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The Seismic Behaviour of Small Reinforced Concrete Beam-Column Knee Joints

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

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ABSTRACT

The majority of research into beam-column knee joints has been under monotonic loading and many of these joints failed to reach their member moment capacity especially under opening moments. A few cyclic knee joint tests have been completed in the United States in the last five years. This report describes the cyclic testing of 11 small knee joints designed to the 1995 Concrete Standard including 3 joints with anchorage plates on the principal reinforcing bars. In addition two joints designed and detailed to the 1965 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 Standard (NZS1301:1995) designs were able to reach nominal moment capacity in both directions up to and including ductility 4 displacements, but subsequent strengths fell off at higher ductilities. Joints with extra diagonal bars across the inner corner were able to sustain their nominal members strengths to higher ductility levels. A nominal horizontal joint shear stress of $0.1f_c'$ (MPa) for knee joints, in ductile frame

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buildings is recommended. This limit is only half of the current NZS3101:1995 Standard recommendation. The 1960's designed joints behaved poorly, as expected, with joint shear and anchorage failures occurring, in both directions, at strength levels below the beam's nominal strength.

The two joints with anchorage plates attached to the ends of the "standard hook" anchorage failed to reach their respective nominal strengths in either direction, while the joint with an anchorage plate welded to each main U-bar sustained moments greater than nominal in both directions prior to ductility 2, but subsequently lost strength under closing moments during higher ductility cycles.

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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
α_v	$\dfrac{0.7}{1+\dfrac{N^*}{f_c' A_g}}$
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 treansverse 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
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)
f_c'	Concrete Compressive Stress, 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

h_b Depth of beam section

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} Maximum moment reached at critical beam section during test

N^* Axial column force, negative when tensile

p Tension reinforcing steel ratio, A_s/bd

p' Compression reinforcing steel ratio, A_s'/bd

R Plain round bar

v_{jh} Horizontal joint shear stress

1. INTRODUCTION

1.1 INTRODUCTION & OBJECTIVES.

Although considerable research effort has been concentrated on 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 multistorey frames or in portal frames, has been negligible. 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. Many differing anchorage and joint shear tie details were tested but generally the knee joints behaved poorly, especially under opening bending moments. In many cases the joints failed to reach their nominal member capacities before failing in the joint region, due either to shear failure (diagonal tension) or loss of anchorage to the beam 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. The addition of joint ties usually resulted in improvements in joint strength but did not guarantee enough 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 (Megget, 1994) and concluded that if attainment of the nominal member strength was required then the beam or column reinforcement ratio, p = A_s/bd would need to be less than about $0.5\sqrt{f_c^{'}}\ /\ f_y$, where f_y and $f_c^{'}$ are the reinforcing yield and 28-day concrete compressive stresses, respectively.

In fact, this is the same "conservative" limit recommended in the 1982 NZ Concrete Code Commentary (NZS3101:1982) for small knee joints under closing moments with no transverse joint ties for the concrete to 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 (1989) 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.

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), a limit suggested independently by Priestley (1993).

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 detail has not been acceptable for seismically loaded joints for three decades, as the bend out of the joint does not help to develop 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 (Megget, 1994).

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 Concrete Standard (1995), 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, (1975).

The amended equations allow a reduction in horizontal joint reinforcement, A_{jh} to 48% and 67% of the NZS3101 1982 Code requirements for knee joints with low axial column forces and joint shear stresses, $v_{jh} \leq 0.167 f_c'$ and for the maximum recommendation of $0.2 f_c'$, respectively. However for similar knee joints the vertical joint shear reinforcement, A_{jv} required lies 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 NZ designed exterior beam-column joints to the current levels of joint shear reinforcement were studied by Cheung et al (1991) 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 older 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 only a 6% decrease for 50 MPa concrete.

From the previous monotonic knee joint work and the few cyclic knee joints tested in the United States of America (see next section), 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 research project 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 (12).

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 (1991). 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 NZS3101 (1995) horizontal joint tie requirements. The only vertical joint steel was 2 - 19 mm diam. 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. The beam and column bars were anchored with standard 90-degree hooks within the joint. Ties had the conventional ACI anchorage of 135 degree hook one end and a 90 degree hook at the other. This tie anchorage arrangement is not permissable in the N.Z. Standard.

Both joints failed to reach their theoretical beam strength; maximum efficiencies (test moment/nominal 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 efficiencies 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 ductilities was more predominant under closing actions, due to the anchorage loss of the top beam bars and the outer column cars. 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 diam.) 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 (1971), 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°, due to construction difficulties, and were fixed at about 30° 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 programme as previously and formed a plastic hinge in the beam with little damage to the joint. The moment efficiencies (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 (1994) 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 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. The designs, except KJ3, fully complied with the then current American Concrete Institute (ACI) Concrete Code (1991). 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 approximately 4), when strain-hardening of 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 increases 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 efficiency, especially under closing moments by carrying the diagonal tension forces across the joint. The average joint shear stresses 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) 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 (1994a and 1994b) and Wallace, McConnell and Gupta (1996), 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 were 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 (1996) 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 this decade with separate maximum values of the joint shear stress shown for opening and closing actions.

TABLE 1 OTHER RESEARCHER'S CYCLIC KNEE JOINT TESTS

		Beam A_s, A'_s	Joint Ties	$\frac{M_{test}}{M_u}$	v_{jh} (MPa)	$\frac{v_{jh}}{\sqrt{f'_c}}$	$pf_y/\sqrt{f'_c}$	f_y (MPa)
Mazzoni	CLOSE	3 #6 top &	2 #3 ties	0.779	4.25	0.655		503
1	$f'_c = 42.1$	*btm*					1.027	
	OPEN	= 855mm^2		0.537	2.24	0.345		
Mazzoni	CLOSE	3 #6 *top &*	4 #3 ties	0.788	4.311	0.664		503
2	$f'_c = 42.1$	*btm*					1.027	
	OPEN	855mm^2		0.602	2.472	0.381		
Mazzoni	CLOSE	3 #6 *top &*	4 #3ties	1.12	5.572	0.79		503
Retrofit	$f'_c = 32.85$	*btm*	+2 #3 vert U				0.940	
	OPEN	855mm^2	+3 #4 diag	0.98	4.767	0.67		
McConnell	CLOSE	5 #6 top	4 #3 ties	0.879	3.461	0.604		455
Wallace	$f'_c = 32.85$	3 #6 btm	4 #3U vert				0.659	
KJ7	OPEN			0.816	1.493	0.261		
Cote Wallace	CLOSE	4 #5 top	3 #3 ties	1.027	2.211	0.327	0.604	
KJ1	$f'_c= 45.7$	2 #5 btm	4 #3U vert					448
	OPEN			1.038	0.573	0.085	0.302	
Cote Wallace	CLOSE	4 #5 top	3 #3 ties	1.048	2.26	0.320	0.585	
KJ2	$f'_c= 49.7$	2 #5 btm	4 #3 U-bars	1.24 ignores diagonal bars				448
	OPEN		2 #3 diags	1.033	0.684	0.097	0.292	
Cote Wallace	CLOSE	4 #5 top	2 #3 ties	1.011	2.176	0.324	0.614	
KJ3	$f'_c= 45.0$	2 #5 btm	2 #3 U-bars					448
	OPEN			1.009	0.562	0.084	0.307	
Cote Wallace	CLOSE	4 #5 top	4 #3 ties	1.054	2.269	0.336	0.610	
KJ4	$f'_c= 45.6$		↑3 in joint?					448
	OPEN	2#5 btm	4#3 U-bars	1.075	0.603	0.089	0.305	

Notes; top = top bars

btm = bottom bars

2. DESIGN OF TEST SPECIMENS

2.1 PART A: CONVENTIONALLY REINFORCED KNEE JOINTS

All the test units were designed to the current Concrete Standard (NZS3101:1995) except for the two beam-column joints designed to the 1964 Model Building Bylaw (NZSS1900 Chap. 9.3, 1964). 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.

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 of $2.5d_b$ = 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 development length, (L_{dh} = 150 mm) by 20%, measured from a point eight beam bar diameters in from the inner column face.

The horizontal joint shear ties comprised 6 sets of 4 mm diam. 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 $\frac{6v_{jh}}{f_c^{'}}$ factor, which now appears as a multiplying factor in the formula 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. The equation was

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

where β= ratio of area of compression beam reinforcement to that of the tension beam reinforcement, to be less than or equal to 1,

C_j= ratio of joint shear force in the direction being considered to the sum of the joint shear forces in both horizontal directions, equals 1 in these tests,

