THE SEISMIC DESIGN AND PERFORMANCE OF REINFORCED CONCRETE BEAM-COLUMN KNEE JOINTS

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SUMMARY

This paper describes the cyclic behaviour of eleven reinforced concrete beam-column knee joints, comprising 8 half-sized units and 3 full-sized joints. Nine of the knee joints were designed to the current New Zealand Concrete Standard (NZS3101 1995), while the remaining two joints were 1960's designs, designed to the pre-capacity design Code of Practice (NZSS 1900, 1964). The 1995 Standard's designed joints predominately behaved in a satisfactory manner, reaching their nominal flexural strength in both directions up to structural ductility 4 displacements. At higher displacement ductilities joint degradation occurred, usually due to the loss of bar anchorage within the joint, especially under closing conditions when the concrete cover at the top and back of the joint had spalled off. The small knee joints were only able to sustain maximum joint shear stress levels of 0.12 $f_c^{'}$ and 0.10 $f_c^{'}$ under closing and opening moments, respectively. The larger joints reached a maximum shear stress of about 0.20 $f_c^{'}$ (the Standard's limit) but joint shear failure occurred before the loss of anchorage. A 25% degradation in joint shear stress sustained occurred between ductility factors 1 and 8 for the 1995 designed joints (progressive cycles to displacement ductility ± 2, ± 4, ± 6 and ± 8).

The 1960's designs behaved poorly, reaching only about 70% of the beam's nominal bending strength in both loading directions, due to sudden joint shear failure. Maximum joint shear stresses of 0.07 $f_c^{'}$ under closing moments were reached but only 0.04 $f_c^{'}$ under opening moments. The joint shear stress continued to decrease with every reversal of simulated seismic force.

INTRODUCTION

The seismic behaviour of interior and exterior reinforced concrete beam-column joints has been extensively researched since the late 1960's (Wallace et al 1996). However the cyclic behaviour of knee joints, found at the top of multi-storey frames, in portal frames or at positions where the building is set back, has not been fully studied. Experimental studies by Mazzoni, Moehle and Thewalt (1991), Cote and Wallace (1994) and McConnell and Wallace (1994a, 1994b) have only recently looked at the seismic cyclic behaviour of knee joints designed to the ACI 318 Code (1991) . These studies concluded that the maximum joint shear stresses likely to be carried were less than 70% of the limiting ACI Code requirement of 1.0 $\sqrt{f_c^{'}}$ (MPa), the limiting shear that the joints were actually designed for. Wallace et al. (1996) concluded that the ACI Code was "unconservative" for knee joints without transverse beams, which are assumed to confine the joint concrete somewhat.

Study of the seismic performance of knee joints designed to the 1995 NZ Concrete Standard (NZS3101 1995) had not been attempted prior to the series of tests described in this paper. The aim was to test small and full scale joints either with U-bars or "standard 90-degree hooks" beam bar anchorage, with the horizontal and

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vertical transverse joint reinforcing as specified by the current Concrete Standard's design equations. Some joints had extra diagonal bars across the joint's inner corner to improve the opening moment strength.
Two 1960's designs were tested to ascertain the behaviour of these non-seismically designed joints in a major earthquake, in an attempt to assign shear strengths to such joints in existing framed structures at different levels of structural displacement ductility.

