150 mm), instead of the specified $12 d_b$. The reason for these short tails was the lack of space to accommodate the bottom beam bar and inner column bar tails in the joint. The full $12 d_b$ could only have been accomplished if column and beam stubs had been added to the joint. In an effort to compensate for this inadequate anchorage, two D16 transverse bars were added to each of the three joint corners with 90-degree bends, as detailed in Figure 8.

Joint 14 was similar to knee 9 but with no transverse D16 bars in the 90-degree bends and with the addition of 2-D12 bars across the joint's diagonal in an attempt to improve the joint's closing moment behaviour at high ductilities, see Figure 9.

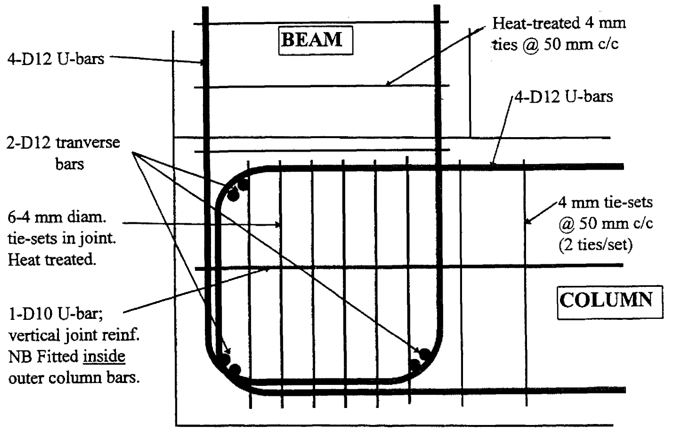

Figure 2: Knee Joint 2 with D12 U-bars

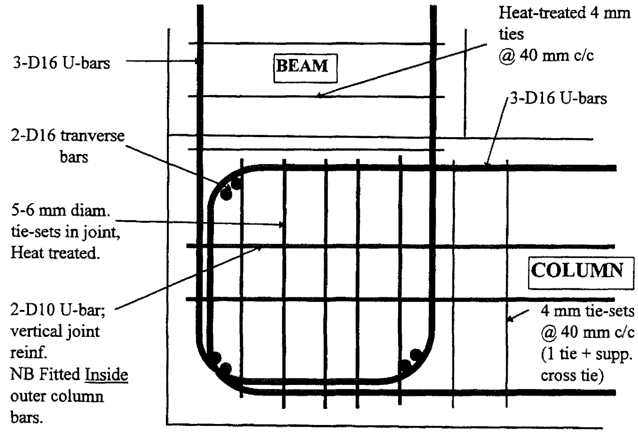

Figure 3: Knee Joint 3 with D16 U-bars

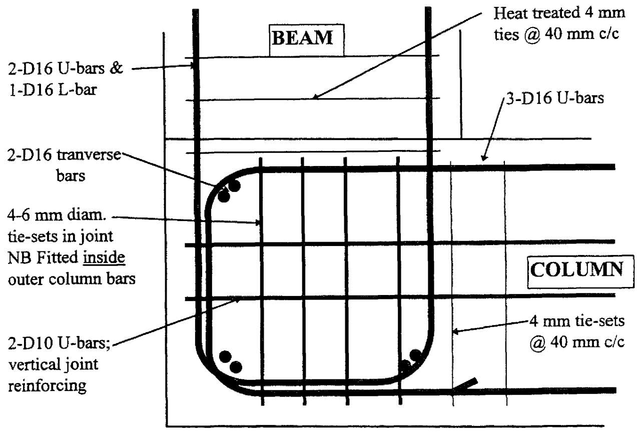

Figure 4: Knee Joint 4 with unequal top & bottom beam bars.

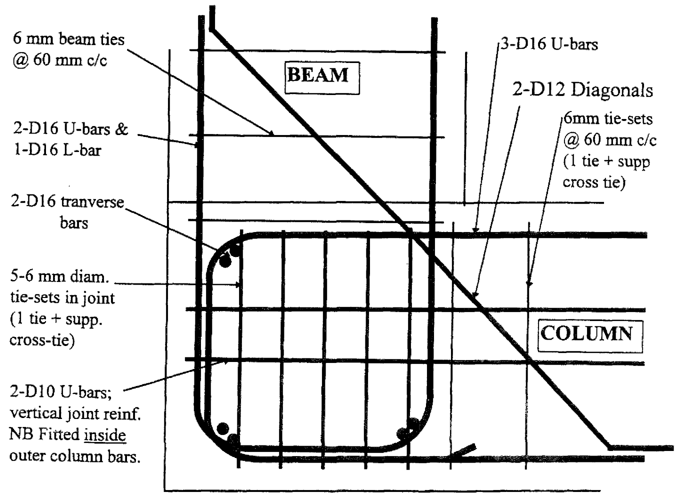

Figure 5: Knee Joint 6 with two extra diagonal bars

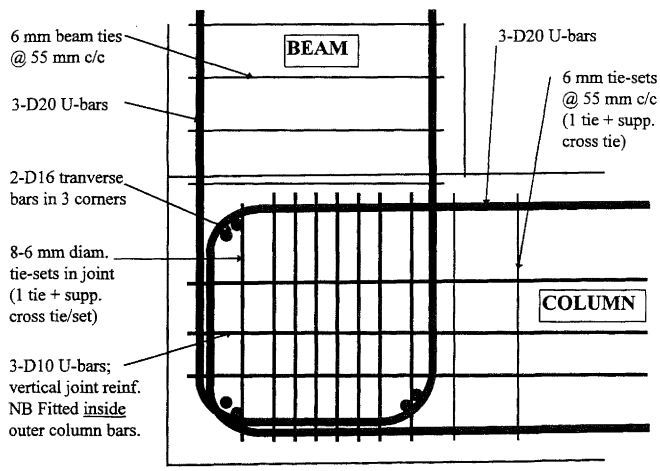

Figure 6: Knee Joint 7 with D20 U-bars

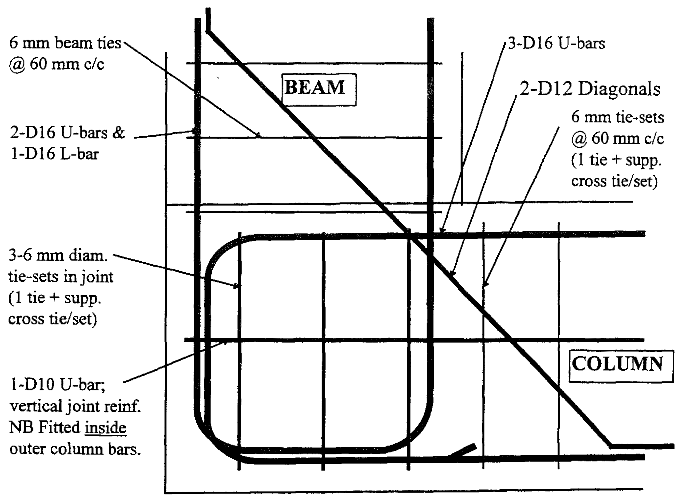

Figure 7: Knee Joint 9 with unequal top and bottom beam bars plus two extra diagonal bars

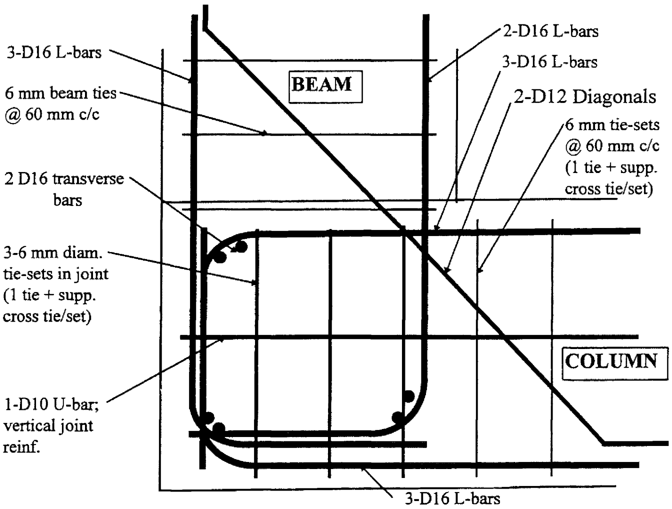

Figure 8: Knee Joint 10, main bars with standard hooks plus two extra diagonal bars across inner corner.

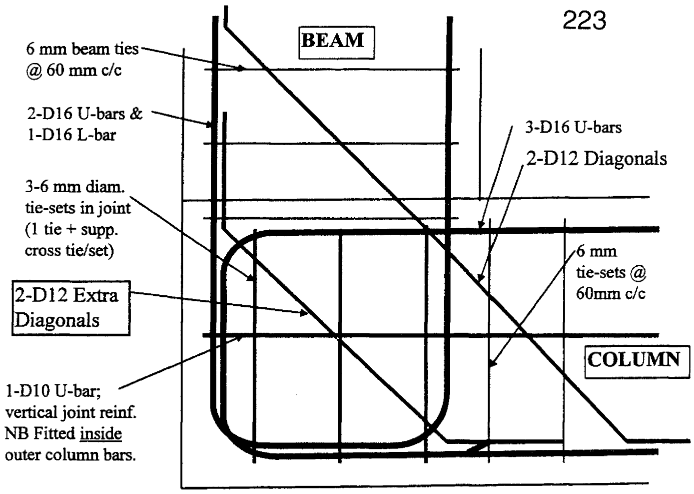

Figure 9: Knee Joint 14 with extra diagonal bars through joint and re-entrant corner.

2.2 1960's Knee Joints

Knees 5 and 8 were designed to the Concrete Code used in the mid *1960's (NZSS1900:Chap. 9.3: 1964)* [18]. This New Zealand Standard had very few clauses specifically related to earthquake loading and detailing considerations. Beam-column joints could be detailed with no transverse joint shear reinforcement in either direction, and poor anchorage details, by today's standards, were common. Beam bars were often bent out of the joint region when hooks were detailed, although the possibility of a positive bending moment at the column face was usually not considered and bottom bars were often cut off near the column face. Plain round bars, without deformations were also commonly used in beams and columns as main bars.

The knee joints 5 and 8 were identical except that joint 5 used plain round bars as principal reinforcing, while joint 8 incorporated deformed bars; 3-16 mm diameter top bars and 2-16 mm beam bottom bars. The beam and column principal bars were provided with 90-degree hooks with 32 mm ($2d_b$) internal radii and a $4d_b$ tail (64 mm). The inner column bars were bent into the joint but the two bottom beam bars were bent down into the column near the column's outer face. Figure 10 shows the reinforcement details of knee joints 5 and 8.

The 1964 Code allowed two types of anchorage for beam and column stirrups. In these tests the better anchored 135-degree bend with a $8d_b$ (32 mm) tail was employed in the beams but the poorer 90-degree bend with $16d_b$ tail was used for the less critical column ties. d_b in this case is the diameter of the **ties**; 4 mm hard drawn wire being used in these units. The 90-degree anchorage behaves badly in yielding members when the cover spalls. The 4 mm drawn wire was equivalent to the 6 SWG wire specified in the 1964 Code. The shear stresses in the members did not exceed the Code's maximum allowable stress ($0.03 f_c' = 0.9$ MPa) and thus all the shear was assumed to be carried by the concrete. The maximum column stirrup spacing was dictated by the 2/3 of member depth requirements = 167 mm, while in the beam it was ≤ 3/4 beam depth = 187 mm.

The three missing knee joints in the sequence (Nos. 11, 12 & 13) had anchorage plates welded to the beam and column bars, and are described in another paper [25].

2.3 Test Construction and Setup

The knee joint units were cast on their sides in one pour using commercial ready-mix concrete with a specified 28 day compressive stress of 25 MPa (30 MPa in joint 7) and a maximum aggregate size of 10 mm. Table 2 shows the f_c' values obtained immediately after testing and the reinforcing tensile yield stresses. The units and the concrete test cylinders were covered with sacking and kept moist for a week after casting.

The knee joints were tested 90 degrees out of prototype position with the beam vertical and the column end tied down to the strong floor with tensioned high strength bolts. The 50 kN hydraulic jack was diagonally positioned between the beam and column ends (see Figure 1), thus applying a lateral and axial force to both the beam and column. The member's axial forces were compressive under closing action and tensile under opening actions. This arrangement closely models the prototype actions under seismic conditions, assuming that the gravity loads are small relative to the seismic axial forces. A load cell measured the applied jack force and displacement portal gauges were used to measure the flexural, shear and axial deformations, using a datalogger. The portal gauges were attached to 6 mm diameter steel studs welded to the principal reinforcing with a 5 mm clear gap around them through the cover concrete. The positions of the portal gauges are shown in Figure 11. The beam-tip displacement, at the elevation of the applied force, was measured with a LVDT, a turnpot displacement transducer and a metre rule as backup. The beam plastic hinge zone flexural displacements were repeated on the back of each unit as a check. None of the reinforcing was strain gauged as past experience has shown that gauges get ripped off when bars begin slipping, as was expected here within the small joint zones.

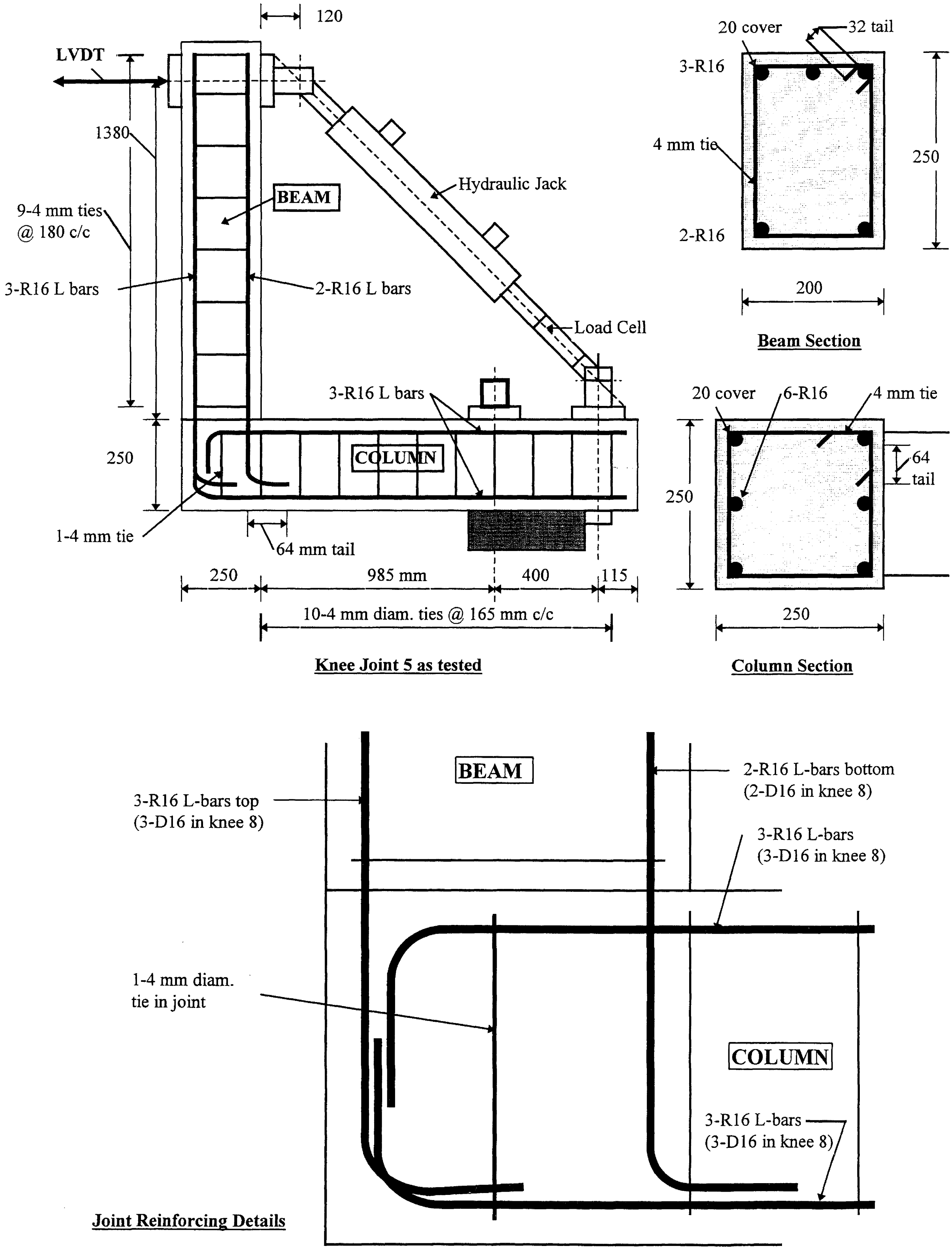

Figure 10: Knee Joint 5 designed to the 1960's Code with plain main bars.
Knee 8 identical except for deformed main bars.