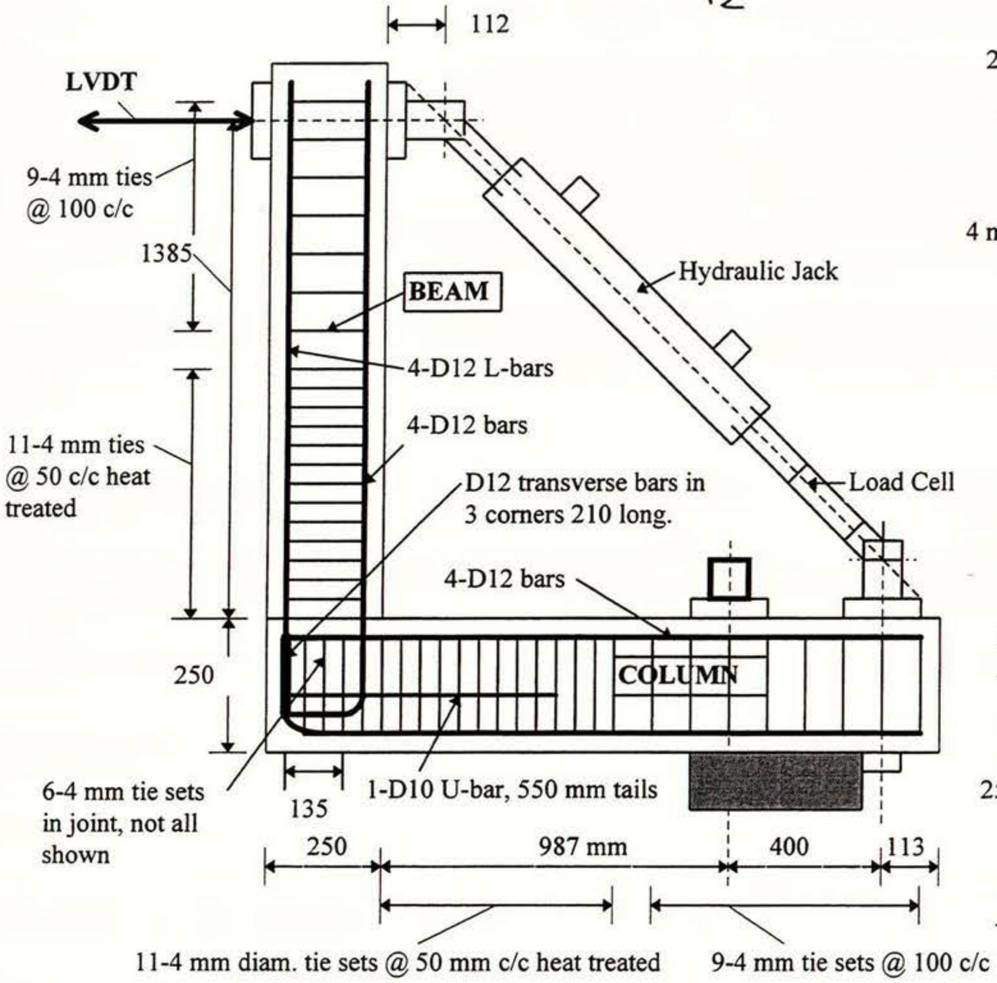
N^*= column axial force, negative if tensile,

A_g= gross column cross-sectional area,

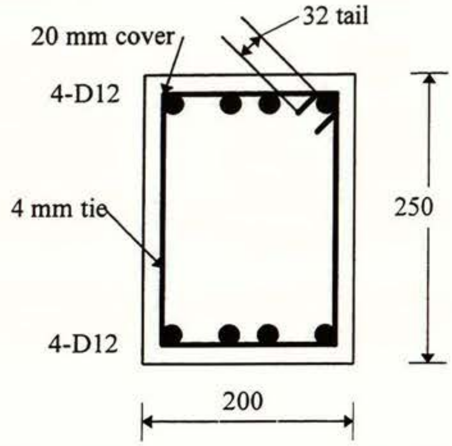
f_{yh}= yield stress of shear tie reinforcing and

A_s= area of tension beam reinforcement at the column face.

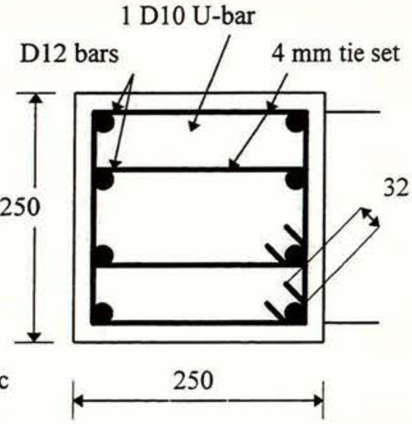
This meant that the ties were theoretically over designed by the 1/0.85 factor (≅ 18%), because $6v_{jh}/f_c^{'} \leq 0.85$ here. However, using the final NZS3101:1995 design formula with the $6v_{jh} / f_c^{'}$ factor included and with f_y = 300 MPa, $f_c^{'}$ = 30 MPa, the actual f_{yh} = 266 MPa and 20 kN axial



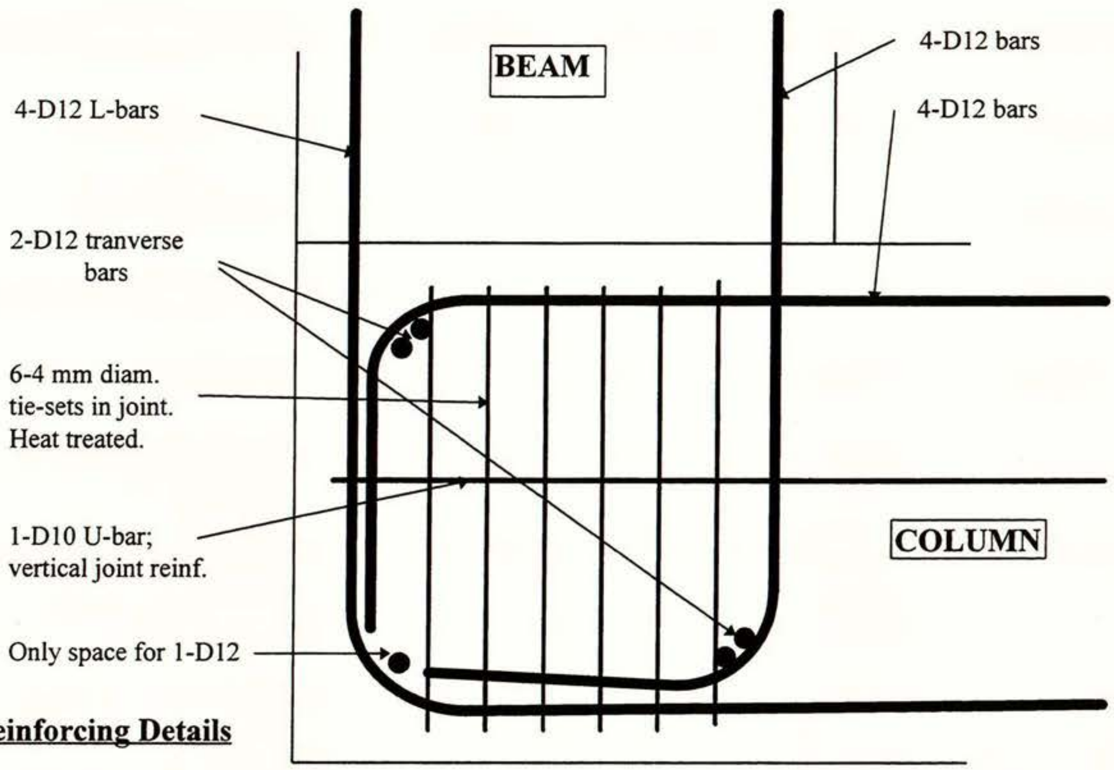
Knee Joint 1 as tested



Beam Section



Column Section



Joint Reinforcing Details

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

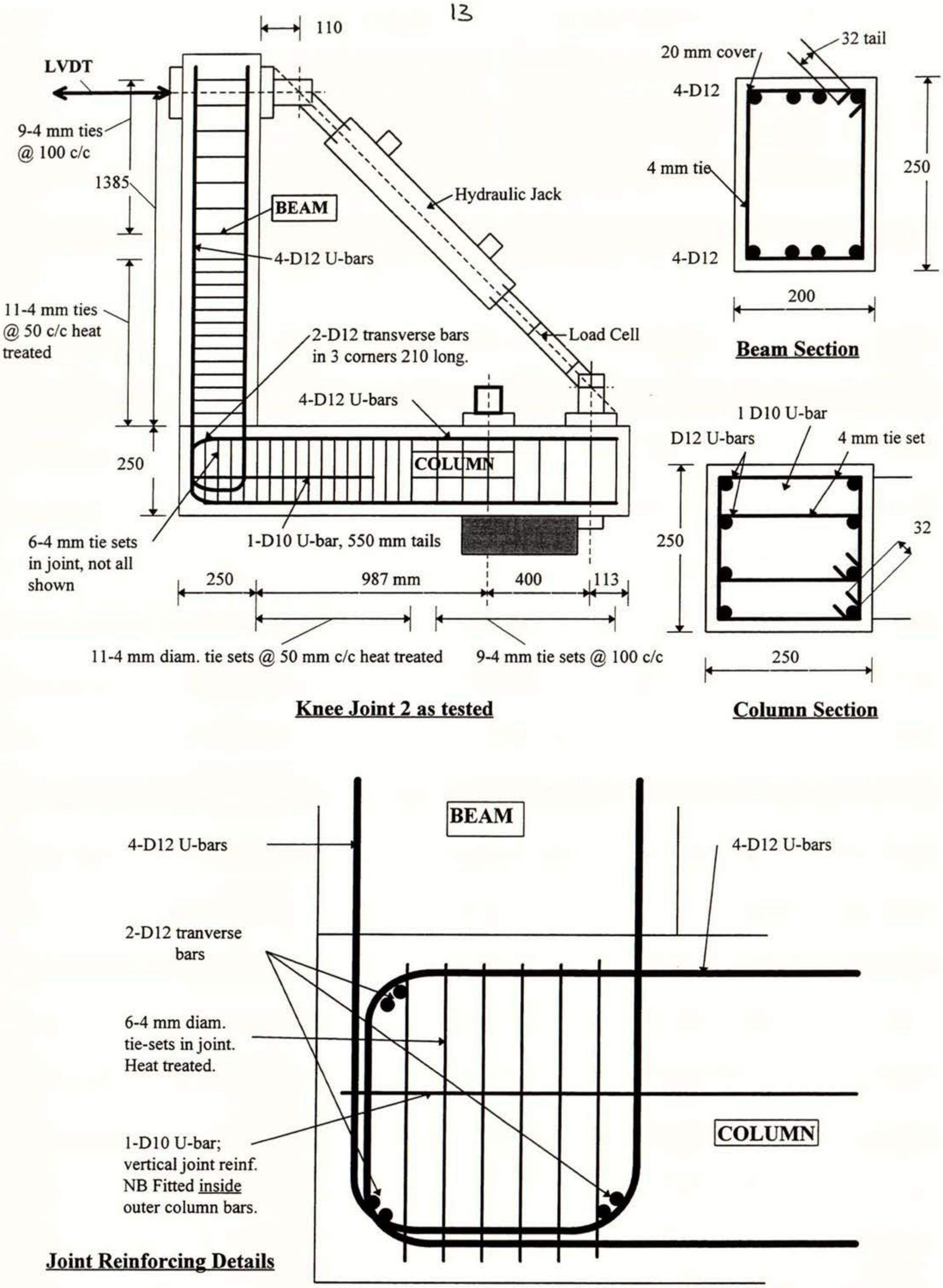


Figure 2: Knee Joint 2 designed to 1995 Standard, with small diameter bars (D12 U-bars).

tension on the column gave an A_{jh} value of 308 mm^2. The actual amount provided was 24 legs of 4 mm diam. ties = 301 mm^2; an under design of only 2%.