TEST SPECIMEN DESIGN

The eleven small-scale joints (Nos. 1-10 and 14) had beams 250 mm deep and 200 mm wide, with columns 250 mm square. The three full-sized joints (A, B and C) had beams 500 mm deep by 200 mm wide intersecting with a column 520 mm deep by 300 mm wide. Table 1 lists the main and transverse reinforcement details for all the knee joints, as well as the material properties (concrete compressive strength at time of testing and reinforcing bar yield stresses). Joints 1, 2 and 7 had an equal number of top and bottom beam bars, while in all other units (except joint C) the ratio of top to bottom bars was 3 to 2. Where the bar anchorage was by U-bars, which continued from beam top to bottom bars, the middle top bar was anchored with a standard 90-degree hook with a 12 bar diameter extension beyond the end of the bend. The exception was knee 10 which had L-bars throughout with an extension of only $9.4\,d_b$ (150 mm), due to the lack of space when anchoring the bottom bars up into the joint. In joint C the beam reinforcing was distributed evenly down the beam depth at 85 mm centres, with these bars anchored with horizontal U-bars, viz. left side beam bar became a beam bar on the right side. The U was placed as far as possible to the back (outside) of the joint, in all joints. For joint C one 6 mm tie-set was placed between alternate HD16 beam bars. In all the other joints the total number of horizontal tie-sets shown in Table 1 were placed between the top and bottom beam bars, as specified in the Standard. In most cases the lower tie-set, which was positioned next to the bottom beam bars was not included in the amount of horizontal transverse joint ties, A_{sh} , required by the design equation. Figure 1 shows the reinforcing layout and dimensions of knee joint 6, with its two extra diagonal bars across the re-entrant corner.

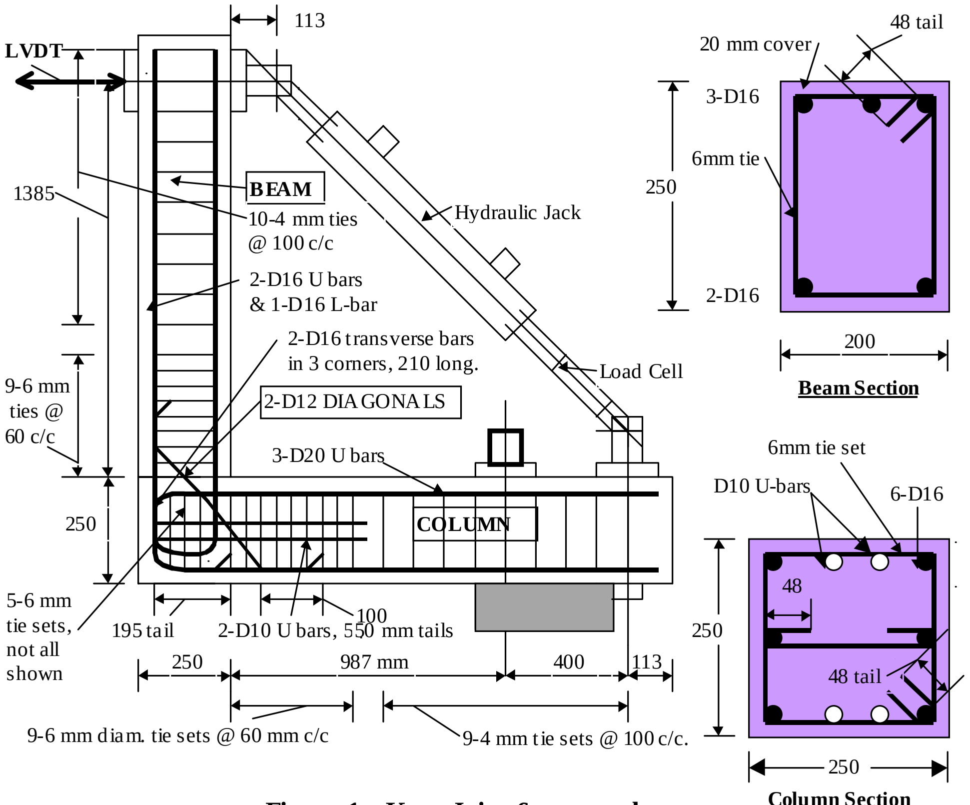

Figure 1: Knee Joint 6 as tested

The two additional short transverse bars (same diameter as the beam bars) positioned inside the 90-degree anchorage bends of the beam and column bars were to assist with the formation of the joint diagonal compressive strut. The 1995 Standard also allows a 20% reduction in the horizontal development length (L_{dh} = distance from near the column face to the beginning of the 90-degree bend The basic L_{dh} calculated for D12, D16 or D20 bars was greater than the length available in the small-scale joints with small column depth.