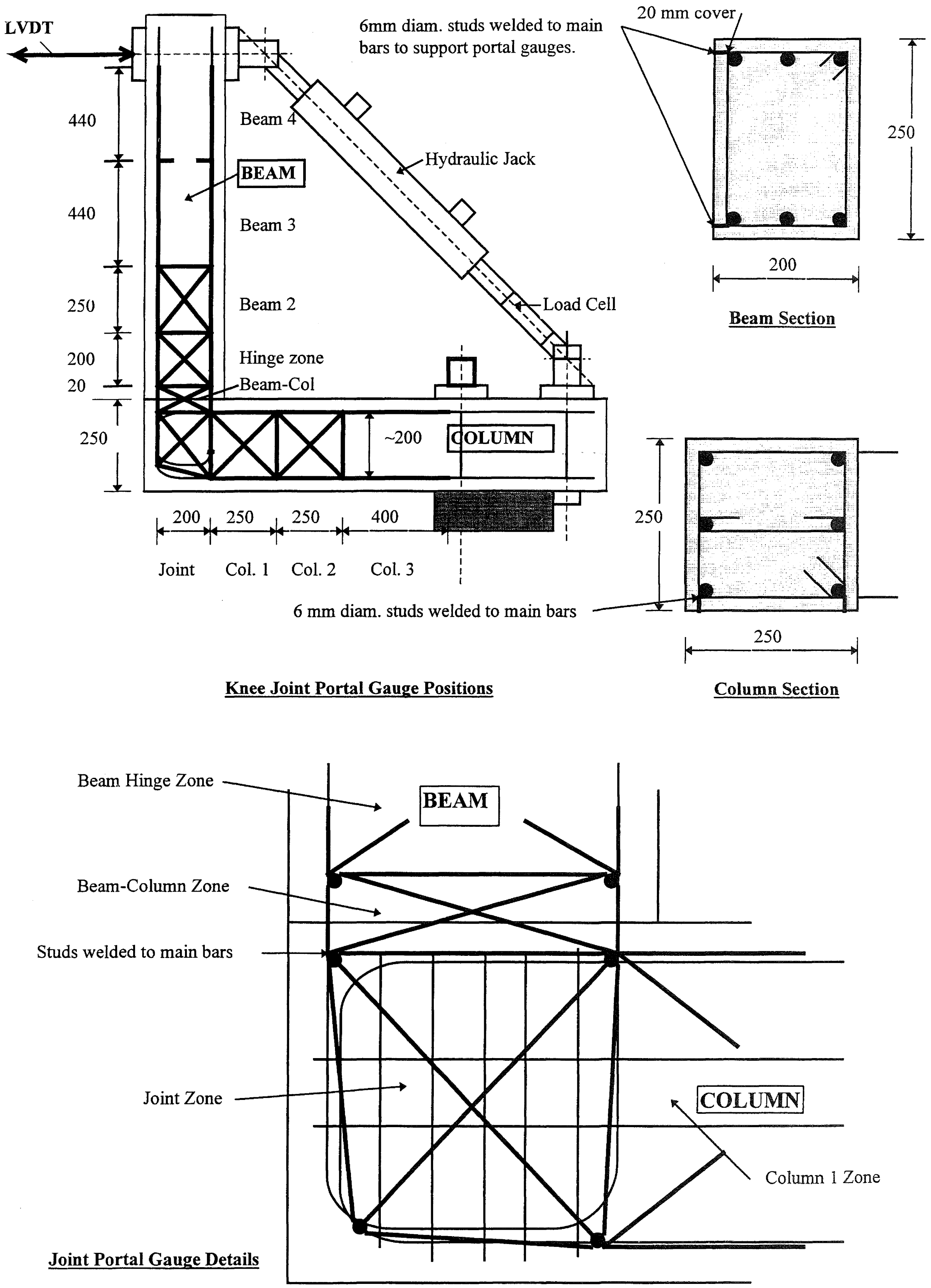

Figure 11: Portal Transducer Gauge positions on studs welded to main reinforcing bars.

TABLE 2: Concrete Compressive Cylinder Strengths (f_c') and Yield Stresses of Reinforcing in Knee Joints

Knee Joint	f_c' (MPa) Concrete	Bar type, diam. (mm)	f_y (MPa) Main bars	Bar type, diam. (mm)	f_{yh} (MPa) Joint Ties	Bar type, diam. (mm)	f_{yv} (MPa) Vert. Jt. U-bars	Bar type, diam. (mm)	f_{yd} (MPa) Diag. bars
1	27.8	D12	358	4ϕ H.T.	266	D10	318		
2	27.8	D12	358	4ϕ H.T.	266	D10	318		
3	34.0	D16	328	6ϕ	378	D10	343		
4	34.0	D16	328	6ϕ	378	D10	343		
5	33.6	R16	355	4ϕ	537				
6	33.6	D16	324	6ϕ	365	D10	337	D12	355
7	50.0	D20	333	6ϕ	378	D10	337		
8	40.4	D16	340	4ϕ	537				
9	39.8	D16	333	6ϕ	322	D10	337	D12	345
10	39.7	D16	333	6ϕ	322	D10	337	D12	345
14	32.4	D16	325	6ϕ	322	D10	337	D12	345

NOTE: H.T. = Heat Treated

2.4 Loading Sequence

The loading sequence previously used for many structural component, sub-assembly tests in New Zealand was again used in these tests. This entails two 'elastic' cycles, up to a force needed to apply about ±0.75 of the beam nominal moment, M_n at the critical column face. The nominal moments were calculated using the actual material properties of steel and concrete (Table 2), including the effects of axial force on the beam. From the displacement reached at the 3/4 nominal moment level the first yield displacement was estimated by linear extrapolation. The next two displacement controlled cycles were to displacement ductility ±2, while subsequent double cycles to ductility factor ±4, ±6 and ±8 were completed. If the sustained load fell to below about half of the nominal yield force in the ductility 6 cycles, the test was usually terminated. As a negative bending moment would normally exist at the column face, prior to the earthquake, the knee joints were forced into the closing position first in each new cycle.

3. TEST RESULTS

3.1 Knee Joint 1:

This joint with 90-degree standard hooks began developing a beam plastic hinge during the ductility 2 cycles, reaching its closing nominal moment strength in the first cycle to ductility 4, but only reached a maximum of $0.95M_n$ under opening moments. However as the displacement cycles continued, the joint progressively failed due to joint shear, joint side cover was loose and the back of the joint cover had fallen off during the ductility 4 cycles. The applied force - beam-tip deflection hysteresis loops are plotted in Figure 12 and show a gradual reduction in load sustained as the displacement ductility increased. The P_{nom} force levels shown are the hydraulic jack forces to produce the nominal positive and negative bending moments at the column face beam section. As the test continued the joint became more distressed with the joint shear deformations contributing about 40 and 60% of the total lateral deflection under closing and opening moments, respectively. The accumulated flexural plus axial deformations and separately the shear deflections calculated at beam-tip are shown in Figure 13 for the cycle peaks throughout the test. Also shown is the LVDT measured beam-tip deflection as a comparison. The error between the summation of the calculated flexural plus axial and shear deformations and the measured deflection plots was a measure of any portal frame inaccuracy and the small flexure and shear deflections not measured near the ends of the beam and column. The shear deformations were larger in the closing direction and became greater than the flexural deformations at ductility 6. Like Mazzoni's [9] tests, spalling of the joint cover caused the loss of anchorage of the hook's tails as the 90-degree bends tended to open, which subsequently allowed them to slip backwards and forwards destroying the joint core. The maximum M_{test}/M_n ratios sustained were 1.03 and 0.95 under closing and opening actions, respectively. At opening ductility 6 the nominal moment was close to being reached but in the corresponding closing cycle the moment carried had reduced to about 90% of M_n.

3.2 Knee Joint 2:

This joint with U-bars performed better than the first unit. A beam plastic hinge continued to form throughout the test and the strength degradation was not as large, as shown in Figure 14; the force-deflection history. During the second cycle to ductility 4 there was a substantial 30% decrease in maximum applied force in both directions, due to beam cover spalling near the column face. This spalling decreased the effective beam depth, thus reducing the section's maximum moment strength. In later cycles the joint's top and back cover did fall off but the side concrete remained intact.

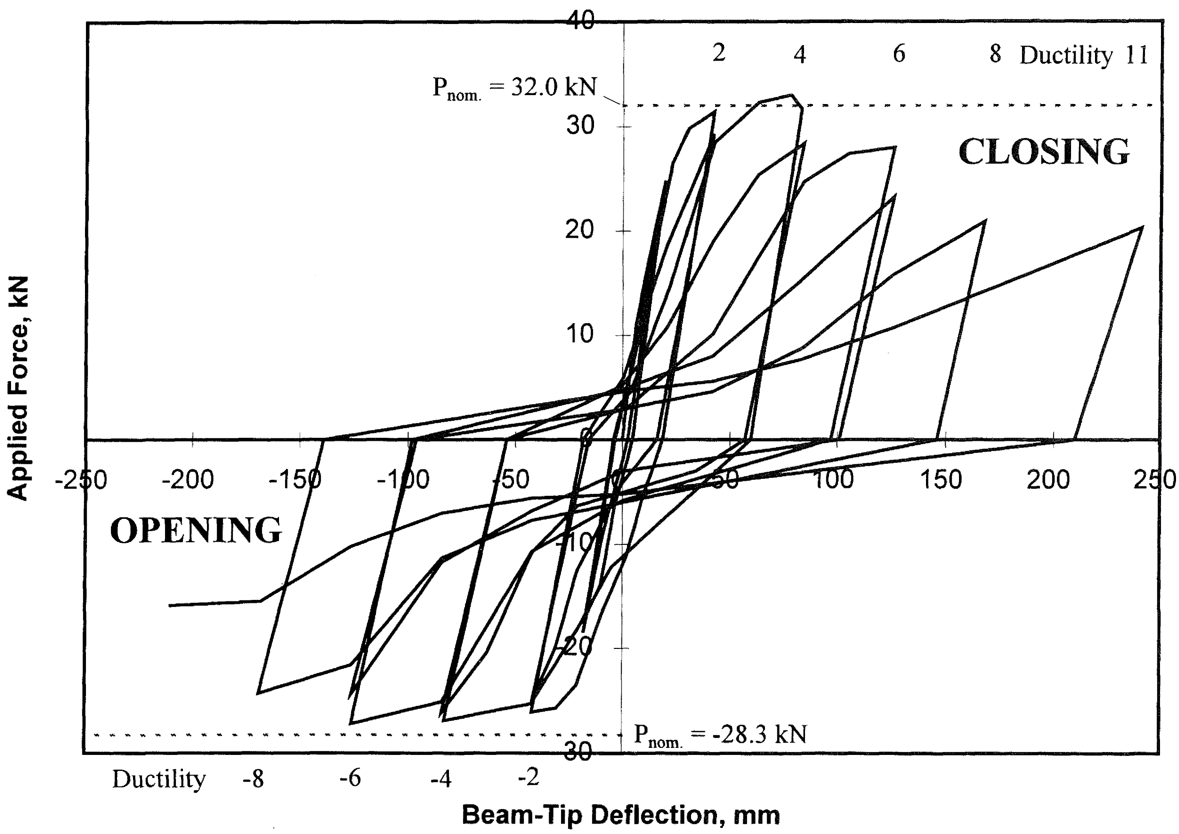

Figure 12: Knee Joint 1 Applied Force - Beam Tip Deflection loops.

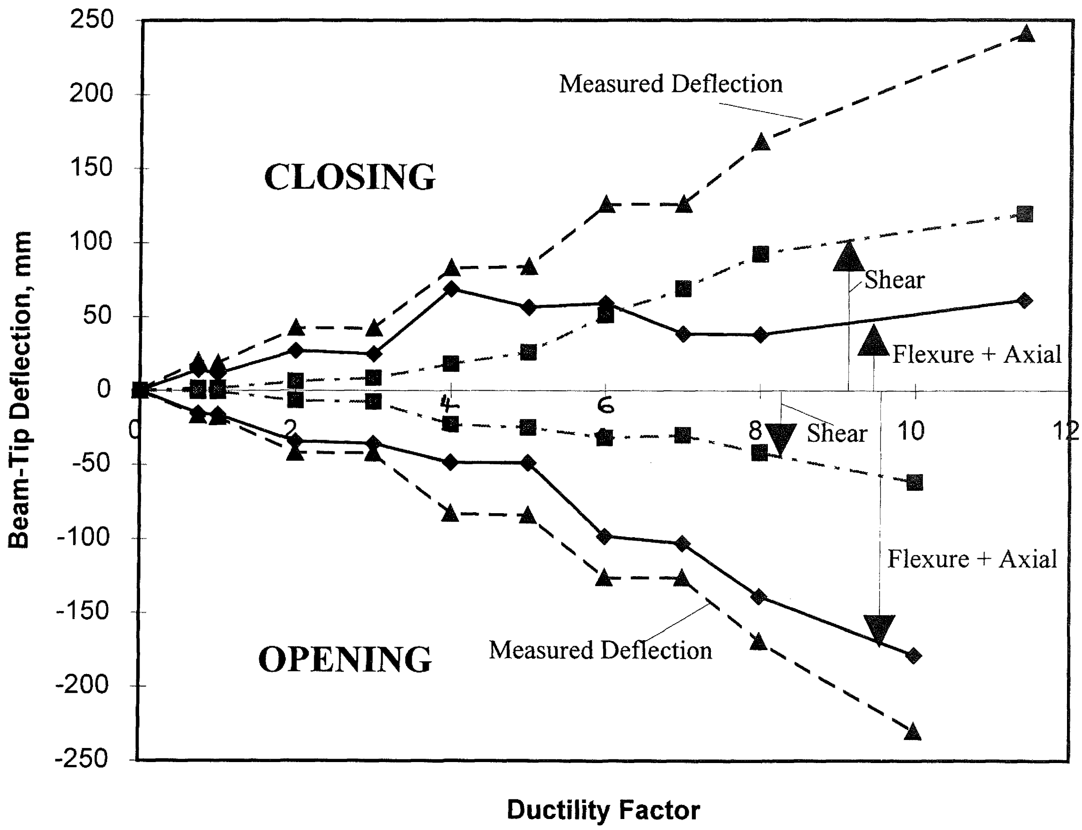

Figure 13: Knee 1 Flexure + Axial, Shear Deformations and Measured Beam Tip Deflections at cycle peaks.