The vertical joint shear reinforcement requirement, A_{jv} was 182 mm^2 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 final design equation was $A_{jv} = \alpha_v \frac{h_b}{h_c} A_{jh} \frac{f_{yh}}{f_{yv}}$, where $\alpha_v = \dfrac{0.7}{1 + \dfrac{N^*}{f_c' A_g}}$ (2)

and h_b and h_c are the depths of the beam and column, respectively.

The actual A_{jv} used was a single D10 U-bar (A_{jv} = 157 mm^2) 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 4ϕ 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, see 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 $0.08f_c^{'}$, which was only 40% of the maximum allowed in NZS3101 (1995).

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 $0.12f_c^{'}$. 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, see Figure 6 for the details. The amount of extra cross-sectional area of diagonal bars was initially recommended by Nilsson and Losberg (1971) to be about 50% of the area of main beam tension reinforcing, established from testing of opening moment knee joints.

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 gave an A_{jh} amount of 290 mm^2 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 mm^2 and 199 mm^2, respectively. These smaller amounts of horizontal joint shear reinforcement are due to the opening moment action being critical in the design of 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, β was equal to 2/3 under closing conditions and 1 (the maximum value) under opening moments. Viz. 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 $6v_{jh}/f_c^{'} < 0.85$. For a full explanation and derivation of the NZS3101:1995 design equations the reader is directed to Paulay and Priestley (1992). 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 f_y 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. 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 $0.19f_c^{'}$, for a concrete compressive strength of 30 MPa, this being close to the 1995 Standard's $0.20f_c^{'}$ limit for v_{jh}. However even though the

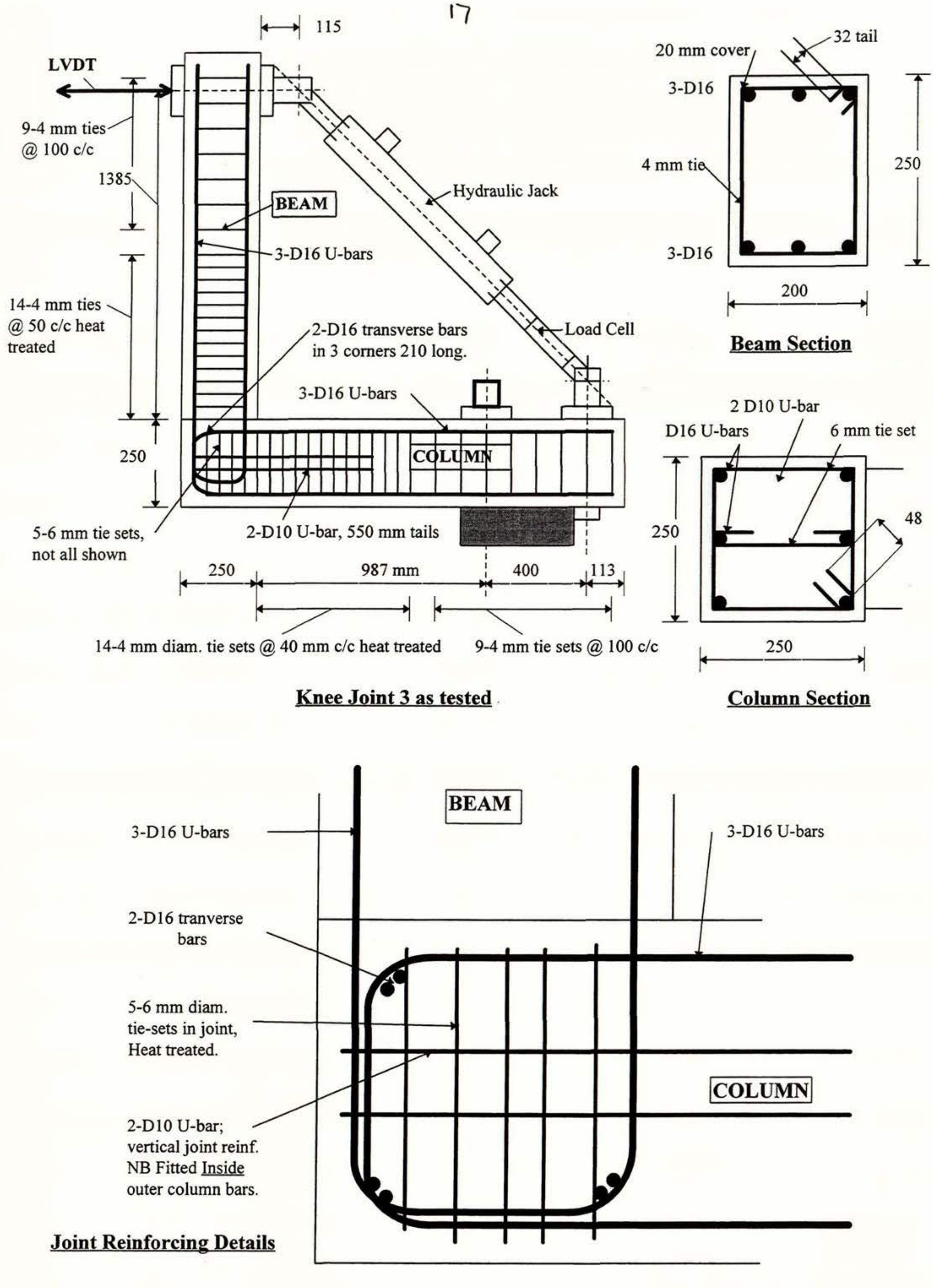


Figure 3: Knee Joint 3 designed to 1995 Standard with D16 U-bars.

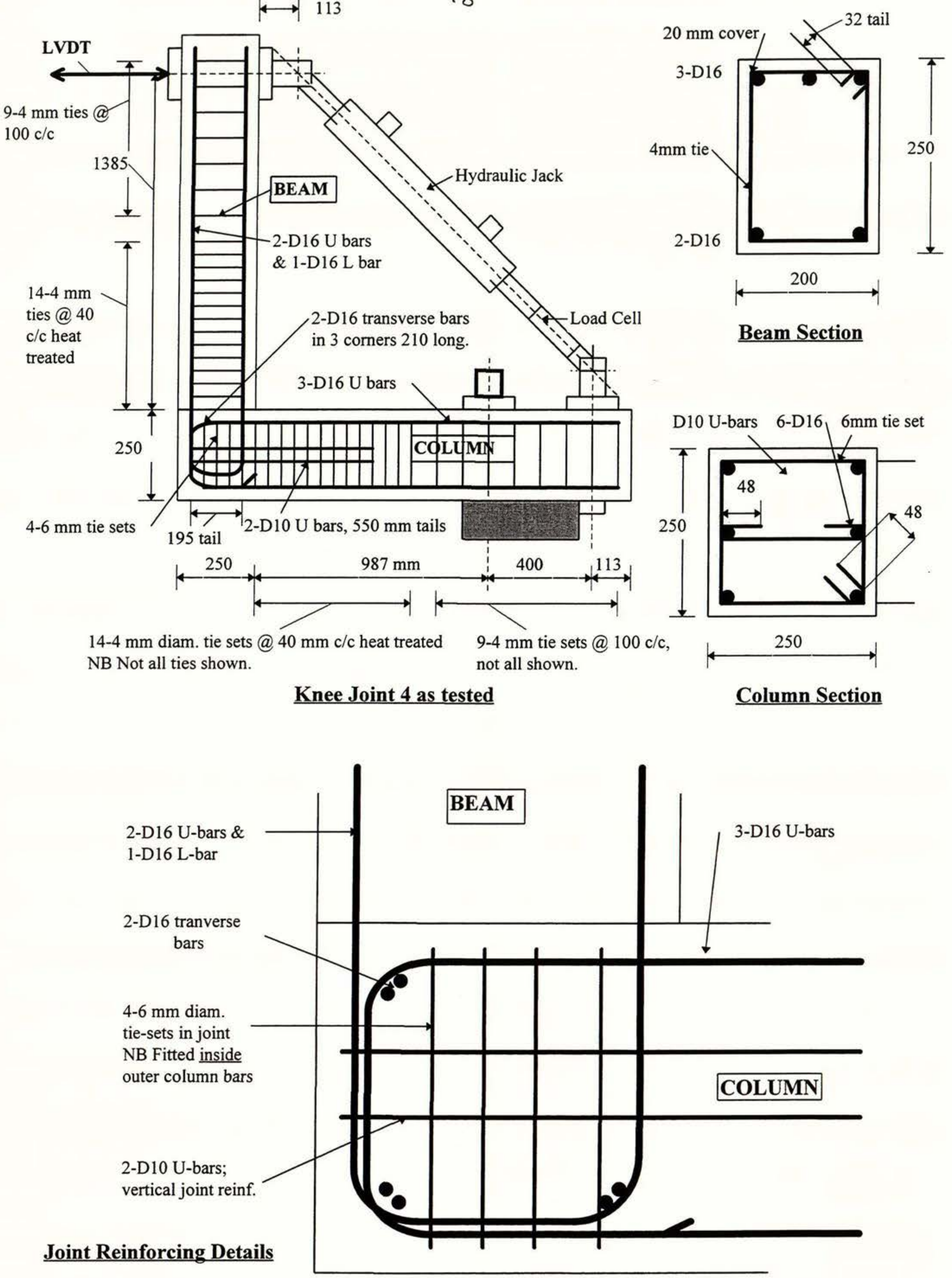


Figure 4: Knee Joint 4 designed to 1995 Standard. U-bars with 1 extra top beam bar.