The column was only slightly stronger than the beam due to the 50 mm wider column in the small-scale joints. A larger flexural column strength is not specified because the Standard does not require plastic hinges to form in the beams at the top of ductile frame structures. The beam stirrups and column ties adjacent to the joint were designed to the potential plastic hinge zone requirements in the Standard (tie spacing less than $6\,d_b$, where d_b is the main beam bar diameter) to preclude buckling when the bars are yielding in compression. Knee joint 14 had two additional diagonal bars through the centre of the joint to improve the closing behaviour of the unit.

The 1960's designs had 90-degree hook for both the beam and column main bar anchorage but the tail beyond the bend was only $4\,d_b$ (64 mm) long. Also the bottom beam bars were bent down into the column near the back of the joint, as allowed in the 1964 Code. No requirement for joint transverse reinforcement was included in the 1964 Code and hence only one 4 mm diameter tie was provided. Beam stirrups consisting of 4 mm ties @ 180 mm c/c (≤0.75 of beam depth) were provided because the shear stress on the concrete was less than 0.03 $f_c^{'}$.

Large joints A and B, and small joint 7 were designed so that the maximum joint shear stress, v_{jh}, approached the limiting value given in the 1995 Standard, 0.2 $f_c^{'}$. However the concrete strength at testing of joint 7 was 65% greater than specified and thus the shear strength reached was only about half of the limit under closing moments. Many of the smaller joints were designed to carry a joint shear stress of about 0.1 $f_c^{'}$ or less, a stress magnitude common and often the practical limit in many knee joints in ductile frames.

LOADING SEQUENCE

Each test specimen was loaded with an identical cyclic programme. The joints were loaded for two "elastic" cycles to about 75% of the theoretical nominal moment at the column face, M_n, in both the closing and then the opening moment direction. From the beam-tip deflections reached, the first yield deflection was estimated and the following displacement controlled cycles to displacement ductilities of ±2, ±4 and ±6 were completed. If at this point the joint continued to carry a moment greater than about 80% of M_n, then a further cycle to ductility ±8, and in some cases an extra cycle to ±10, were attempted. In knee C as the $0.75\,M_n$ moment would be reached *after* yielding of the top or bottom row of beam bars, the first "elastic" cycles were taken to only half the nominal moment and the yield displacement extrapolated from there taking into account the curved yield plateau. This was caused by 3 rows of beam bars progressively yielding as the moment was increased. The yield deflection in knee C was taken as 33 mm, compared with 40 and 45 mm assumed for knees A and B with their larger amounts of beam reinforcing. The small joints had a first yield deflection of about 25 mm in most cases.

The joints 1-10 and 14 were loaded with a diagonally positioned hydraulic jack, which not only provided the beam and column bending moment but also applied a small axial compression to the beam and column under closing actions and a tensile force under opening moments. The large units were loaded with a force perpendicular to the beam's axis, thus applying an axial force to the column only. The beam's nominal moment was calculated taking the axial beam force into account for the smaller joints. A full description of the small knee joint test results and design details are given in Megget (1998).

TEST RESULTS

Moment Strength:

The envelopes of the ratio of test maximum moment to the nominal beam moment at each cycle peak are plotted against the beam-tip displacement ductility for each knee joint in figs. 2 and 3. Figure 2 is for the conventionally reinforced joints (1, 2, 3, 4, 7 and A) as well as the 1960's joints 5 and 8. Figure 3 shows the non-conventional joints (6, 9, 10 14, B and C) with either extra diagonal bars across the inner joint corner or distributed beam bars in joint C.