In this test the joint shear deformations remained relatively constant over the entire test, while the beam hinge rotations accounted for approximately 80% of the total drift in both directions. Figure 15 shows the calculated, accumulated shear and flexural deflections at each cycle peak. The U-bars retained their anchorage within the joint and as a result the joint core remained secure. U-bar anchorages appeared much better than "standard hooks" in small sections under cyclic loading. The moment 'efficiencies', M_{test}/M_n , were almost identical to those attained in knee 1, namely 1.02 under closing conditions and 0.97 under opening moments. The lack of full strength under opening conditions was due to the arching action of the compression field bending down to form the diagonal joint strut and so reducing the effective depth at the beam-column interface, as described by Ingham, Priestley and Seible [22]. The maximum horizontal joint shear stresses reached in knees 1 and 2 were nearly identical, the average being $0.094 f_c^{'}$ (or $0.49\sqrt{f_c^{'}}$ (MPa)) closing and $0.079 f_c^{'}$, ($0.42\sqrt{f_c^{'}}$ (MPa)) opening.

The Concrete Standard's requirement of 1-D10 U-bar as vertical joint shear reinforcement appeared satisfactory but the use of two U-bars may have facilitated less joint damage in knee joint 1, as it would have restricted the column bar hooks from trying to straighten out.

3.3 Knee Joint 3:

This unit had a larger beam reinforcement ratio (p = 1.36%) incorporating U-bars and transverse joint reinforcement about 40% over the Concrete Standard's specification. This joint behaved better than the previous two, in that it maintained its nominal closing strength up to ductility 10 and had only a 20% reduction in opening strength in the first cycle to ductility -8. Full closing moment strength (44.5 kNm) was reached in the first cycle to ductility 4 and 97% of the opening nominal strength was attained at the second cycle at ductility -2, as shown, in the applied force versus beam-tip deflection plot in Figure 16. The closing force reached a value close to that required to yield the reinforcing at the column face in the test's first cycle, instead of the $0.75M_n$ peak. This was due to human reading error and didn't affect the displacement controlled cycles later in the test. The second closing cycle to $0.75M_n$ however was stopped at about $0.50M_n$ because the deflection was greater than the overloaded first cycle. The two opening cycles to $0.75M_n$ were almost identical.

Diagonal joint cracking occurred in the first 0.75 displacement ductility closing cycle with the opposing diagonal cracks forming in the first opening cycle to ductility -2. A plastic beam hinge began to form during the ductility 2 cycle but new cracking in the joint zone continued. During the first opening ductility -4 cycle the outer corner of the joint was pushed off and splitting cracks had formed around the position of the outer beam and column bars causing the back and top joint cover to become loose. The four main cracks in the beam hinge region continued to open at this stage. In the ductility ±6 cycles the joint progressively deteriorated but the main column face hinge crack also continued widening. Thus the inelastic rotation was occurring both in the joint and in the beam-column zone, rather than in the preferred beam plastic hinge. This can be seen in Figure 17, which shows the calculated shear and flexural (+axial) deflections measured at the beam-tip at each cycle peak. The flexural rotations were being caused by the slipping of the U-bars within the joint, rather than yielding of the beam bars in the beam plastic hinge. This slip was the cause of nearly 150 mm of the beam-tip deflection in the ductility 6 cycles, while the beam hinge zone deflection was only causing about 25 mm of the tip deflection. Only when the very wide column face crack closed did the slipping stop and some strength was then able to be sustained by the beam. The maximum joint shear stresses sustained in knee 3 were $0.095 f_c^{'}$, ($0.55\sqrt{f_c^{'}}$ (MPa)) and $0.077 f_c^{'}$, ($0.45\sqrt{f_c^{'}}$ (MPa)) under closing and opening conditions, respectively.

3.4 Knee Joint 4:

This knee joint had unequal top and bottom beam reinforcement ratios, one less horizontal joint tie-set than knee 3 and the same 2-D10 U-bars as vertical joint reinforcement. The applied load - beam-tip deflection plot is reproduced in Figure 18 and shows that the nominal strength was reached in both directions but the closing strength dropped off by 25% in the first cycle to ductility 6. In the second cycle to this displacement ductility there was a further drop of 25% in attained strength. The large reduction in opening strength only occurred in the second cycle to ductility -6, where about $0.60M_n$ was reached. This was due to large pieces of concrete cover spalling off the back and top of the joint, reducing the effective depth of the section at the critical column face and thus decreasing the moment able to be carried.

The loss of cover inevitably caused a loss of anchorage in the joint and slipping of the beam U-bars began to occur. In this joint the largest component of the beam-tip deflection was due to flexural rotation in the short beam-column zone (the region 40 to 240 mm out from column face, see Figure 11), while the beam plastic hinge deflection was about 60% of the beam-column zone deflection at ductility 4 but reduced to less than half that at higher ductility factors, emphasising that most of the deflection was due to rotation of the beam-column zone and bar slip. The shear deformations in this test were almost negligible. Figure 19 plots the envelopes of the M_{test}/M_n ratio against the displacement ductility for knee joints 1, 2, 3, 4 and 7. The moment ratio shown is only for the first cycle at each ductility factor, a drop in moment ratio always occurred in the second cycle to each specific ductility factor. The best closing moment performance was from knee 3, while knee 4 was marginally better than the others under opening moments.

3.5 Knee Joint 6:

This unit was identical to knee joint 4 except for the addition of 2-D12 diagonal bars across the inner joint corner and the addition of an extra 6ϕ joint tie-set. The force-displacement loops in Figure 20 show that the nominal closing and opening strengths were exceeded in the ductility 2 and ductility -2 cycles. The maximum strengths reached were 9% and 21% greater than the closing and opening nominal strengths, respectively. Any additional nominal strength due to the 2 diagonal bars at the column face was neglected in the calculation of M_n, but some effect must be assumed in the 21% increase in strength, as strain-hardening of the beam bars would normally be expected to only produce a smaller

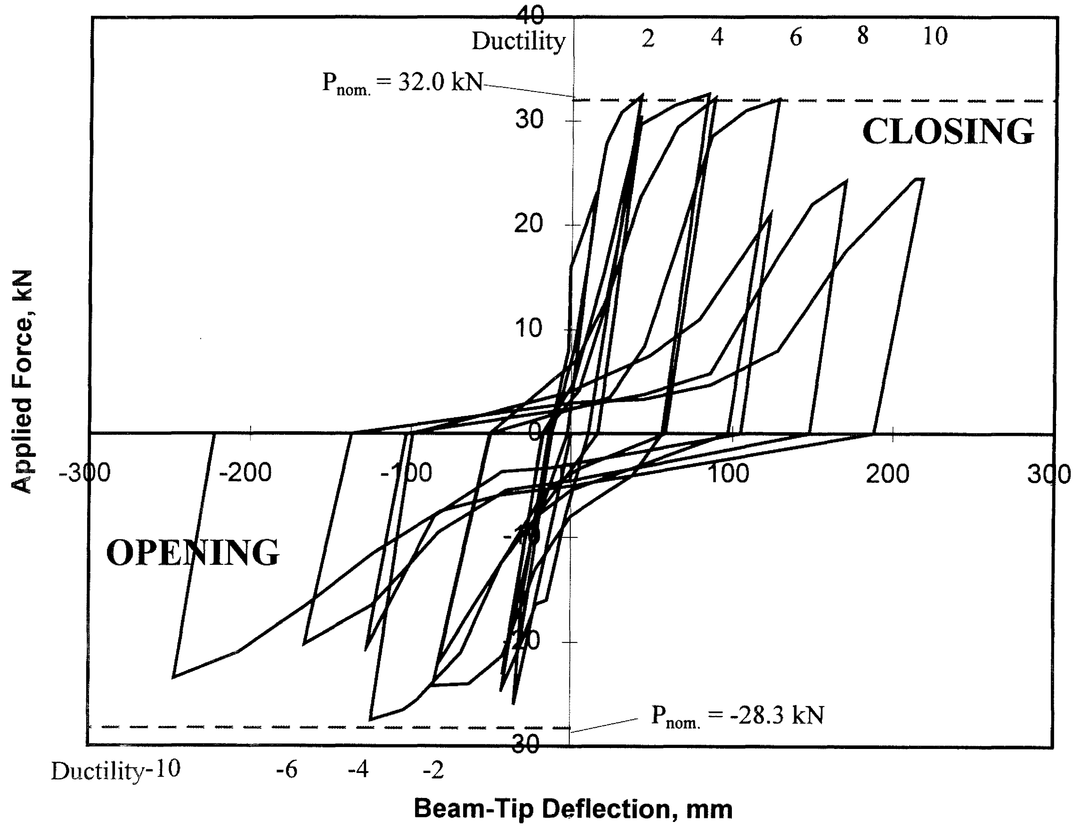

Figure 14: Knee 2 Applied Force - Beam Tip Deflection loops.

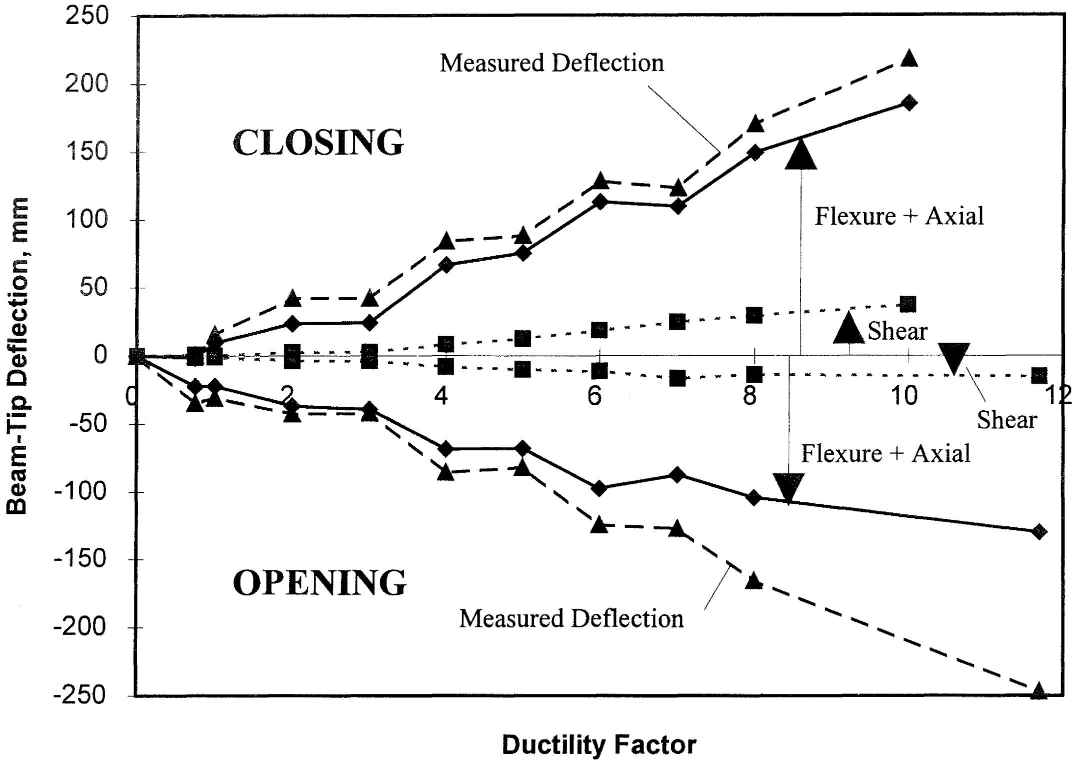

Figure 15: Knee 2 Flexure + Axial, Shear Deformations and Measured Beam Tip Deflections at cycle peaks.

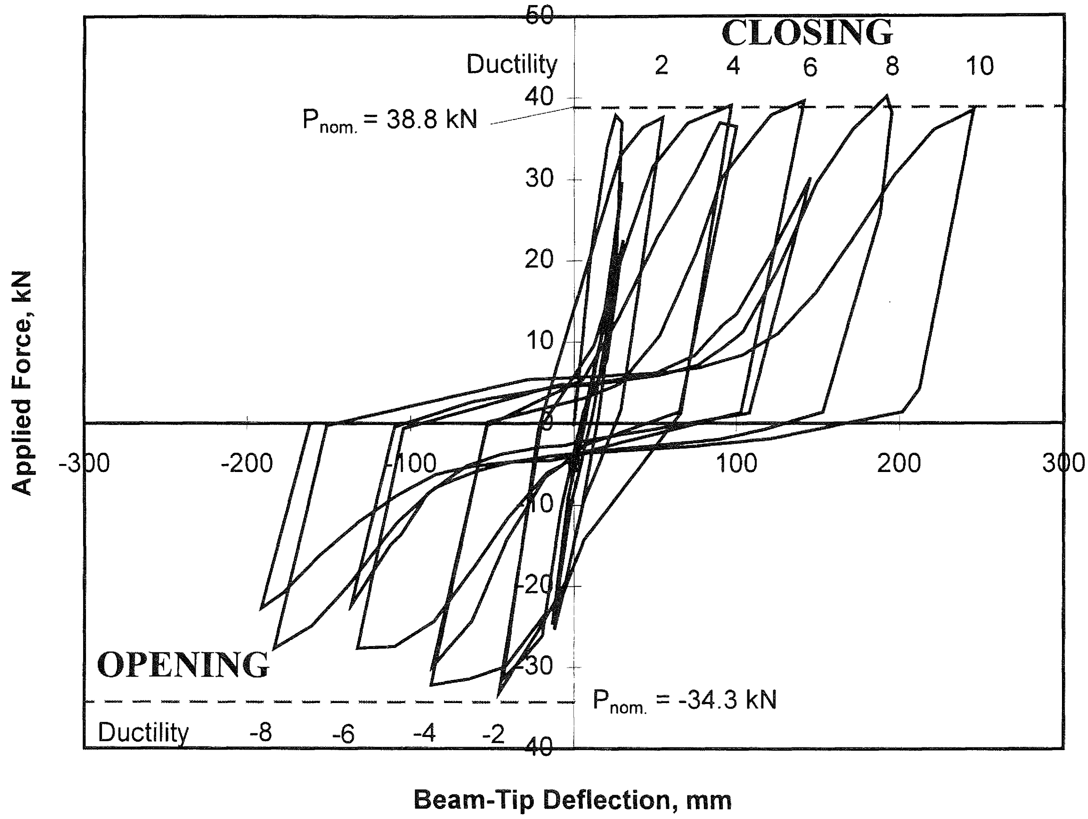

Figure 16: Knee 3 Applied Force - Beam Tip Deflection loops.