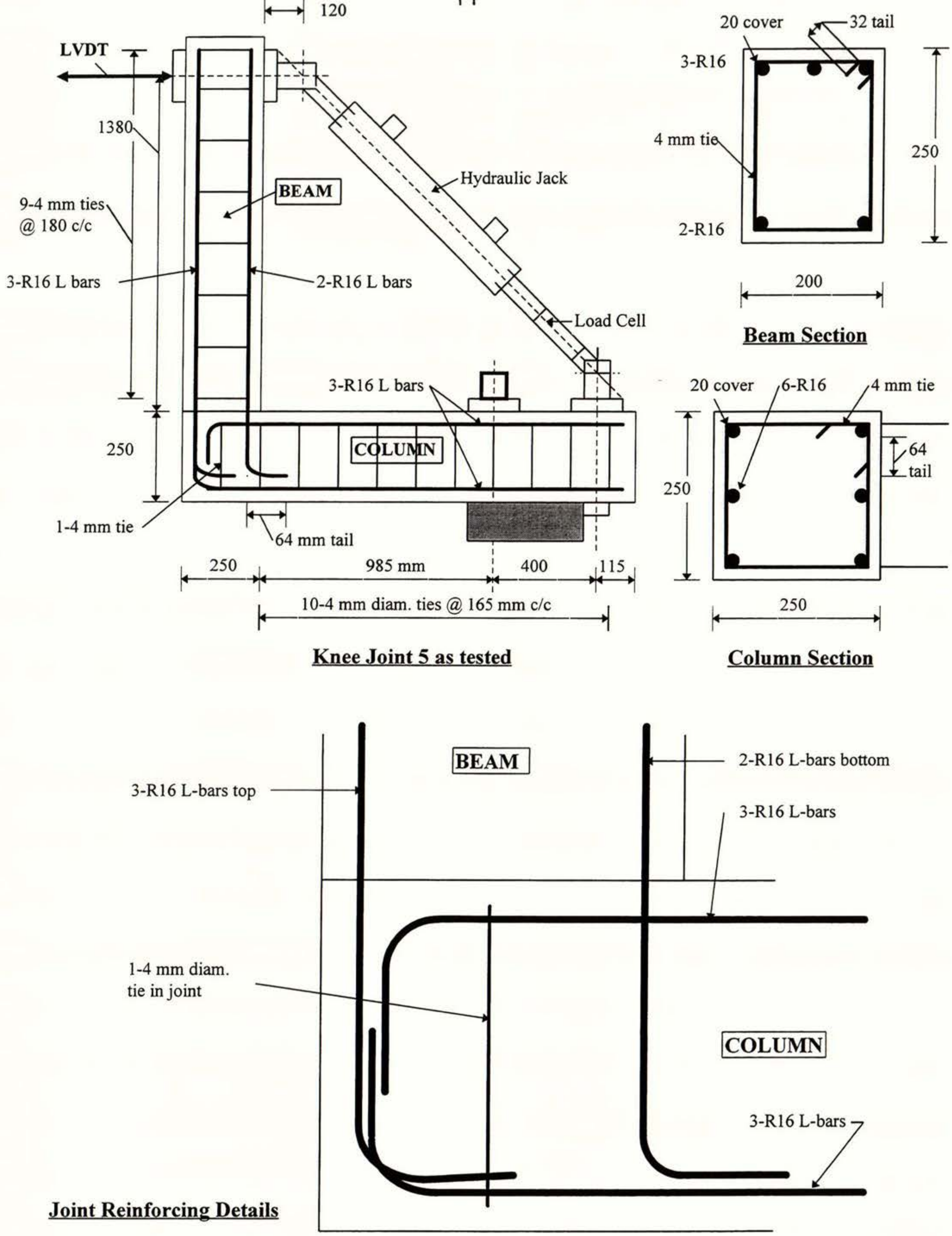
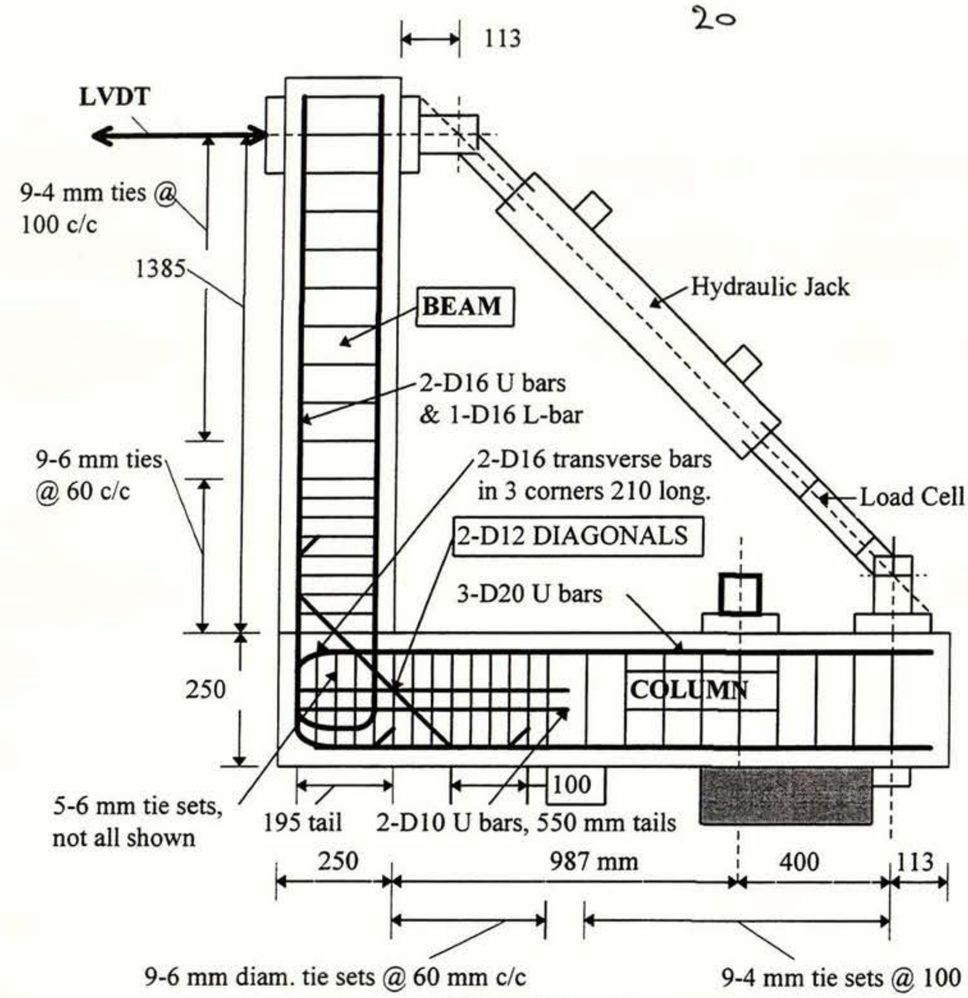
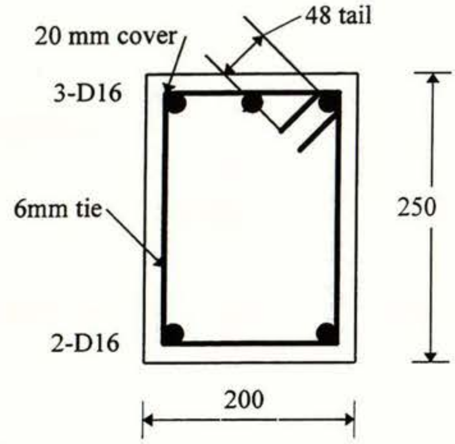


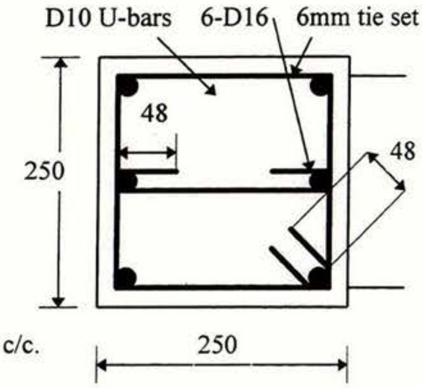
Figure 5: Knee Joint 5 designed to 1960's Code with non-deformed main bars.