For all joints the maximum moment sustained approached or exceeded the nominal beam moment by ductility 2 displacement in both directions, except for the 1960's joints which only reached a strength of about 0.7 M_n and subsequently the strength fell away rapidly, due to joint shear failure. The 1995 Standard designed joints continued to reach moments close to M_n in the ±4 ductility cycles and then began to decrease in strength during the ductility 6 cycles (closing), as the cover concrete was lost from the back and top of the joint. This concrete loss resulted in anchorage degradation in the top beam bars, thus reducing the strength able to be sustained in later cycles.

Table 1

Knee Joint	**Beam Bars** Type Transv Top Bottom Bars f_y (MPa)	**Column Bars** f_y	**Joint** Ties Horiz. Vert. U bar f_{yh} f_{yv}	**Diagonal bars** f_{yd}	**Concrete Strength** $f_c^{'}$ (MPa)
1	4-D12 4-D12 L Yes 358	8-D12 358	6-4 mm 1-D10 266 318	----	27.8
2	4-D12 4-D12 U Yes 358	8-D12 358	6-4 mm 1-D10 266 318	----	27.8
3	3-D16 3-D16 U Yes 328	6-D16 328	5-6 mm 2-D10 378 343	----	34.0
4	3-D16 2-D16 U & L Yes 328	6-D16 328	4-6 mm 2-D10 378 343	----	34.0
5 1960's	3-R16 2-R16 L No 355	6-R16 355	1-4 mm Nil 537	----	33.6
6	3-D16 2-D16 U & L Yes 324	6-D16 324	5-6 mm 2-D10 365 337	2-D12 355	33.6
7	3-D20 3-D20 U Yes 333	6-D20 333	8-6 mm 3-D10 378 337	----	50.0
8 1960's	3-D16 2-D16 L No 340	6-D16 340	1-4 mm Nil 537	----	40.4
9	3-D16 2-D16 U & L No 333	6-D16 333	3-6 mm 1-D10 322 337	2-D12 345	39.8
10	3-D16 2-D16 L Yes 333	6-D16 333	3-6 mm 1-D10 322 337	2-D12 345	39.7
14	3-D16 2-D16 U & L No 325	6-D16 325	3-6 mm 1-D10 322 337	2-D12 + 2-12mid-jt. 345	32.4
A	3-HD28 2-HD28 U & L Yes 479	6-HD28 479	6-R10 sets 1-HD20 331 477	----	38.1
B	3-HD28 2-HD28 U & L Yes 479	6-HD28 479	6-R10 sets 3-HD12 331 486	2-HD20 477	28.2
C	12–HD16 distributed U Yes 472	6-HD24 453	5-R10 sets 3-HD12 331 486	----	41.9

The opening moment performance was usually better than the corresponding closing strength, due to better anchorage of the bottom beam bars in the less damaged lower portion of the joint, where the column intersected.

Moments greater than about 80% of M_n were carried up to the opening ductility 10 cycle. The joints with the extra diagonal bars tended to have higher M_{test} / M_n values at ductility 2 and 4 cycles, due to the extra strength contribution of these bars (neglected in the M_n calculations). The reduction in strength was similar to the conventional joints at higher opening ductilities. The reduction in strength under closing moments for the non-conventional joints was greater than for the conventional joints, due to the greater damage and anchorage loss sustained in the earlier opening cycles (greater opening moment resulting in wider cracks and greater spalling of the cover concrete). The knee joint with distributed beam bars (C) behaved very well up to ductility ±4 levels but then deteriorated quickly, due to core concrete loss in the beam plastic hinge and buckling of the small HD16 beam bars in compression. Large knee B with diagonal bars behaved relatively poorly due to a diagonal joint shear failure occurring during the ductility 2 cycles. A joint shear failure did not occur in joint A due to it having concrete 35% stronger than that used in joint B.