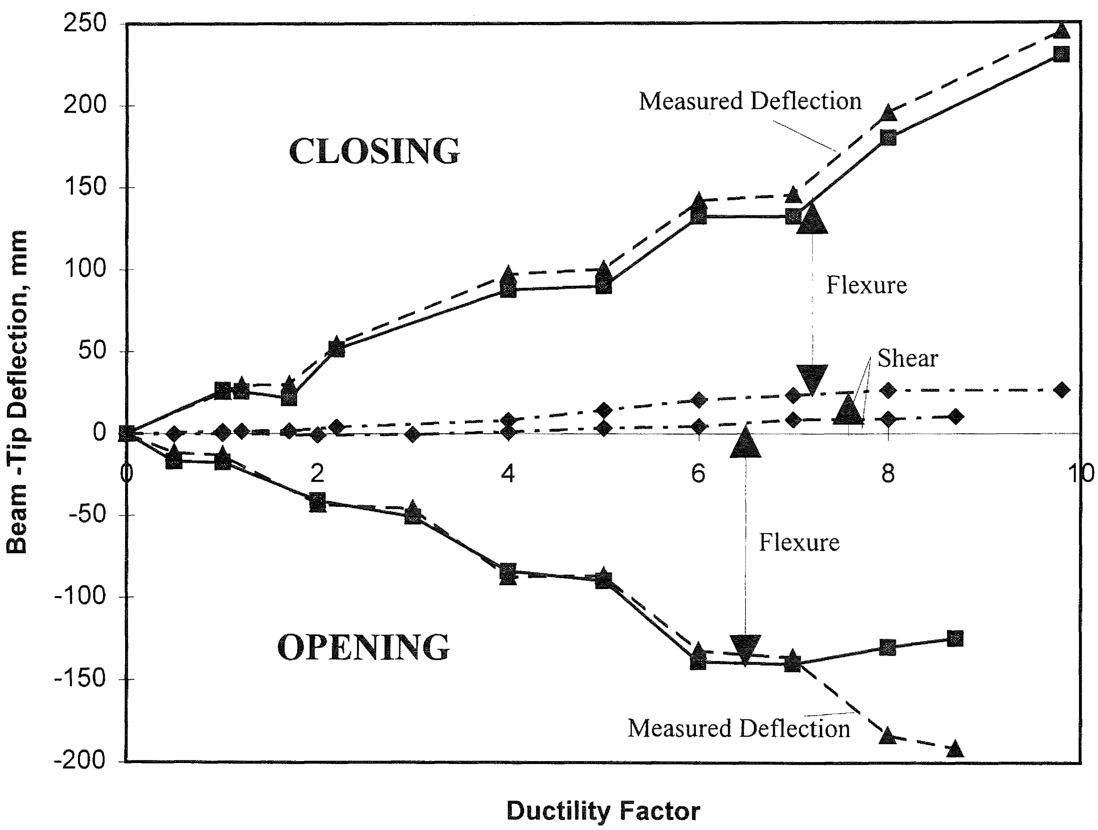

Figure 17: Knee 3 Flexural + Axial, Shear Deformations and Measured Beam Tip Deflections at cycle peaks.

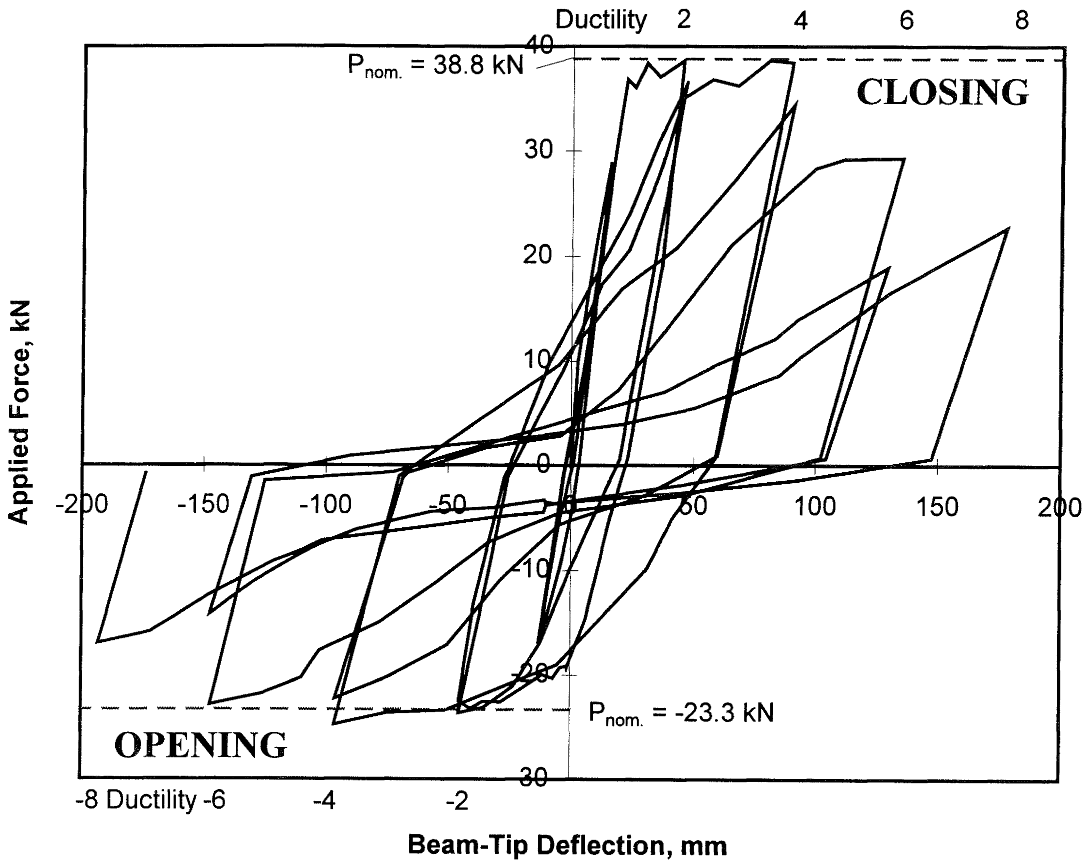

Figure 18: Knee 4 Applied Force - Beam Tip Deflection loops.

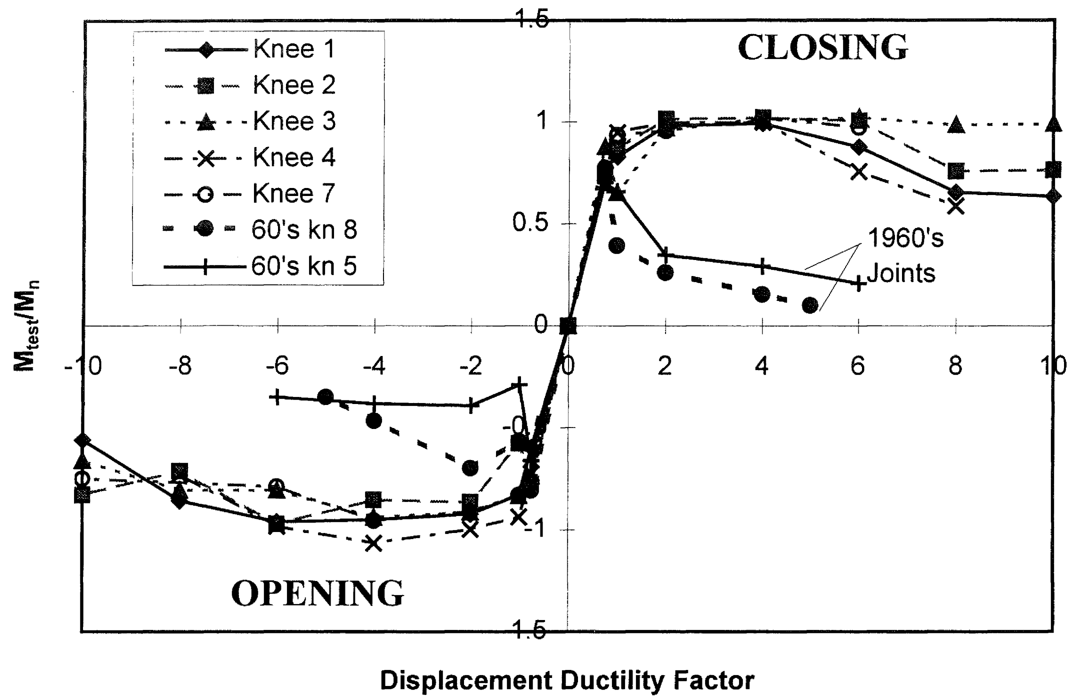

Figure 19: Applied Moment envelopes for conventionally reinforced knee joints, including two 1960's joints.

increase. This knee joint behaved very well, sustaining its nominal strength in both directions up to the ductility ±8 cycles, where a slight decrease in closing moment occurred. A substantial decrease in strength, greater than 20%, only occurred in the ductility ±10 cycles, when loss of beam cover reduced the effective depth by nearly 20%. Although very fine diagonal cracks formed across the joint in the ductility ±2 cycles, the joint remained virtually undamaged until the second ductility 4 cycle, when the splitting cracks formed around the outer bend of the column bars. These cracks widened and the back and top joint cover fell off in the first opening cycle to ductility -6.

The major beam cracks widened in the plastic hinge zone through the ductility ±2, ±4 and ±6 cycles, with the outer beam cover spalling off over a plastic hinge length equal to the beam depth, at ductility -6. The inner beam hinge cover spalled in the next cycle to ductility 8.

The core of the joint remained secure during this test, with only minor cracking evident in the side cover concrete, with negligible joint shear deformation evident during testing. In the ductility 8 cycle the beam-column zone flexure accounted for 100 mm of beam-tip deflection, while the plastic hinge zone accounted for 30 mm and the joint shear only about 3 mm. Figure 21 is a photograph of the joint at the first cycle at opening ductility -6, showing the minor joint damage and the obvious beam plastic hinge. No evidence of beam bar slip through the joint was seen in this test; shear failure or anchorage loss did not occur in this test.

As described earlier, the horizontal and vertical joint shear reinforcement were considerably over designed in knee joint 6, but this excess had the desired effect of allowing the joint zone to remain fully elastic. This did not occur in any of the earlier joints. The maximum joint shear stresses reached in this test were $0.10 f_c'$, ($0.58\sqrt{f_c'}$ (MPa)) and $0.064 f_c'$, ($0.37\sqrt{f_c'}$ (MPa)) under closing and opening moments, respectively.

3.6 Knee Joint 7:

In this joint the maximum feasible amount of beam and column principal reinforcement was designed for the size of the test unit's beam. The problems of placing the larger D20 bars and the 8 sets of horizontal joint ties in the confined joint space were time consuming and caused the cover to the main bars to increase to 23 mm, rather than the desired 20 mm.

This knee joint behaved well, up to and including the ductility ±4 cycles, reaching its nominal closing strength and just failing to reach its opening nominal strength by 5%, as shown in the hysteresis loops in Figure 22. It continued to sustain a moment of about $0.90M_n$ in the second cycles to ductility 4 and -4 and reached a moment of $0.97M_n$ in the closing ductility 6 cycle before the moment sustained dropped to 79% of M_n in the first opening cycle to ductility -6. In the next cycle to ductility 6, closing, the applied load reduced to 69% of the nominal strength.

Fine diagonal joint cracks formed in the first opening cycle to 0.75 of the yield moment and in the opposing direction in the second closing $0.75M_n$ cycle. By the ductility 2 cycles there were three diagonal joint cracks in both directions. The column face crack predominated in the ductility 2 cycles with two other major beam hinge cracks also opening. By the second opening cycle to ductility -2 the joint's back and top cover concrete had split away and during the ductility 4 cycles this cover fell off. Spalling of the beam cover at the inner corner also occurred in the same cycles. During the first ductility ±6 cycles more joint side cover spalled, from the outer corner inwards, till almost all the side cover had broken away by the end of the second ductility -6 cycle. The close spacing of the horizontal joint ties facilitated the spalling of the side cover. However the joint core remained well confined and seemingly little damaged. The beam top and bottom cover had spalled up to the second main crack, 100 mm out from the column face. Shear deformations remained small during the test, contributing less than 5 mm to the beam-tip deflection at the ductility ±4 cycles, when the beam hinge was contributing about 30 mm under closing actions. However in the ductility ±6 cycles the joint shear deformations almost tripled (closing), while the beam hinge rotations remained unchanged, showing joint degradation. At this stage some slipping of the beam bars through the joint occurred, showing that the reduction in hook development length (L_{dh}) in this unit was causing bond failure within the joint. L_{dh} was only 80% of that required for "standard hooks" although this unit used U-bars. The Standard [10] implies that U-bar anchorages are less efficient than 90-degree hooks but this series of tests contradicts that. The over designed amounts of transverse joint shear reinforcement did confine the joint and prevent a shear failure but bond failure still occurred due to the large diameter bars and the relatively small column depth.

The maximum horizontal joint shear stresses reached in knee 7 were $0.095 f_c'$, ($0.69\sqrt{f_c'}$ (MPa)) closing and $0.085 f_c'$, ($0.60\sqrt{f_c'}$ (MPa)) under opening moments. This appears to be the practical limit of v_{jh}, for knee joints with small section dimensions.

3.7 Knee Joint 9:

This joint was identical to knee 6 with diagonal bars across the re-entrant corner but it contained only three 6ϕ tie-sets within the joint (5 sets in knee 6) and only one D10 U-bar vertically in the joint instead of the two in knee 6. Also there were no transverse bars positioned in the 90-degree bends of the beam and column main bars within this joint.

As expected this joint did not behave as well as knee joint 6; the closing and opening maximum M_{test}/M_n values reaching 0.96 and 1.15, respectively, compared with 1.09 and 1.21 in joint 6. Knee 9's force-deflection response is shown in Figure 23. The loading sequence was reversed from that used previously, with the first cycle to $0.75M_n$ in the *OPENING* direction. The opening nominal strength was exceeded till the second cycle to ductility -6, after which it carried about 90% of M_n up to displacement ductility of nearly -12. The closing cycles however decreased in strength to 75% of M_n in the second cycle to ductility 4 and further strength reductions to about $0.5M_n$ being reached in the second cycle to ductility 6 and the cycles to ductilities 8 and 11.