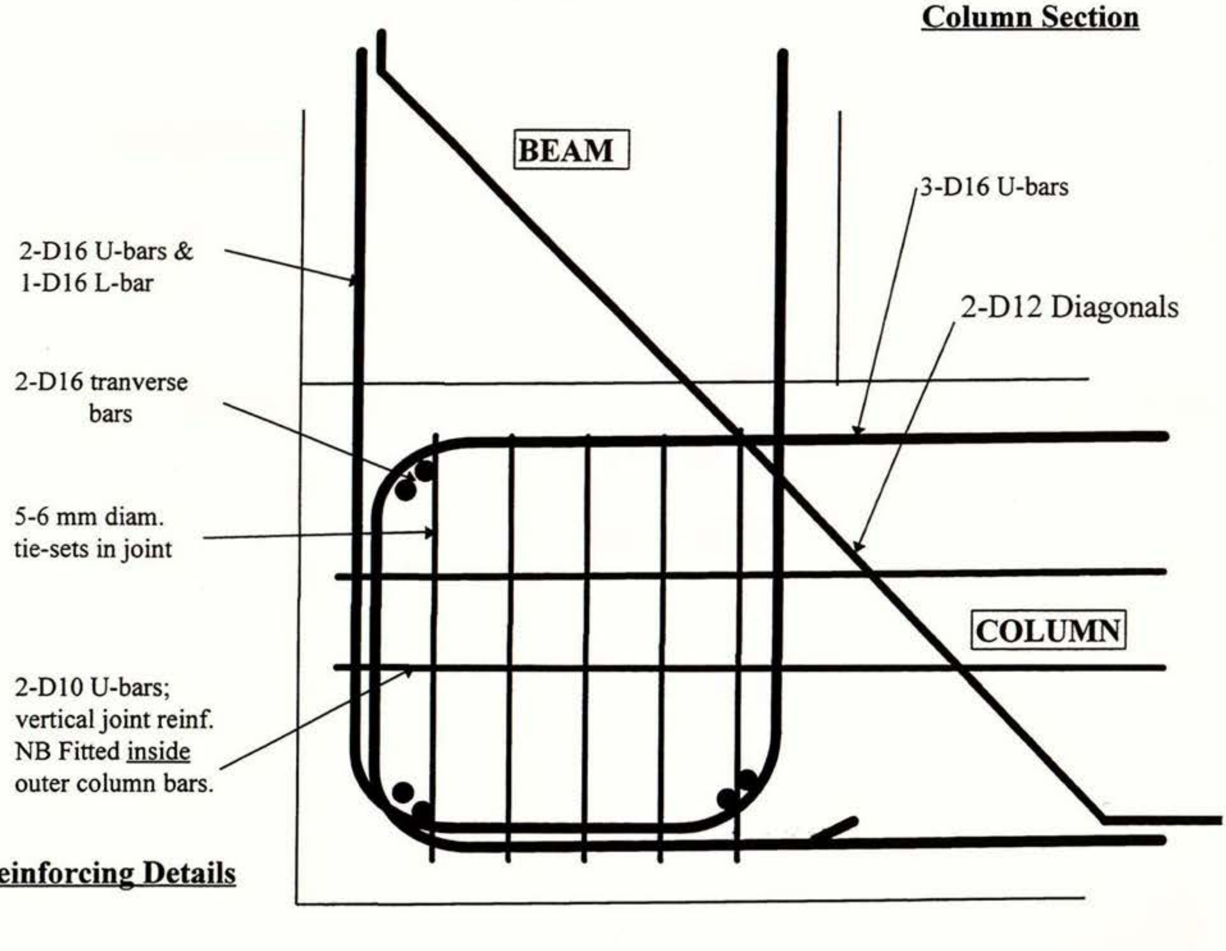
Knee Joint 6 as tested



Beam Section



Column Section



Joint Reinforcing Details

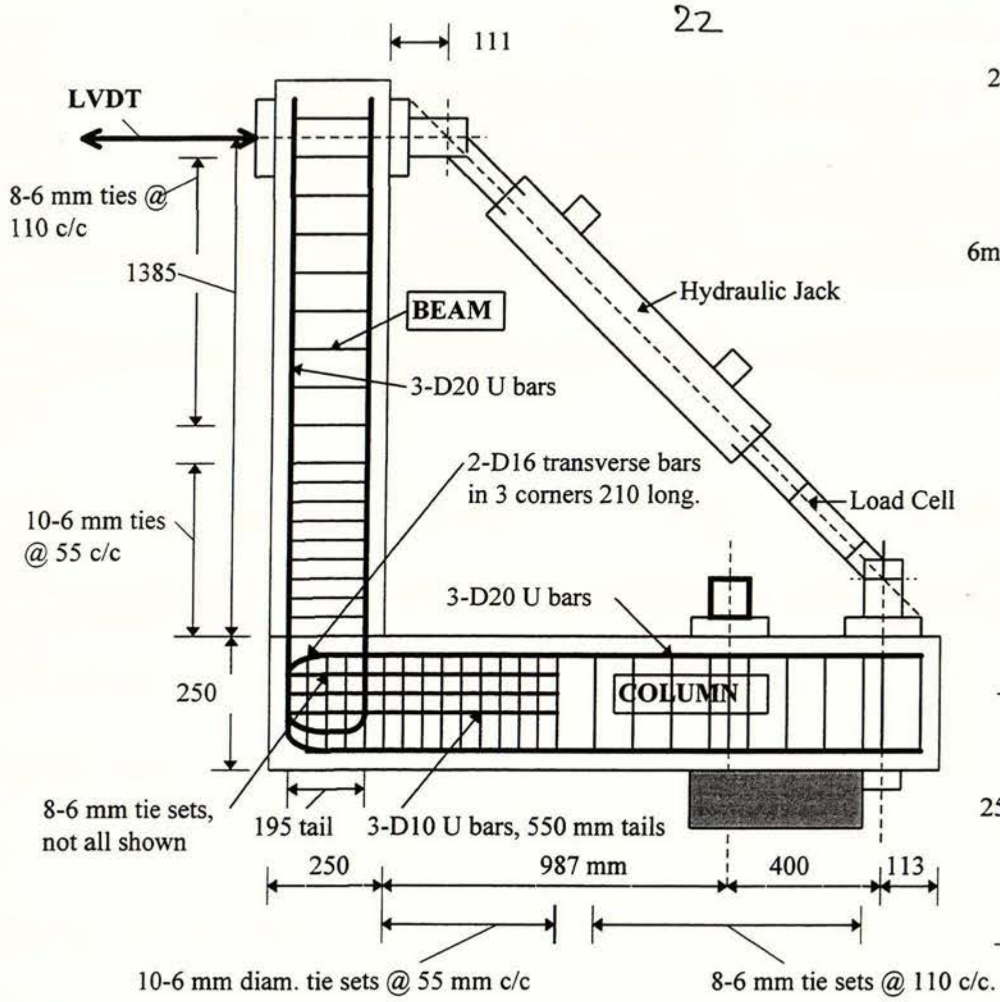
Figure 6: Knee Joint 6 designed to 1995 Standard plus two extra diagonal bars.

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%. In this case the closing condition is critical due to the larger v_{jh} stress producing a larger $6\, v_{jh}/f_c'$ factor. If the actual f_c' of 50 MPa was used the A_{jh} amount dropped to 469 mm^2 (due to the lower $6\, v_{jh}/f_c'$ factor) and the horizontal joint ties were now over designed by 45%.

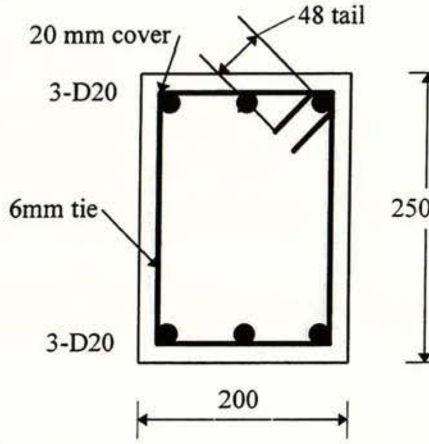
Three D10 U-bars were provided for vertical joint shear reinforcement, A_{jv} = 471 mm^2. For f_c' = 30 MPa and f_{yv} = 337 MPa, A_{jv} required was 440 mm^2 under closing conditions with the column compression force of 41 kN. Therefore the vertical reinforcement was 7% over designed. However the over designed margin increased to 30% if the actual f'_c value was used in the design formula.

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. There were only 3 sets of 6ϕ 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 9 gives the reinforcing details.

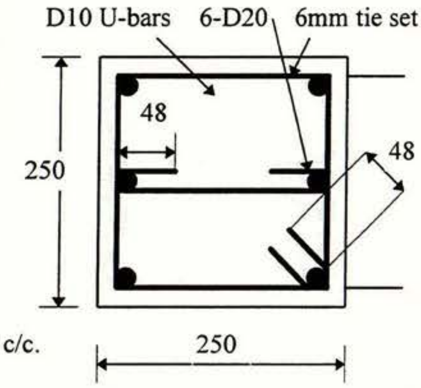
Knee 10 was identical to knee 9 except all the principal beam and column reinforcing was anchored in the joint with standard 90-degree hooks. The tails on these hooks were only $9.4d_b$ = 150 mm long, instead of the specified $12d_b$. The reason for these short tails was the lack of space to accommodate the bottom beam bar and inner column bar tails into the joint. The full $12d_b$ could



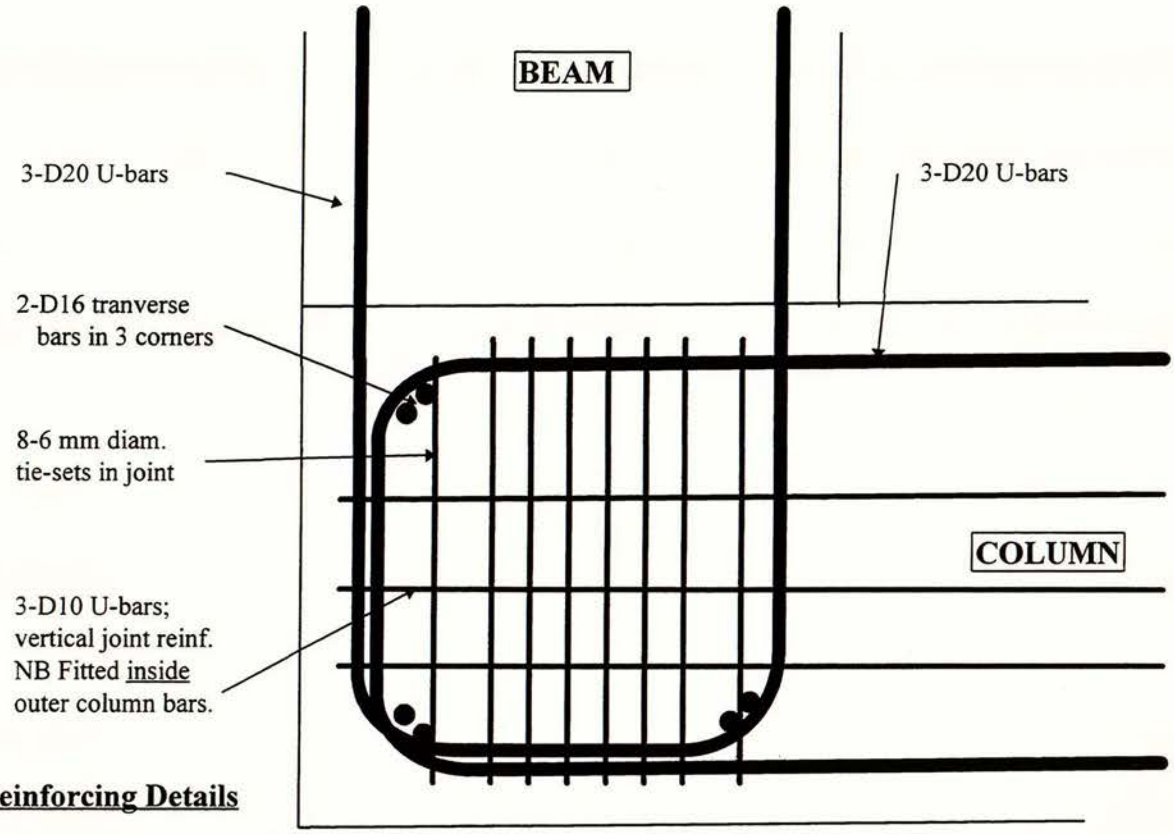
Knee Joint 7 as tested



Beam Section



Column Section



Joint Reinforcing Details

Figure 7: Knee Joint 7 designed to 1995 Standard with D20 U-bars.