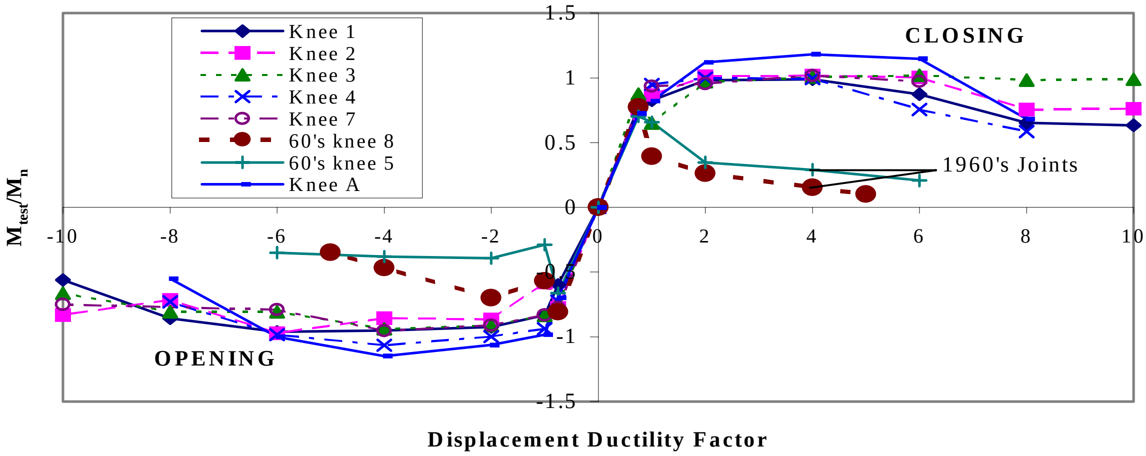

Figure 2: Applied Moment / Nominal Moment envelope for conventionally reinforced knee joints, including the two 1960's joints.

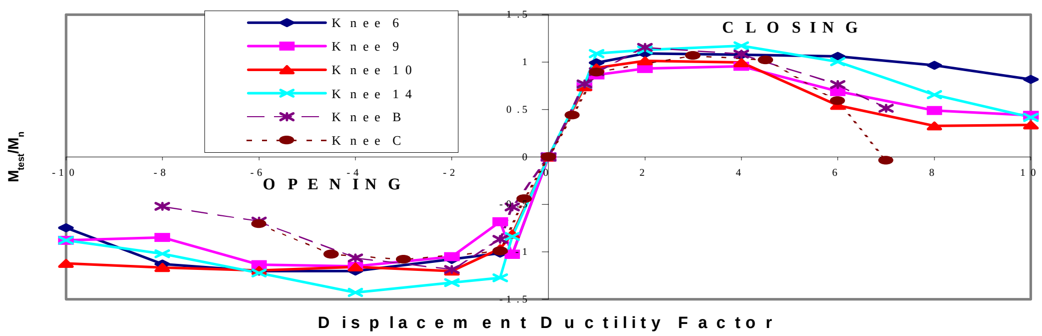

Figure 3: M test/M n for non-conventional knee joints with diagonal bars across re-entrant corner, except for Knee C with distributed beam bars (over beam depth).

Joint Shear Stress:

The NZ Concrete Standard (1995) specifies a maximum nominal joint shear stress of 0.2 f_c', to restrict the diagonal compressive stresses in the joint core concrete. The corresponding nominal joint shear stress in the 1991 ACI Code committee recommendations is $1.0\sqrt{f_c'}$ (MPa) for type 2 corner joints of ductile frames. A limiting value of 42 MPa for the compressive concrete strength is specified for use in the equation for the joint shear force. The earlier 1982 NZ Code specified a limit of 1.5$\sqrt{f_c'}$ (MPa) but this was reduced in the 1995 Standard, where v_{jh} was directly proportional to f_c', due to the failure mechanism in joints with shear ties being usually diagonal compression, rather than diagonal tension failure (proportional to $\sqrt{f_c'}$) prevalent in unreinforced joints. Table 2 gives the maximum test moment to nominal moment ratios and the maximum joint