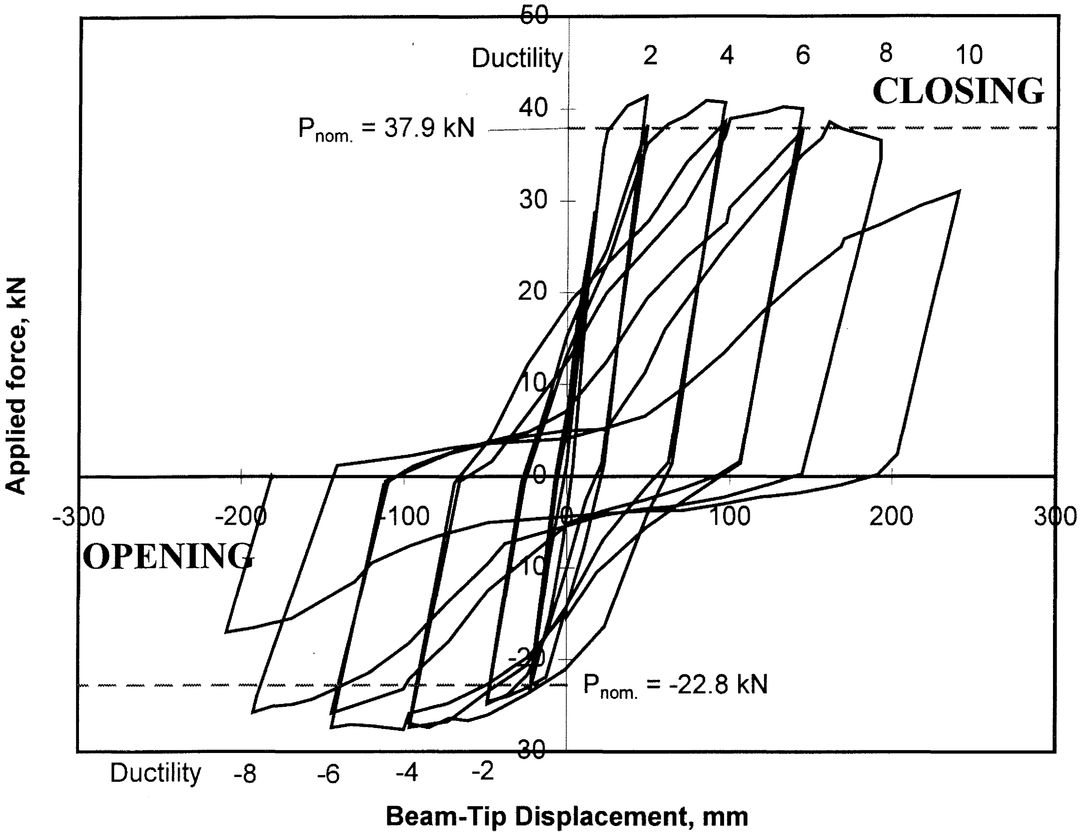

Figure 20: Knee 6 Applied Force - Beam Tip Deflection loops.

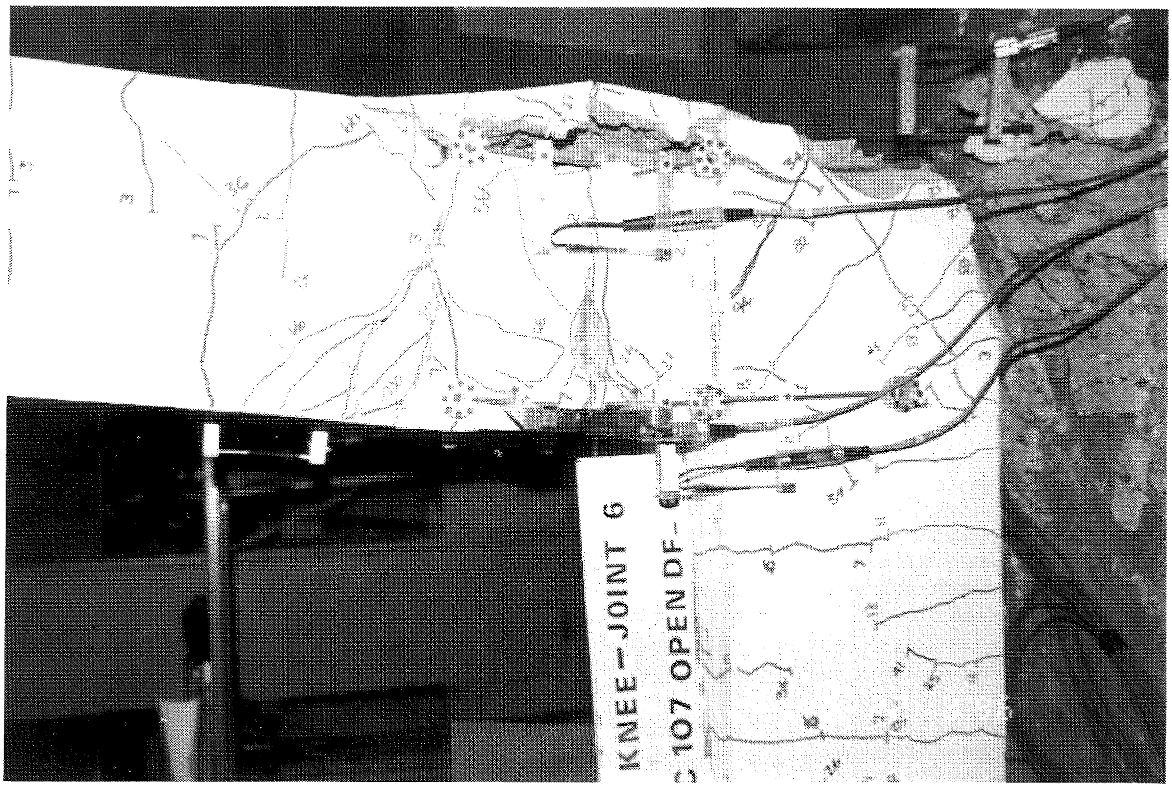

Figure 21: Knee Joint 6 at opening structural ductility factor = -6.

Beam hinging occurred during the first opening cycle to ductility -2, while joint splitting cracks occurred around the back and top of the joint, on the line of the outer column bars. Extensive minor cracking formed at the joint's top. During the ductility ±2 cycles the column face crack widened to about 2 mm, while under closing conditions the crack 150 mm out from the column face was about 4 mm wide.

At the opening ductility -4 cycle four beam cracks had opened to widths greater than 1 mm, while during the closing ductility 4 cycle the first diagonal joint cracks formed. During the second opening ductility -4 cycle the cover concrete began to spall from the back and top of the joint and the opposing diagonal cracks formed. From this point on main bar slip occurred within the joint, resulting in very little stiffness in the joint at low force and ductility levels. The column face crack opened to a width greater than 10 mm at ductility 4 in the closing direction, while the other beam cracks now remained closed. In the opening direction the diagonal bars enhanced the strength and allowed the beam hinge to continue forming with 4 beam hinge cracks opening, while strengths greater than nominal were still being attained.

In the ductility ±6 cycles the remaining top and back joint cover fell off and up to 20 mm of bar slip was apparent at the column/joint interface, in the closing direction. In the second opening cycle to ductility -6 the moment reduced to 90% of M_n, due to the loss of cover concrete at the outer column face causing a decrease in the beam's effective depth. This moment level was maintained at the subsequent opening ductility -8 and -11 peaks.

At the end of the test the joint concrete within the core looked secure, with most of the side cover still in place and the diagonal cracks only about 0.5 mm wide. The main reason for the worsening performance in the joint in the closing direction was the loss of anchorage to the outer beam and column bars. The opening behaviour was excellent, at least up to ductility factor -6, with only a small decrease in strength at higher ductilities.

3.8 Knee Joint 10:

The only detail differences between joints 9 and 10 was that joint 10's principal beam and column reinforcing was anchored within the joint with standard 90-degree hooks, but because of a shortage of space the specified $12d_b$ (192 mm) straight tail was reduced to 150 mm. To provide the full $12d_b$ tail would have necessitated short beam and column stubs.

The applied force versus beam-tip deflection hysteresis loops are reproduced in Figure 24. Performance under opening bending moments was excellent, with the nominal strength being exceeded up to and at ductility -10 displacements. However under closing conditions there were strength reductions at the second cycle to ductility 4 and beyond. The maximum strength efficiencies (M_{test}/M_n) reached were 1.02 and 1.20 under closing and opening moments, respectively, this being about a 5% better performance than that of knee 9.

The reduction in strength at closing ductility 4, second cycle, can be partially explained by the beam concrete crushing at the re-entrant corner. When the cover concrete is ignored, the theoretical beam section strength reduces to 42.5 kNm (M_n = 45.6 kNm for gross section) and the experimental closing moment sustained at this point was approximately 38 kNm. Some bar slip was also probably occurring, which would have stopped the outer bars reaching their yield stress.

During the second opening cycle to ductility -2 the outer corner of the joint began to spall, while in the ductility ±4 cycles the column face concrete crushed and the back and top joint cover began to fall off near the column face. Under opening conditions a full beam plastic hinge had formed with 5 main cracks opening up over a length of about 400 mm starting from the column face.

In the ductility ±6 cycles most of the back and top joint cover spalled and substantial bar slip was occurring, especially to the beam bars, with the associated reduction in load carrying capacity under closing moments. The outer top beam cover for a length of 100 mm from the column face was also loose and fell off in the ductility 10 cycle. No further diagonal joint cracking occurred during the test and the joint core concrete was secure at the end of testing.

3.9 Knee Joint 14:

Figure 25 shows the force- beam tip deflection loops for knee 14, while Figure 26 is the M_{test}/M_n ratio envelopes versus the ductility factor for knee joints 6, 9, 10 and 14; the units with the extra 2-D12 bars across the joint's re-entrant corner. Knee 14 exhibited excellent ductile behaviour up to ductility 4 in each direction. The maximum M_{test}/M_n values reached were 1.17 and 1.36 under closing and opening moments, respectively. At ductility 4, closing for the second time, the force carried dropped to $0.98M_n$, while at ductility -6 for the first time in the opening direction, the force reached fell to $1.20M_n$. These force reductions occurred after the outer corner of the joint spalled, resulting in an anchorage failure, which in turn allowed the outer beam bars to slide back and forth. In the opening moment direction the inner beam cover had crushed, reducing the effective depth and thus reducing the nominal beam moment.

There was a continual drop in strength attained for each subsequent cycle, as the outer bars continued to slip to a greater degree. The addition of the double transverse bars in the bends probably would have reduced the slip at the lower ductilities (see knee 9) but otherwise the behaviour of knees 9 and 14 were very similar with higher initial strengths being reached in knee 14 at ductilities 2 and 4, in both directions.

The first diagonal joint cracks formed during the first closing cycle to ductility 2, while the opposing diagonal crack formed in the next half cycle to ductility -2. However the cracks were very fine and remained that way throughout the test. The first crack across the outer corner also formed in the same half cycle. The major top beam cracks occurred about 100 mm out from the column face, where the extra joint diagonals terminated.

Beam hinging continued at least up to ductility ±4 over a hinge length of about 300 mm. By the first cycle to opening ductility -4 it was obvious that the outer corner was being pushed off, with a major splitting crack on the centre-line of the column bars around the 90-degree bend.

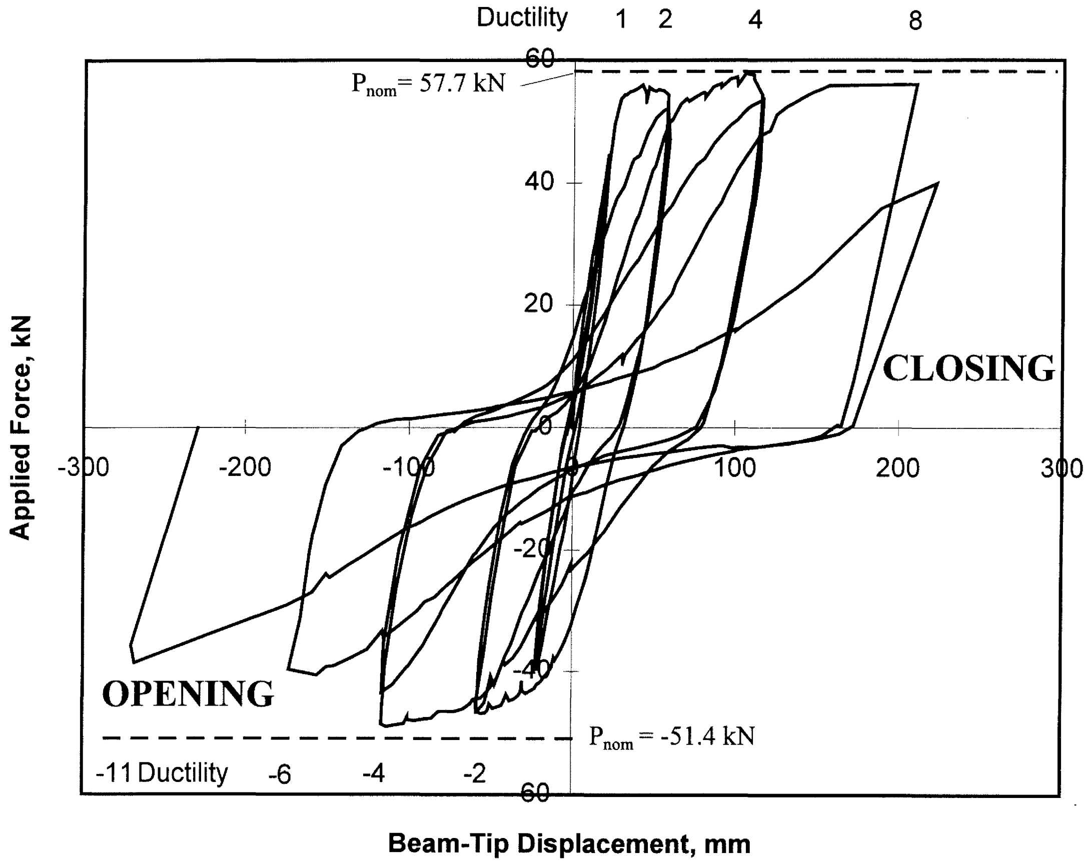

Figure 22: Knee 7 Applied Force - Beam Tip Deflection loops.