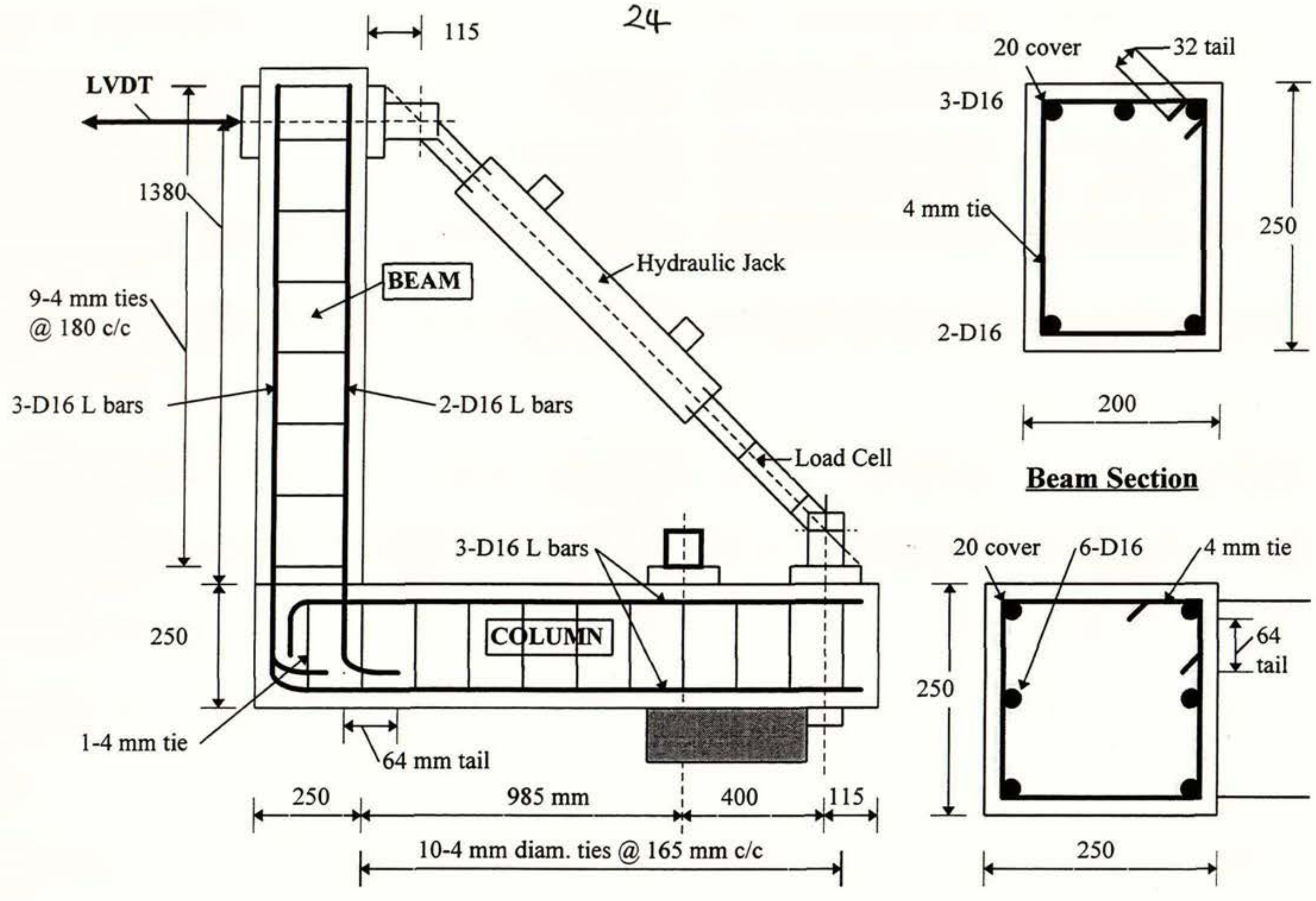
only have been accomplished if column and beam stubs had been added to the joint. In an effort to compensate for this inadequate anchorage, two D16 transverse bars were added to each of the three joint corners with 90-degree bends, see Figure 10.

An extra knee joint, No. 14 was tested while this report was being written and its results are included in Appendix A. 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 A1.

2.2 1960's KNEE JOINTS

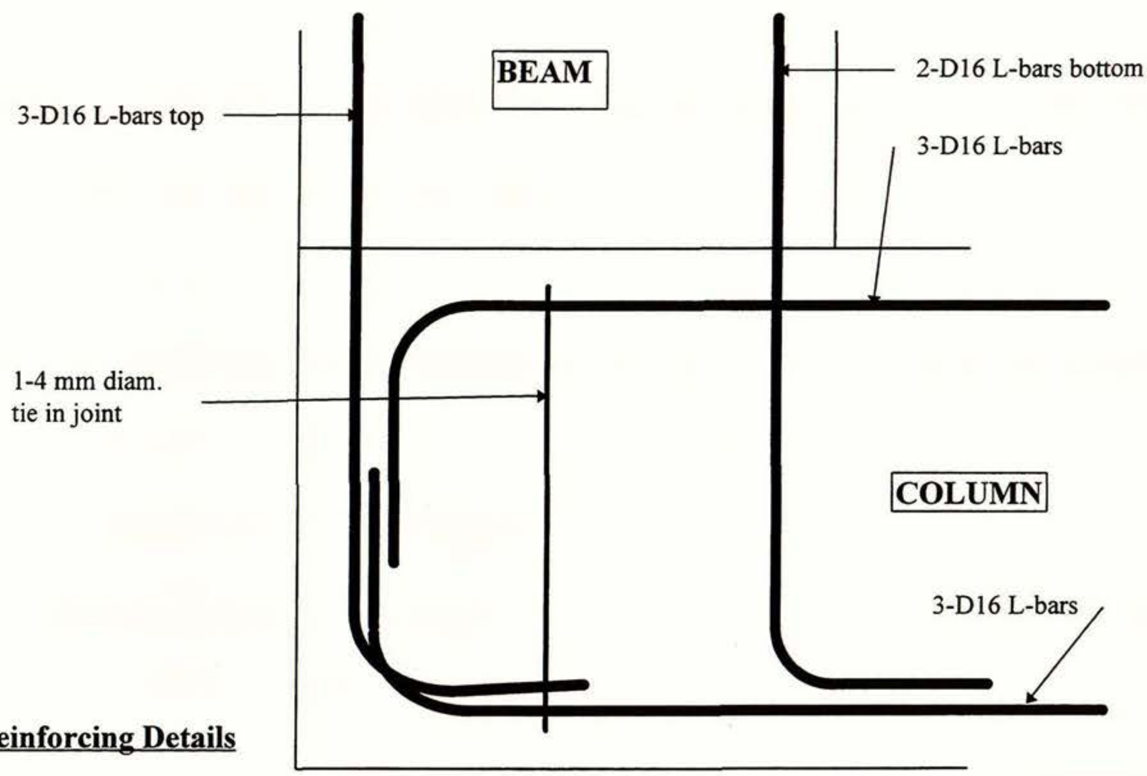
Knee joints 5 and 8 were designed to the Concrete Code current in the mid 1960's (NZSS1900:Chap. 9.3: 1964). This Code had very few clauses specifically related to earthquake loading and detailing considerations. Beam-column joints could be detailed with no transverse joint shear reinforcement in either direction, and poor anchorage details, by today's standards, were common. Beam bars were often bent <u>out</u> 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 just 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



Knee Joint 8 as tested

Column Section



Joint Reinforcing Details

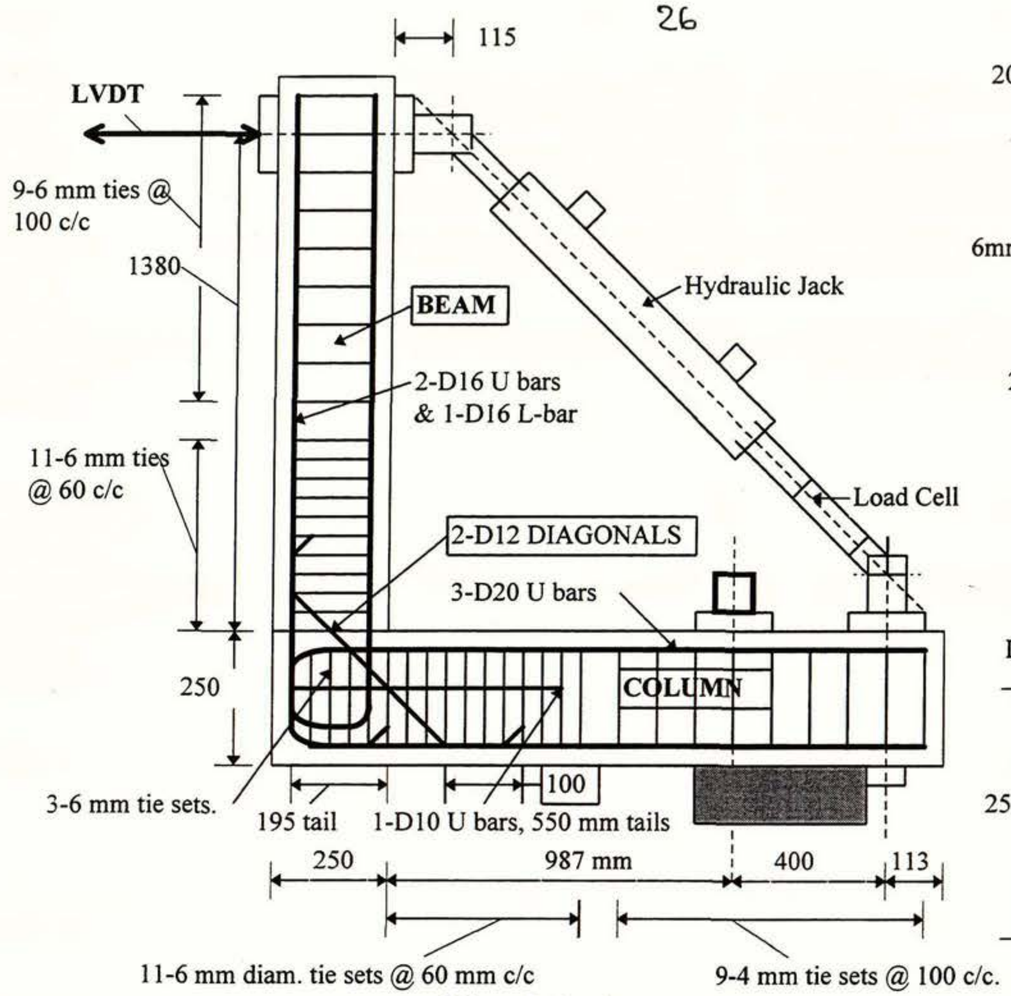
Figure 8: Knee Joint 8 designed to 1960's Code with deformed main bars.

but the two bottom beam bars were bent down into the column near the column's outer face. Figures 5 and 8 show the reinforcement details of knee joints 5 and 8, respectively.