Table 2

Knee Joint	M_{test}/M_n maximum Close	Open	$\frac{A_{jh}}{A_{jh}(Code)^*}$	$v_{jh}/\sqrt{f_c'}$ (MPa) max. Close	Open	v_{jh}/f_c' max. Close	Open	Type of failure
Conventional 1	1.03	0.95	0.98	0.49	0.41	0.092	0.078	Joint Shear
2	1.02	0.97	0.98	0.50	0.42	0.095	0.079	Bond
3	1.02	0.97	1.11	0.55	0.45	0.095	0.077	Bond
4	1.00	1.06	1.32	0.54	0.34	0.093	0.058	Bond
7	1.02	0.95	1.01	0.72	0.60	0.102	0.085	Shr & Bond
A	1.18	1.15	1.09	1.08	0.71	0.176	0.114	Bond
1960's 5	0.71	0.67	No Code ties	0.42	0.23	0.072	0.039	Joint Shear
1960's 8	0.77	0.81	No Code ties	0.40	0.24	0.063	0.038	Joint Shear
Non-conventional 6	1.09	1.20	1.71	0.58	0.38	0.102	0.065	Beam Cover loss
9	0.96	1.15	1.27	0.49	0.34	0.077	0.054	Bond
10	1.02	1.20	1.27	0.52	0.36	0.082	0.057	Bond
14	1.17	1.36	1.13	0.65	0.44	0.113	0.077	Bond
B	1.16	1.19	1.09	1.24	0.85	0.233	0.160	Joint Shear
C	1.07	1.08	1.02	0.80	0.81	0.124	0,125	Bar buckling

* A_{jh} neglects the joint tie-set touching the bottom beam bars and A_{jh} (Code) calculated using f_y = 300 or 430 MPa and f_c' = 30 MPa.

shear stresses divided by both $\sqrt{f_c'}$ and f_c' in the closing and opening directions for each knee joint. The ratios of the actual amount of horizontal joint tie reinforcement (A_{jh}) provided to the amount of ties required using the 1995 Code design equation are also included. The contribution to the joint shear force of the extra diagonal bars in the non-conventional joints has not been included in the Code calculation of A_{jh}.

It is not only the maximum joint shear stresses which are of interest but the degradation of joint strength during a major earthquake is of particular importance to designers and consultants considering seismic retrofitting of buildings. This is especially so for joints in buildings built before "capacity design" techniques were codified. Figures 4 (conventional joints) and 5 (non-conventional) show the envelopes of joint shear stress divided by the concrete compressive strength at the first cycle peak at successive ductility factors, in both directions.

For the small-scale joints the decrease in v_{jh} / f_c' was small up to ductility 4 and –6 in the closing and opening directions, respectively. At larger ductility factors the joint shear stress reduced to between 70 and 80% of the maximum earlier reached. The large joint A reached its maximum joint stresses of approximately 0.18 f_c' closing and 0.11 f_c' opening at ductility ±4 with little reduction at ductility 6, but then a substantial decrease occurred at ductility 8 in both directions, as bond failure occurred in the top beam bars anchored in the joint.

The non-conventional joints showed similar tendencies with negligible drop off in v_{jh} under opening moments in the small joints, while the larger joints exhibited large reductions at ductility factors greater than 4 in both directions. The excessive reduction in joint C's shear strength was due to the buckling of the beam bars in the plastic hinge rather than any degradation within the joint, which remained virtually undamaged during the testing. However joint B's poor performance at ductilities greater than ±4 was due to joint shear failure; the maximum joint shear stress of 0.23 $f_c^{'}$ was unable to be sustained at high ductilities.