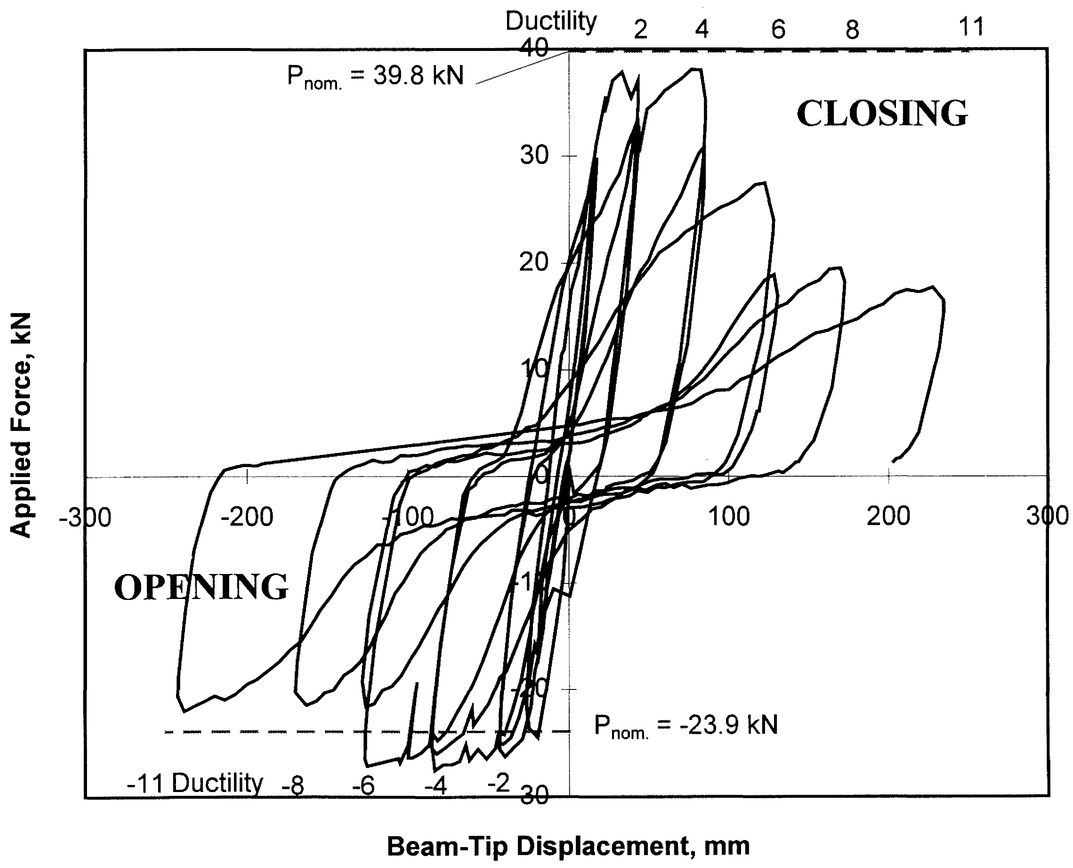

Figure 23: Knee 9 Applied Force - Beam Tip Deflection loops.

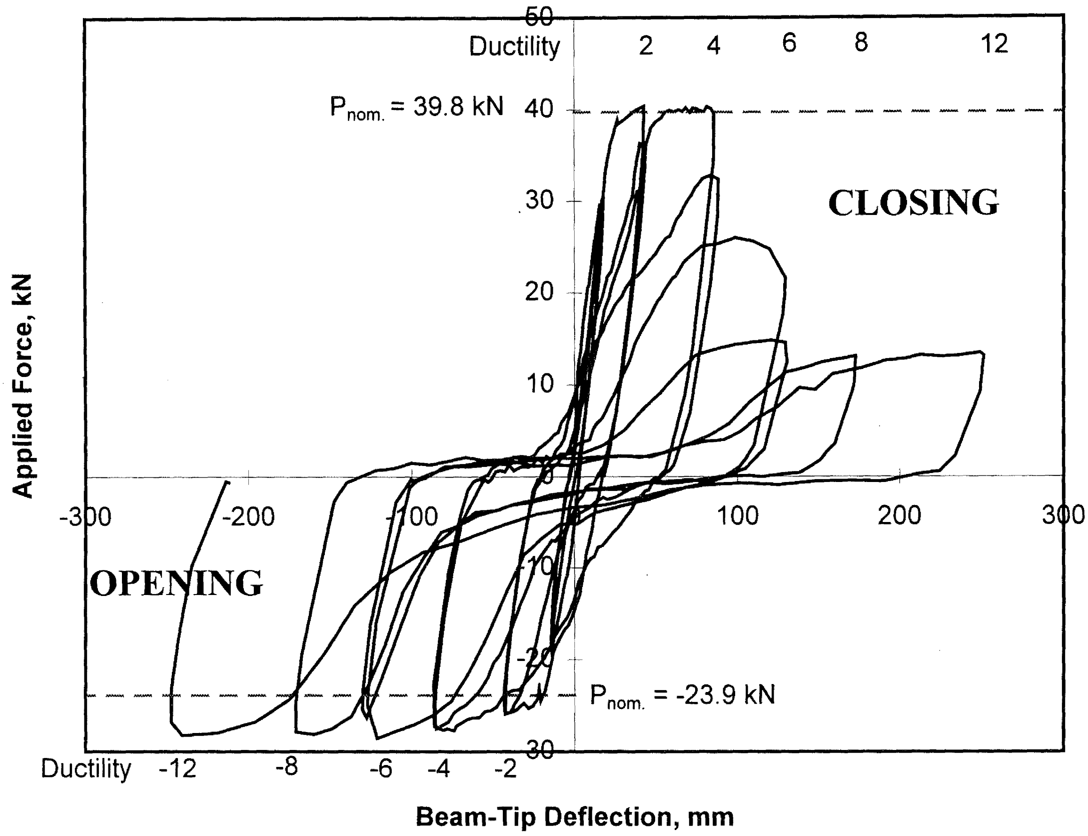

Figure 24: Knee 10 Applied Force - Beam Tip Deflection loops.

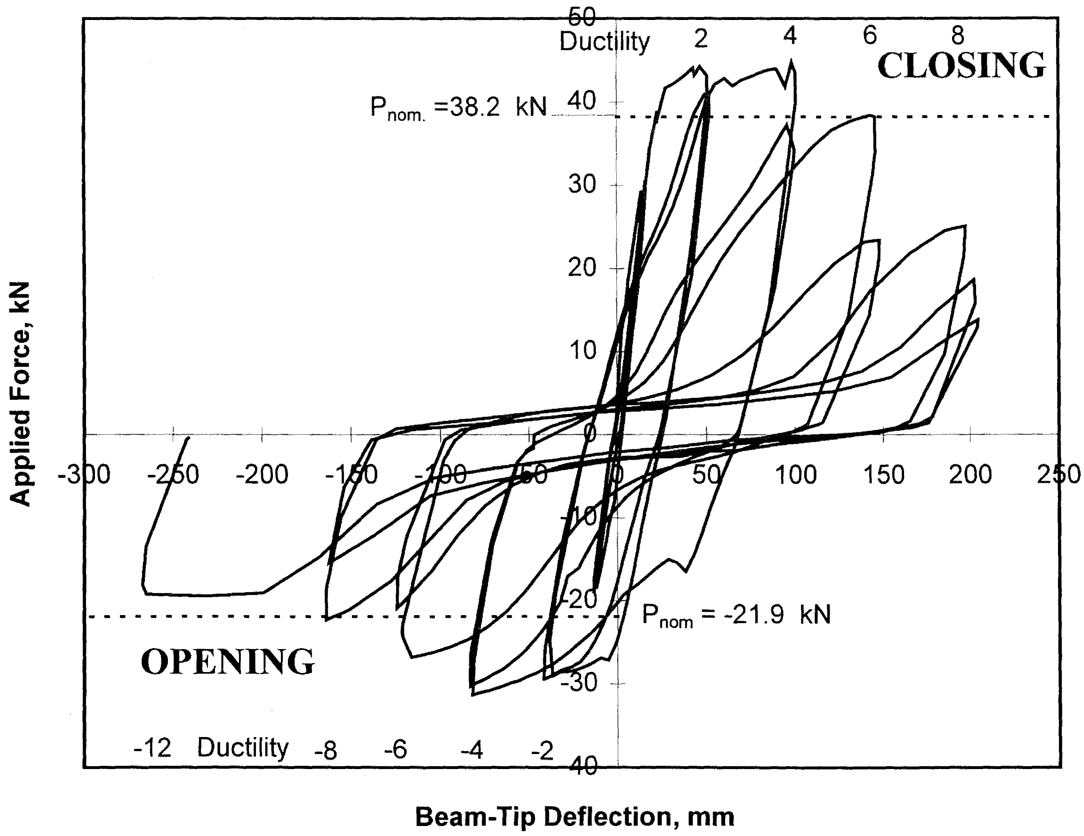

Figure 25: Knee 14 Applied Force - Beam Tip Deflection loops.

4. RESULTS OF 1960's KNEE JOINTS

4.1 Knee Joint 5:

This joint had R16 plain round bars as principal beam and column reinforcement. The only joint shear reinforcement was a single horizontal 4ϕ drawn wire tie.

The first reversed cycle to about 75% of the nominal moment to cause yielding at the column face occurred without incident. However when attempting to reach the same moment for the second time in the opening direction, two diagonal joint cracks suddenly opened up and a maximum moment of only 67% of M_n was attained.

Upon reversing the load direction the knee joint sustained a force of 26.5 kN (equivalent to 65% of M_n) before joint diagonal cracks opened in the opposite direction and there was a subsequent reduction in strength to about 35% of M_n. Continued cycling at displacement ductilities of ±2 and ±4 saw strengths of only between 30 and 40% of the nominal moment being reached. The maximum strengths reached in this test were only 71% and 67% of the nominal beam moments under closing and opening actions, respectively. This low strength level was expected as $pf_y / \sqrt{f_c'}$ = 0.833 (greater than the 0.5 value predicted for M_{test} to at least equal M_n), when compared with the poor performance of similar joints unreinforced for shear, studied in Europe in the 1970's, under monotonic forces, Megget [5]. The full force-displacement loops are shown in Figure 27.

Some minor beam and column cracking occurred in the first cycle but the predominant damage was in the joint zone where a premature shear (diagonal tension) failure caused loss of anchorage to the inadequate beam hooks and substantial slipping through the joint was then initiated. The majority of the beam-tip deflection was due to bar slip measured in the beam/column zone, while the rest came from bending and shear distortion of the joint region. The plastic hinge deflections were negligible, as expected, due to no yielding of the bars. The back corner of the joint fell off during the first ductility 4 cycle and the back joint cover was loose. It was obvious that the large diagonal cracks passed right through the joint and although the column face crack grew ever wider, it was due to beam bar slip and not yielding.

The maximum horizontal joint shear stresses were low at $0.072 f_c'$ $(0.42\sqrt{f_c'}$ (MPa)), closing and $0.04 f_c'$ $(0.23\sqrt{f_c'}$ (MPa)), opening. Priestley [23] recommended a maximum principal tensile stress of $0.29\sqrt{f_c'}$ (MPa) for exterior joints with beam bars bent <u>away</u> from the joint, as the bottom bars were here, while Park [24] used a maximum stress of $0.25\sqrt{f_c'}$ (MPa) for the same exterior joint case. The recommended principal tensile stress for bars bent down <u>into</u> the joint was $0.42\sqrt{f_c'}$ (MPa), as obtained here for the closing case where the top bars were indeed bent into the joint. The principal joint stresses in these tests were almost identical to the horizontal joint shear stresses, due to the axial column stresses being less than 10% of the joint stresses.

4.2 Knee Joint 8:

This joint was identical to knee joint 5 except that the principal beam and column reinforcement were deformed bars. The test behaviour of this joint was very similar to the previous 1960's joint, in that the maximum strengths reached under closing and opening moments were 77 and 81% of the nominal strengths, respectively. The improvement in both directional strengths was probably due to the higher compressive strength of the concrete (f_c' = 40.4 MPa) allowing a higher failure shear stress in the joint, rather than much improvement in the bond (anchorage) strength. As in the previous joint, this joint failed in shear before yielding occurred in the beam bars. In the first closing cycle to 0.75 of M_n one diagonal joint crack formed and in the opening portion of the cycle, while the force applied reached nearly 20 kN, it then dropped off quickly to about 16 kN as a large diagonal joint crack opened in the opposite direction to the crack which had formed in the previous cycle. A joint shear failure occurred at this time. On reversing the jack pressure, the force reached about -26 kN before reducing to -20.3 kN, with the formation of a long splitting crack around the joint's outer corner, a precursor to an anchorage failure.

During the first cycle to ductility factor ±2 the force applied continued to reduce till it reached only 28% of the nominal moment at ductility 2, while in the opening direction a higher strength of 78% of M_n was sustained at ductility -2. The final cycles showed a continual drop off in strength reached and the test was terminated at the end of the second cycles to ductility ±4. As in knee joint 5, the joint region was totally destroyed at the end of the test, with the beam bars slipping through the joint by a considerable amount. This joint is shown in Figure 28 at opening ductility -4, first cycle. Note the lack of flexural cracks in the beam plastic hinge zone, confirming the elastic behaviour recorded in that region.

The maximum joint shear stresses reached were $0.063 f_c'$ $(0.40\sqrt{f'_c}$ (MPa)) and $0.038 f_c'$ $(0.24\sqrt{f_c'}$ (MPa)) under closing and opening conditions, respectively. These stresses were a little less than those recommended by Priestley [23] for unreinforced exterior joints. It was expected that the shear stress limit for knee joints would be less than the equivalent exterior joint, which have a column above the joint adding some confining effects to the joint zone. Fuller details of the experimental behaviour of all the knee joints tested in this research project can be found in the Earthquake Commission Report by the author [26].

7. DISCUSSION

Table 3 shows a summary of results for all the knee joints tested. Included are the strength ratio, M_{test}/M_n, the material ratios, $pf_y / \sqrt{f_c'}$ and the maximum horizontal joint shear stresses, v_{jh} divided by both $\sqrt{f_c'}$ (MPa) and f_c'.