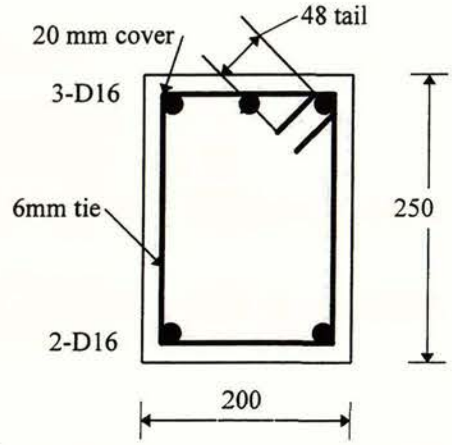
The 1964 Code allowed two types of anchorage for beam and column stirrups. In these tests the better anchored 135-degree bend with a $8d_b$ (32 mm) tail was employed in the beams but the poorer 90-degree bend with $16d_b$ tail was used for the column ties. d_b in this case is the diameter of the ties; 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 ($0.03f'_c$ = 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 beams it was ≤ 3/4 beam depth = 187 mm.

2.3 TEST CONSTRUCTION AND SETUP

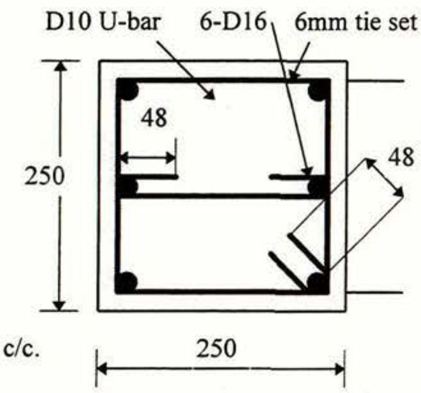
The knee joint units were cast on their sides in one pour using commercial ready-mix concrete with a specified 28 day compressive stress of 25 MPa (30 MPa in joint 7) and a maximum aggregate size of 10 mm. Table 2. shows the f_c' values obtained immediately after testing and the reinforcing tensile yield stresses. The units were covered with sacking and kept moist for a week after casting, as were the concrete cylinders (3 per pour).