The two 1960's joints failed in joint diagonal tension and exhibited lower shear stress capacity as the ductilities increased. At ductility factor ±5 the joint shear stress was less than 0.02 $f_c^{'}$, considerably less than the 0.06 $f_c^{'}$ recommended for unreinforced *interior* beam-column joints (Hakuto et al. 1995). The max. joint shear stress carried by the knee joints described here was only about 50% of that recommended for interior joints by Hakuto.

CONCLUSIONS

A maximum joint shear stress of about 0.15 $f_c^{'}$ is recommended for beam-column knee joints when transversely reinforced as per the current NZ Concrete Standard (NZS3101 1995).

Unreinforced 1960's designed knee joints did not sustain maximum joint shear stresses greater than about 0.40 $\sqrt{f_c^{'}}$ (MPa) , failed due to joint shear and carried small decreasing shear stresses at ductilities >1.

Joints with extra diagonal bars across the re-entrant corner had improved opening moment strength but exhibited no better joint shear capability at higher ductilities, than the corresponding conventionally reinforced joints.

Damage to the concrete on the top and back of the knee joint usually reduced the anchorage to the top beam bars at ductilities greater than 4 and lead to closing moment strength degradation.

Transverse bars within the 90-degree bends improved bond and strength reliability, while continuous U-bars exhibited better cyclic strength than "standard hooks" in small joints.

The joint shear stress capability reduced to about 70% of the maximum at ductility factors greater than 6 in 1995 NZ Standard designed joints, in both moment directions.

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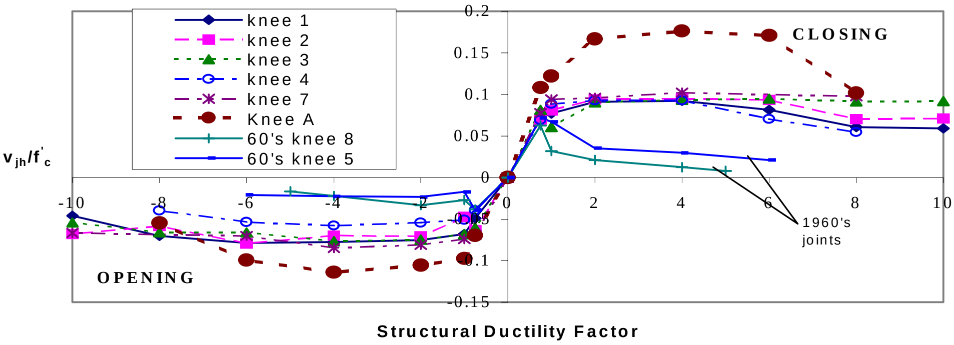

Figure 4: Joint Shear Stress/f'$_c$ versus Displacement Ductility for conventionally reinforced knee joints.

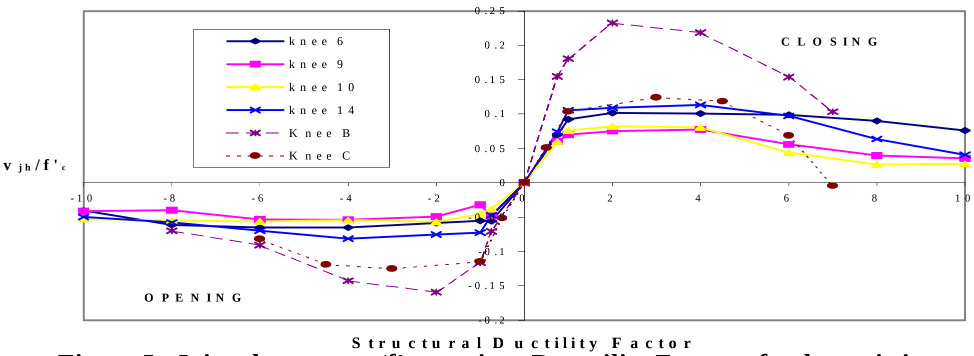

Figure 5: Joint shear stress/f'c against Ductility Factor for knee joints with extra diagonal bars across the re-entrant corner & knee C with distributed beam bars.