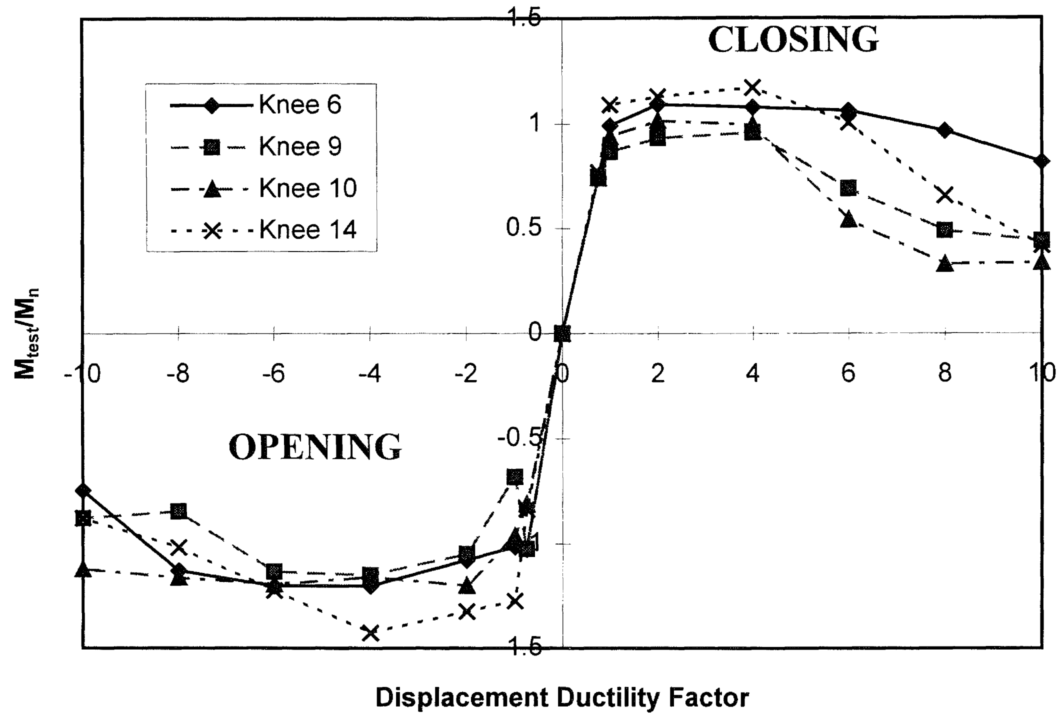

Figure 26: Applied Moment/Nominal Moment - Ductility Factor Envelopes for knee joints with diagonal bars across the re-entrant corner.

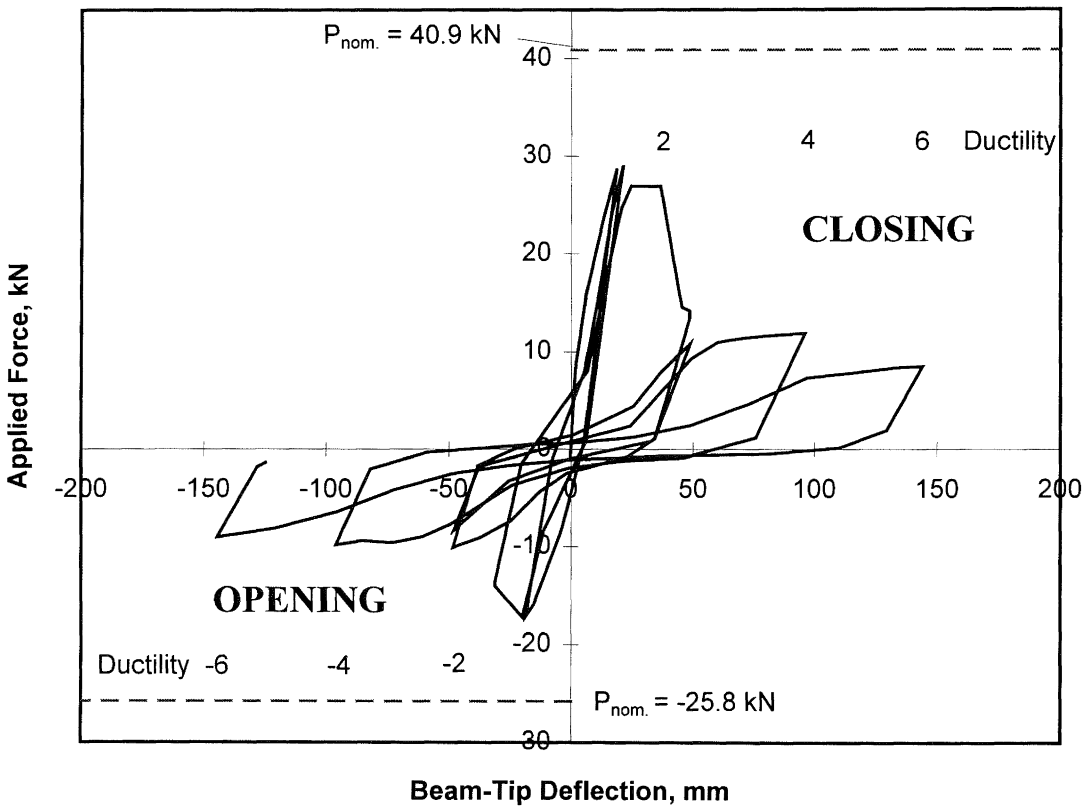

Figure 27: Knee 5 (1960's design) Applied Force - Beam-Tip Deflection loops.

TABLE 3: Main and transverse reinforcing details, Material Ratio, maximun $M_{test}/M_{nominal}$ and Joint Shear Stress Ratios for tested knee joints.

Knee Joint		Beam Bars Top Bottom	Transv. Bars in 90° bend?	Diagonal Bars	Horizontal Joint Ties, Vertical U-bars	$pf_y / \sqrt{f_c'}$	M_{test} / M_n maximum	$v_{jh} / \sqrt{f_c'}$ (MPa) max.	v_{jh} / f_c' max.
1	Close	4-D12 L	Yes		6-4 mm	0.686	**1.03**	0.486	0.092
	Open	4-D12 L			1-D10	0.686	**0.95**	0.414	0.078
2	Close	4-D12 U	Yes		6-4 mm	0.686	**1.02**	0.500	0.095
	Open	4-D12 U			1-D10	0.686	**0.97**	0.418	0.079
3	Close	3-D16 U	Yes		5-R6	0.764	**1.02**	0.554	0.095
	Open	3-D16 U			2-D10	0.764	**0.97**	0.447	0.077
4	Close	3-D16 U&L	Yes		4-R6	0.764	**1.00**	0.541	0.093
	Open	2-D16 U			2-D10	0.510	**1.06**	0.337	0.058
6	Close	3-D16 U&L	Yes	2-D12	5-R6	0.760	**1.09**	0.582	0.102
	Open	2-D16 U			2-D10	0.507	**1.20**	0.379	0.065
7	Close	3-D20 U	Yes		8-R6	1.009	**1.02**	0.723	0.102
	Open	3-D20 U			3-D10	1.009	**0.95**	0.598	0.085
9	Close	3-D16 U&L	No	2-D12	3-R6	0.718	**0.96**	0.488	0.077
	Open	2-D16 U			1-D10	0.479	**1.15**	0.341	0.054
10	Close	3-D16 L	Yes	2-D12	3-R6	0.712	**1.02**	0.517	0.082
	Open	2-D16 L			1-D10	0.479	**1.20**	0.357	0.057
14	Close	3-D16 U&L	No	2-D12+	3-R6	0.777	**1.17**	0.645	0.113
	Open	2-D16 U		2-D12jt.	1-D10	0.512	**1.36**	0.462	0.081
5	Close	3-R16 L	No		1-4 mm	0.833	**0.71**	0.415	0.072
1960's	Open	2-R16 L			Nil	0.555	**0.67**	0.228	0.039
8	Close	3-D16 L	No		1-4 mm	0.728	**0.77**	0.400	0.063
1960's	Open	2-D16 L			Nil	0.485	**0.81**	0.243	0.038

7.1 Joint Shear Stresses:

For a knee joint the closing horizontal joint shear stress is given by

$$v_{jh} = \frac{A_s f_y}{b_c h_c} \tag{3}$$

For beams and columns of equal depth and in these tests the beam width, $b = 0.8b_c$,

$$b_c h_c \approx \frac{b}{0.8}\frac{d}{0.9} \text{ then}$$

$$v_{jh} \cong 0.72 \frac{A_s}{bd} f_y \tag{4}$$

Priestley [8] recommended a maximum joint shear stress limit of about $0.4\sqrt{f_c'}$ (MPa) for unreinforced exterior joints with beam bars bent *into* the joint. Equating these two expressions for v_{jh} gives

$$p \le 0.55 \frac{\sqrt{f_c'}}{f_y} \tag{5}$$

For knee joints and under opening actions, where the joint shear stress will be reduced by the effect of any beam axial tension, equation (5) could be reduced to

$$p \le 0.50 \frac{\sqrt{f_c'}}{f_y} \tag{6}$$

That is, the same equation as described in the 1982 Code [6] for unreinforced knee joints.

The results of the majority of opening knee joint tests with conventional anchorage details are produced in Figure 29, including the cyclic tests completed in the United States (see section 1.1) and the 11 knee joint tests described here. The maximum strength ratios, M_{test} / M_{nom} are plotted against the "material ratio", $pf_y / \sqrt{f_c'}$ values noting that many of the non-cyclic test units had no transverse shear reinforcing in their joints. The line $M_{test} / M_n = 1$ is shown when the material ratio is less than 0.5 and the degrading strength line,

$$\left(M_{test} / M_n \right)_{\max} = 1.2 - \frac{0.4 pf_y}{\sqrt{f_c'}} \tag{7}$$

when $pf_y / \sqrt{f_c'} \ge 0.5$ is plotted. This degrading strength expression overestimated the strength ratio sustained by the two 1960's designed joints (5 and 8) under both opening and closing moments. Equation (7) could be used to estimate the likely maximum strength of unreinforced (for joint shear) knee joints constructed to codes of practice in use before the capacity design approach was incorporated into seismic concrete codes.

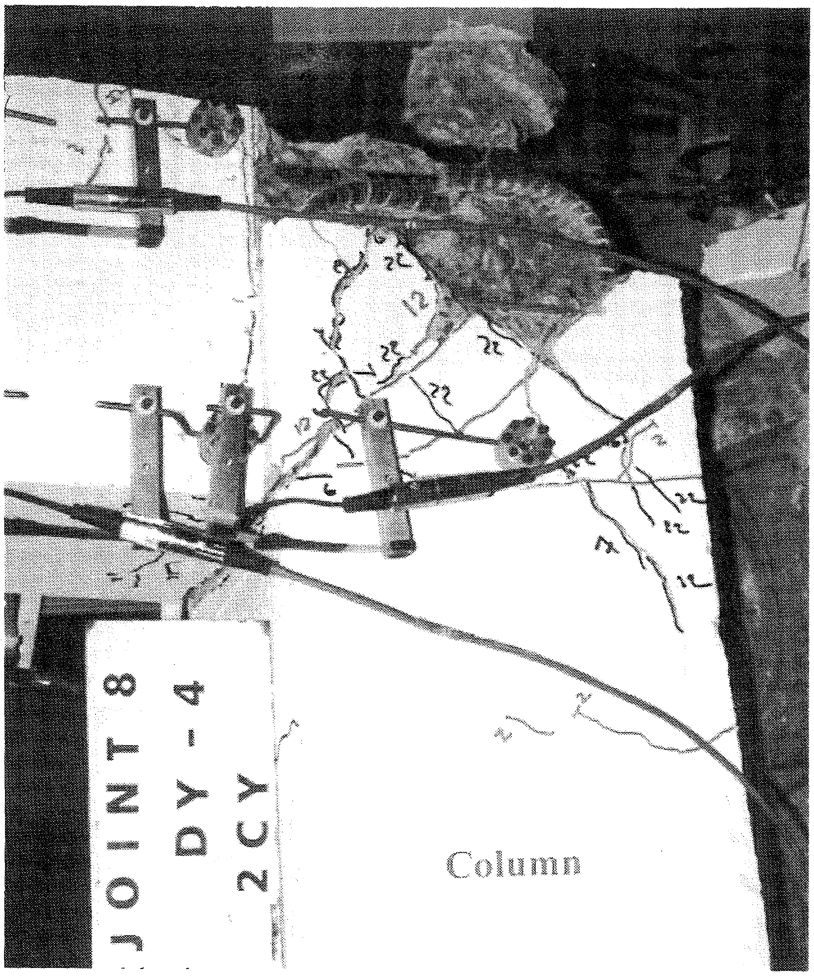

Figure 28: Knee Joint 8 (1960's design) at opening structural ductility factor -4, second cycle.

By including transverse joint shear reinforcing (horizontal ties and intermediate column bars or extra U-bars) the shear strength of the knee joint can be substantially increased above the limits quoted for unreinforced joints. The maximum joint shear stresses achieved in the tests described here were $0.102 f_c'$ and $0.723\sqrt{f_c'}$ (MPa) under closing conditions in knees 6 and 7 respectively. Under opening conditions the maximum was $0.085 f_c'$ or $0.60\sqrt{f_c'}$ (MPa) in joint 7, see Table 3.

From these small-scale tests it is recommended that the upper limit for v_{jh} should be about $0.12 f_c'$ for knee joints, considerably less than the $0.2 f_c'$ limit given in the current Concrete Standard [10]. For knee joints where the beam (or column) is expected to yield under positive, opening moments the joint shear stress limit should probably be restricted to $0.10 f_c'$. Larger shear stresses may be possible in large sections but these full-sized units have not yet been tested.

One important aspect that designers need more information about is how joints degrade during earthquakes with respect to joint shear stresses as ductility ratios increase. Figures 30 and 31 plot the ratio of the horizontal joint shear stress to the concrete strength (v_{jh} / f_c') against the displacement ductility factors during testing. Figure 30 is for the conventionally reinforced joints, including the two 1960's joints, while Figure 31 plots the knee joints with the extra diagonal bars across the joint's re-entrant corner.

The closing joint shear stresses in the non-conventional joints degrade to a greater extent than the conventional joints, as the ductility increases. The only feasible reason for this is that in the opening cycles the stresses reached are higher and cause more concrete cover damage, which in turn causes more anchorage loss in the outer bars under closing moments, thus reducing the shear stress sustained. The degradation of shear stress under opening moments shows little difference between the groups of knee joints. For conventional joints the maximum closing shear stress of about $0.1 f_c'$ decreased to approximately $0.075 f_c'$ at displacement ductilities of 8 or more. The degradation was much greater for the unreinforced 1960's joints, where the shear stresses decreased from a maximum of about $0.07 f_c'$ (closing) to about one third of that at ductility 4. Such joints will perform very poorly during a large earthquake, causing disintegration of the knee joint and possible collapse of the upper storey beams.

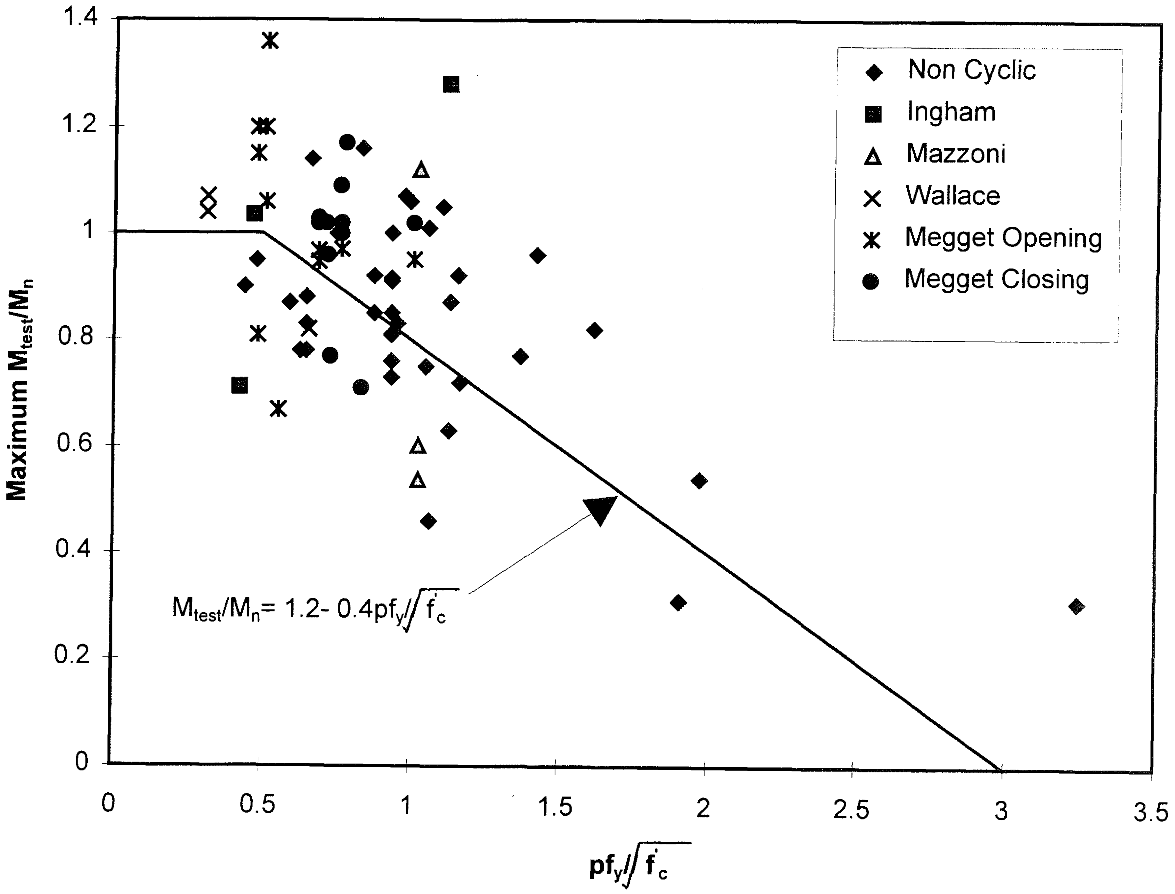

Figure 29: Opening Knee Joints maximum M_{test}/M_n versus Material Ratio for various researchers.

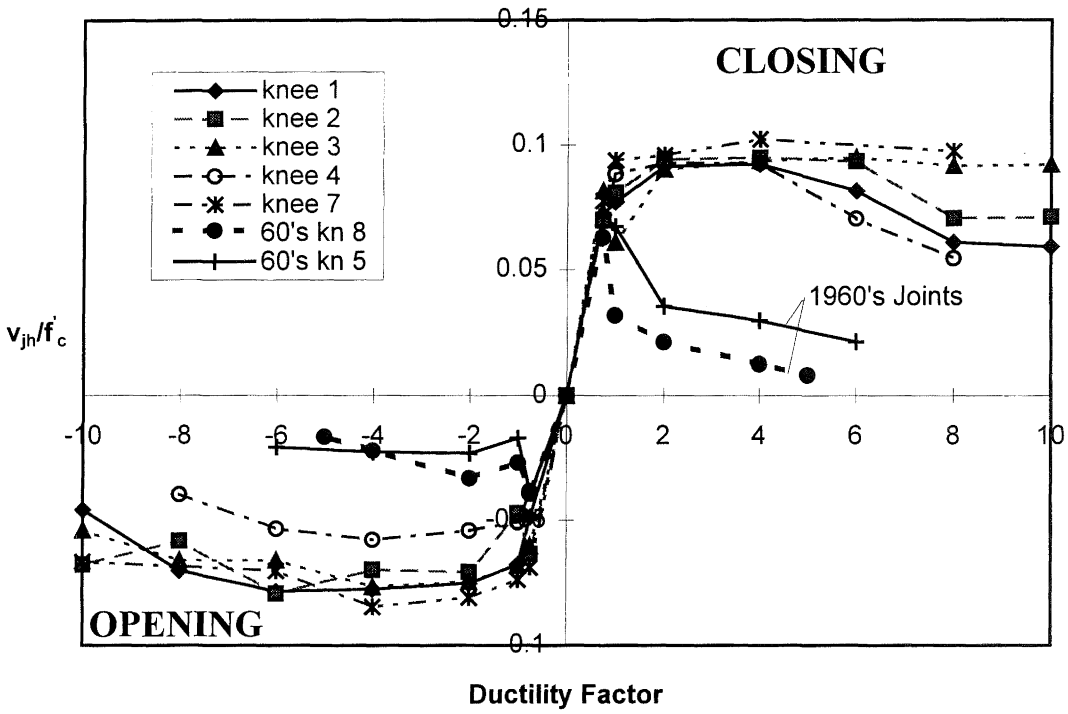

Figure 30: Joint Shear Stress/ f'_c versus Structural Ductility Factor for conventionally reinforced knee joints.

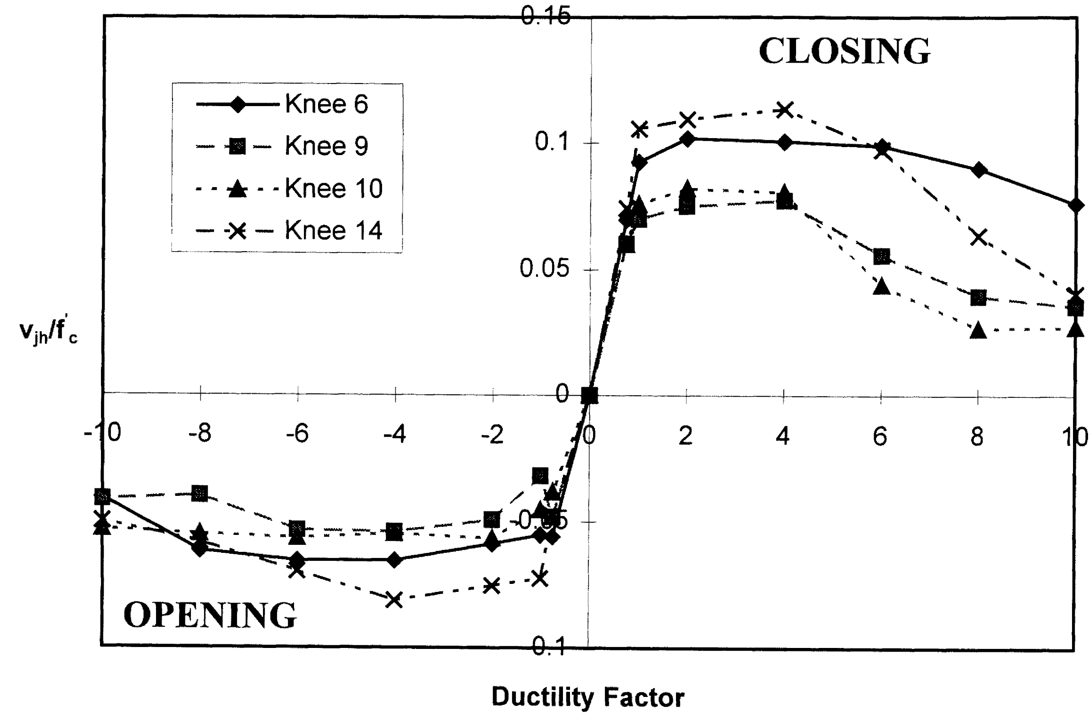

Figure 31: Joint Shear Stress / f_c' versus Structural Ductility Factor for knee joints with diagonal bars across the re-entrant corner.

7.2 Joint Shear Reinforcing:

As all the 1995 Concrete Standard designed joints reached within 5% or better of their nominal beam moment in both directions, it can be concluded that the amounts of horizontal and vertical joint shear reinforcing (A_{jh} and A_{jv}) specified in the NZ Standard [10] are marginally sufficient. Testing of joints with less than the Standard's requirements for A_{jh} would probably result in joint shear failures. A small increase (10-15%) in the horizontal joint shear ties may need to be recommended for knee joints with high levels of shear stress. Also these small knee joints only required one U-bar vertically in most cases and a better performance would have resulted if two U-bars (or extra intermediate column bars, suitably anchored) had been detailed in joints 1, 2, 9 and 10. Extra vertical joint bars would have helped restrain the anchorage of the top beam and the column bars at the top of the joint, where splitting and loss of joint concrete cover was the main problem in these knee joints. The larger number of horizontal joint ties required by the Standard effectively held the vertical tails of the beam bars and the column bar anchorage length at the outer joint face during up to 10 reversing yield cycles. The Standard should specify not less than 2 intermediate column bars or vertical tie legs in knee joints on each side of the joint in the plane of the frame being considered.

7.3 Main reinforcing bar anchorage:

The anchorage of the principal beam and column bars within a knee joint is difficult to sustain over several inelastic reversing cycles of moment. Due to the lack of a column above the joint and the very low axial column forces associated with knee joints, there is little confinement of the concrete at the top and outer joint faces. Splitting cracks occurred along the main bars at ductilities of about 2 and in the following inelastic cycles the cover spalled from the top and outer faces, resulting in the loss of anchorage and subsequent bar slip in and out of the joint. The use of U-bars instead of "standard hook" details gave better performance in the small knee joints tested. With a U-bar, once the anchorage is lost near the column face the force is anchored further along the U and at high ductilities the anchorage is often occurring near the second 90-degree bend.

The addition of the two transverse bars within the 90-degree bend did assist the diagonal compression strut to form and also restrained the beam bars from slipping in and out of the joint, to some extent. Knee 6 was similar to knee 9, but with the addition of the double transverse bars, its strength behaviour was better overall, with the maximum strength sustained in both directions being more than 5% greater than that reached in knee 9. Knee 10, identical to 9 except for standard hooks and the transverse bars also sustained maximum moments 5% greater than knee 9 in both moment directions. The use of a larger number of small diameter main reinforcing bars is recommended in small section knee joints, even though the congestion within the joint will be increased. Satisfactory anchorage of the principal beam and column bars is more difficult in knee joints than in exterior joints and from this research it appears to be the critical consideration when detailing joints to sustain several reverse yielding cycles, without degradation of the joint occurring.

8. CONCLUSIONS

1. The seismic design requirements in the NZ Concrete Standard (NZS3101:1995), for an exterior beam-column joint's transverse shear reinforcement, gave satisfactory joint shear behaviour for small reinforced concrete knee joints under cyclic loading. A joint shear failure would result if the amounts of transverse joint reinforcing were reduced below those specified. In some tests the specified amount of vertical joint shear steel appeared to be the minimum required for shear failures not to occur. It may be advisable to increase the amount of vertical transverse reinforcing in small knee joints, if excellent ductile behaviour is required. A minimum of 2 vertical tie legs or suitably anchored intermediate column bars in each in-plane joint side is recommended.

2. A maximum joint shear stress of $0.12 f_c'$ under closing moments and a corresponding maximum of $0.10 f_c'$ under opening moments is recommended for small sized knee joints. The maximum joint shear stresses sustained were about $0.1 f_c'$ for small knee joints designed to the 1995 Concrete Standard. This is half of the specified maximum limit in the Standard. It is virtually impracticable to design for $0.2 f_c'$ joint shear stresses in small exterior joints, due to the limit on principal beam bar reinforcement ratios, $p_{max} = \dfrac{f_c' + 10}{6 f_y}$, where p_{max} cannot be greater than 0.025.

3. The anchorage of main beam and column bars in joints using continuous U-bars produced better inelastic cyclic behaviour (moment strength at specific ductilities) than "standard 90-degree hooks" with 12 bar diameter tails. Standard hooks tend to lose their anchorage earlier in knee joints, due to the splitting off of the joint's exterior corner, especially under closing moments.

4. The addition of double transverse bars within the 90-degree anchorage bends of the main bars improved the cyclic performance of the knee joints by enhancing the diagonal compressive joint strut and improving anchorage by increasing the bar's resistance to slipping out of the joint.

5. Additional diagonal bars across the joint's re-entrant corner increased the joint's opening moment strength by up to 20%, allowing a beam plastic hinge to form, rather than brittle degradation of the joint at strengths below the beam's nominal moment.

6. Joints with large main bars, which did not have the specified standard hook anchorage length, L_{dh} failed due to bond loss, and bar slip (virtually non-energy absorbing) became the predominant component of the joint's rotation.

7. Knee joints designed to the 1960's Code of Practice [18] failed to reach their nominal strength in both opening and closing directions and the strength sustained at displacement ductility ±2 were less than half of the corresponding nominal beam moment. The maximum closing joint shear stresses sustained were about $0.4\sqrt{f_c'}$ MPa for these virtually unreinforced joints (little horizontal and no vertical transverse ties). The corresponding maximum stress for opening moments was $0.24\sqrt{f_c'}$ MPa, which approximately agrees with Priestley's [23] recommendation for unreinforced joints with beam bars bent *away* from the joint.

8. A 25% drop in the joint shear stress sustained occurs between ductility factors of 1 and 8 for knee joints designed to the 1995 New Zealand Concrete Standard, while for joints designed to the 1960's Code the decrease is about 60% at ductility levels of 4. Very poor seismic performance can be expected from knee joints detailed to the 1960's Code, even during medium strength earthquakes.

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NOTATION

μ	Displacement ductility factor, (Beam-tip deflection/deflection when M_n first reached)
β	Ratio of area of compression beam reinforcement to that of tension beam reinforcement, but always ≤ 1
ϕ	Bar diameter
A_g	Gross column cross-sectional area
A_{jh}	Cross-sectional area of horizontal joint ties in beam-column joint
A_{jv}	Cross-sectional area of vertical transverse joint reinforcing
A_s	Area of main reinforcing bars in beam section
A_s'	Area of compression reinforcing steel in beam section
b	Width of beam section
b_c	Width of column section
C_j	Ratio of horizontal joint shear force in the direction being considered to the sum of that joint shear force and the joint shear force in the other orthogonal horizontal direction = 1, where there are no transverse beams entering joint
d	Effective depth of beam section
D	Deformed bar
d_b	Diameter of main bar (or diameter of tie in 1960's Code)
d_b	Depth of beam section
f_c'	Concrete compressive cylinder strength, MPa
f_y	Yield stress of main reinforcing steel
f_{yd}	Yield stress of additional diagonal bars across re-entrant corner
f_{yh}	Yield stress of horizontal joint shear ties
f_{yv}	Yield stress of vertical joint transverse reinforcing
H.T.	Heat treated (wire ties)
h_c	Depth of column section
L_d	Development length for reinforcing bar
L_{dh}	Development length of standard 90-degree hook
M_n	Nominal beam moment (actual beam axial force included in calculation) using actual reinforcing steel and concrete material properties
M_{test}	Moment reached at critical beam section during test
N^*	Axial column force, negative when tensile
p	Tension reinforcement steel ratio, A_s/bd
p'	Compression reinforcement steel ratio, A_s'/bd
p_{max}	Maximum reinforcement ratio specified in code
R	Plain round bar
v_{jh}	Horizontal joint shear stress