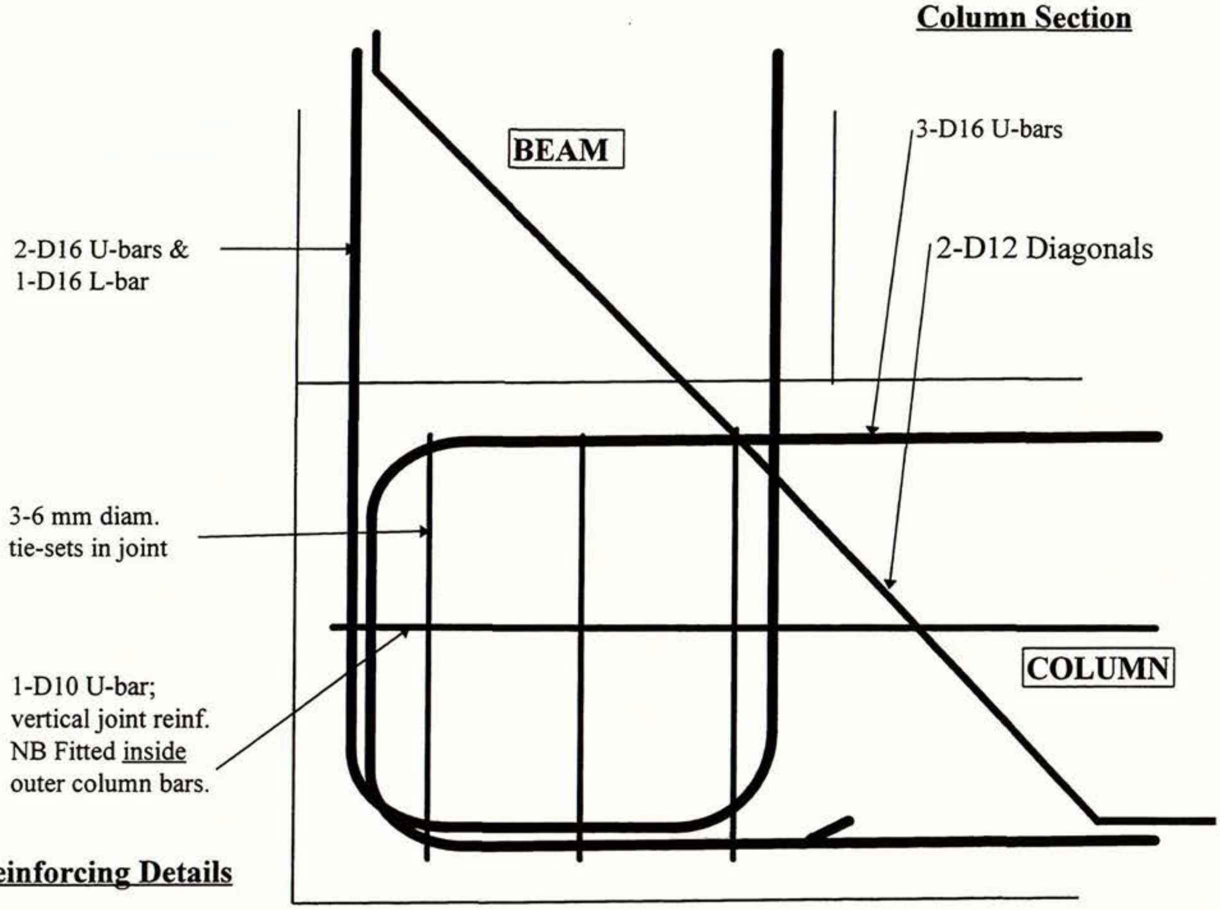
Knee Joint 9 as tested



Beam Section

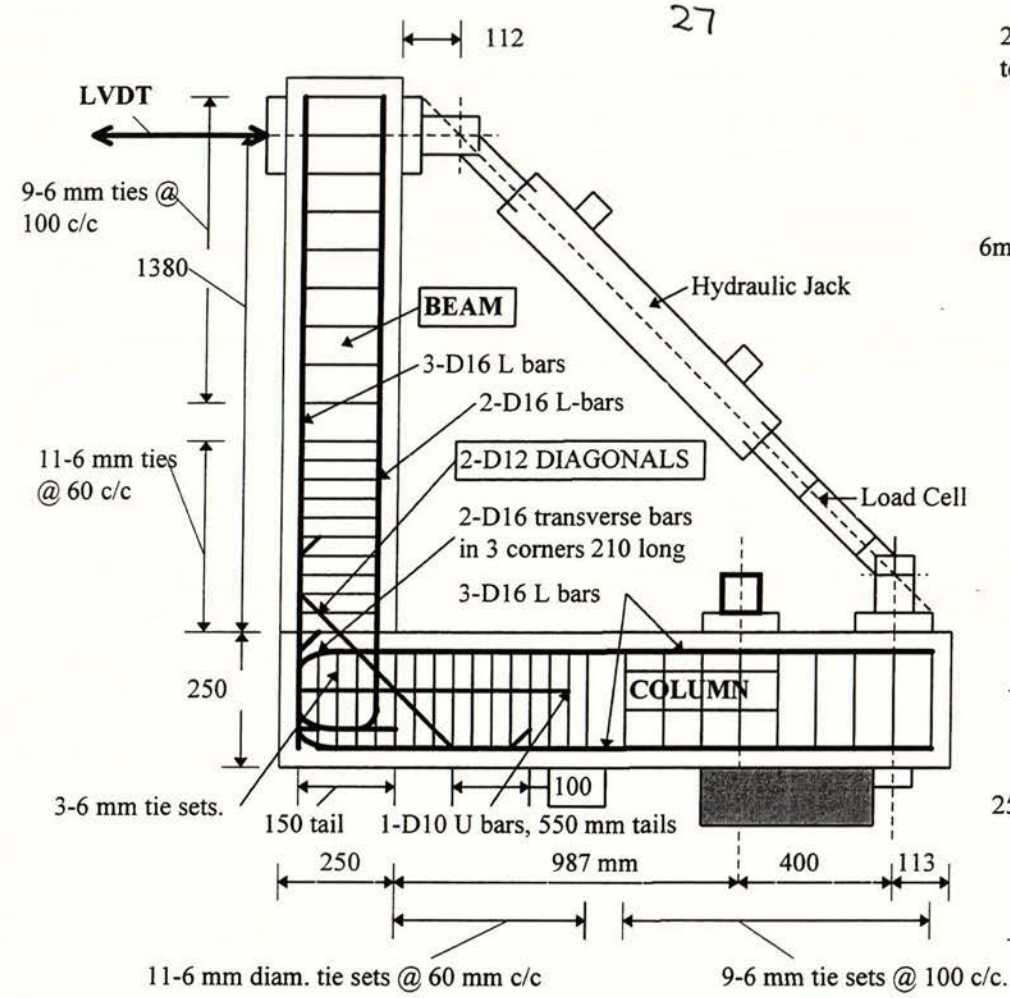


Column Section

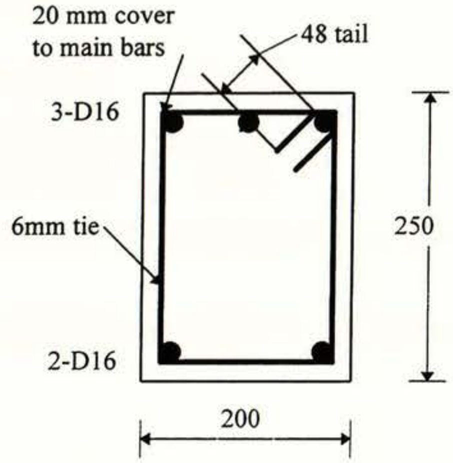


Joint Reinforcing Details

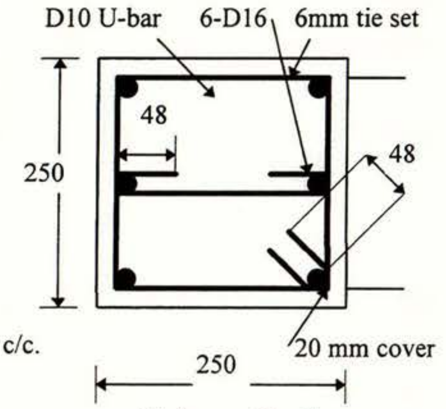
Figure 9: Knee Joint 9 designed to 1995 Standard with unequal top and bottom beam bars plus two extra diagonal bars.



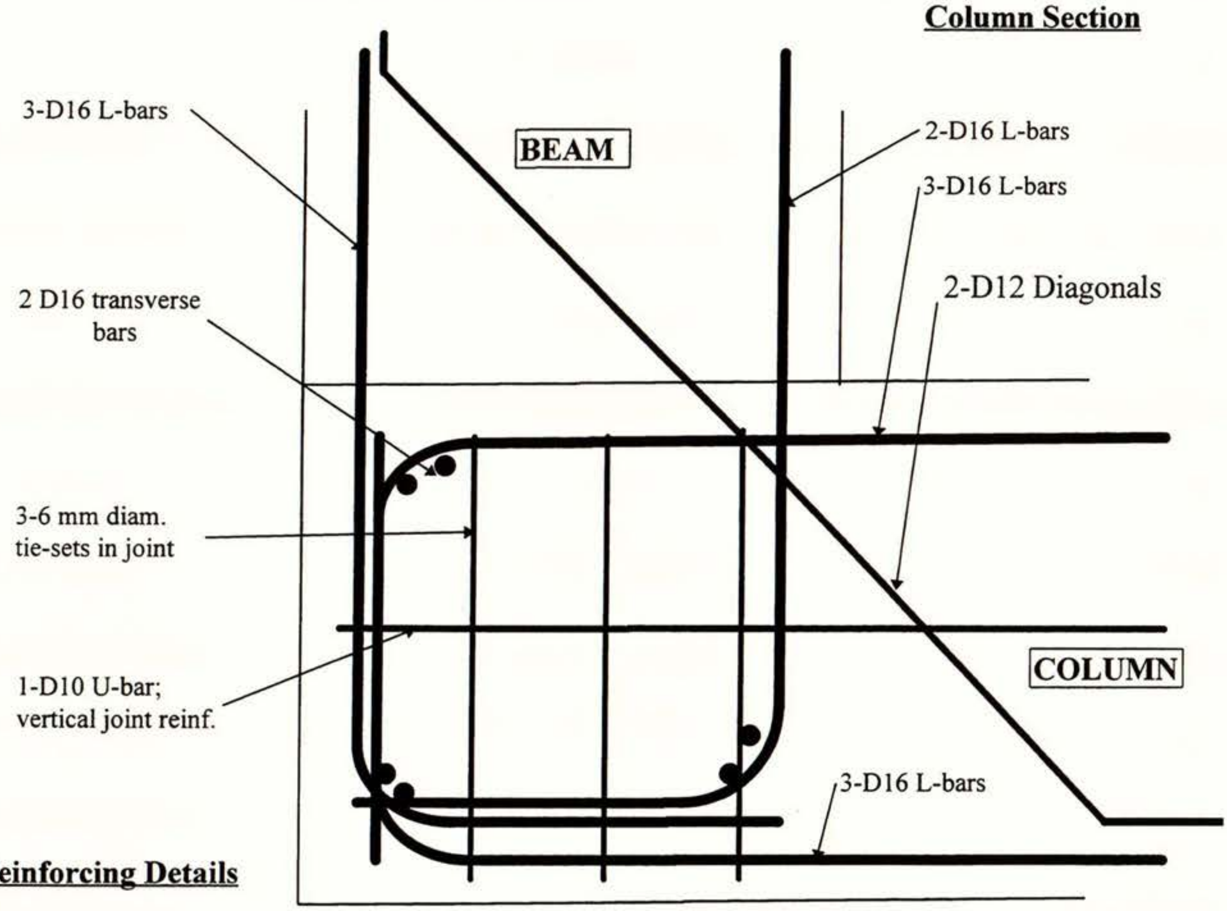
Knee Joint 10 as tested



Beam Section



Column Section



Joint Reinforcing Details

Figure 10: Knee Joint 10 designed to 1995 Standard; main bars with standard hooks, plus two extra diagonal bars.

TABLE 2: Concrete Compressive Stresses (f_c') and Yield Stresses of Reinforcing for Knee Joints 1 to 10.

Knee Joint	f_c' (MPa) Conc.	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

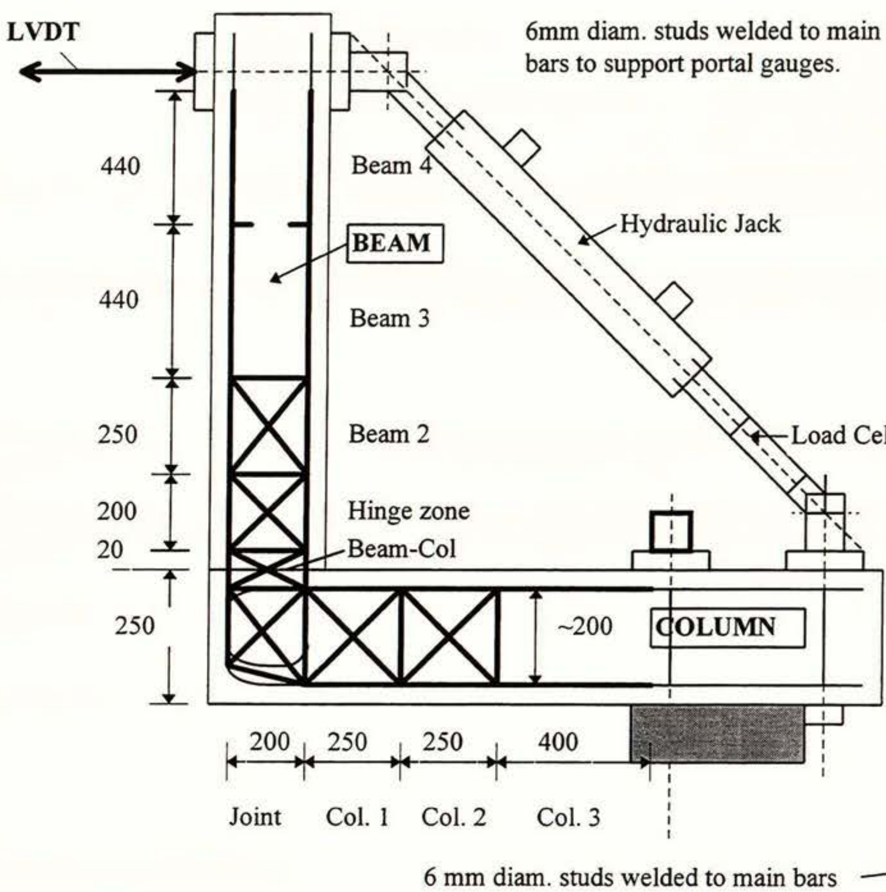
NOTE: H.T. = Heat Treated

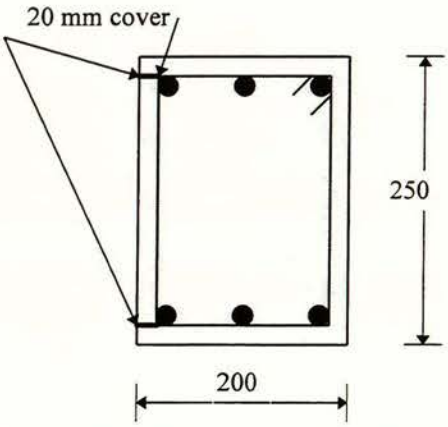
The knee joints were tested 90 degrees out of prototype position with the beam vertical and the column end tied down to the strong floor with tensioned high strength bolts. The 50 kN hydraulic jack was diagonally positioned between the beam and column ends (see Figure 1), thus applying a lateral and axial force to both the beam and column. The members 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. 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.

2.4 LOADING SEQUENCE

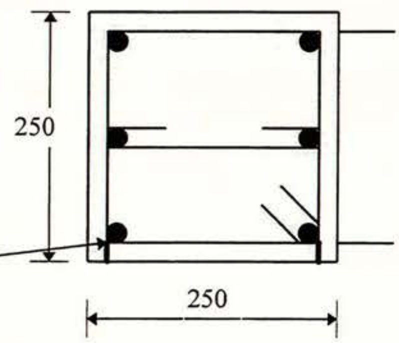
The loading sequence previously used for many structural component, sub-assembly tests in New Zealand was again used in these tests. This entails two 'elastic' cycles, up to a force needed to apply about ±0.75 of the nominal moment, M_n at the critical column face. The nominal moments were calculated 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. Figure 12 is a graphical representation of the loading sequence.

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.



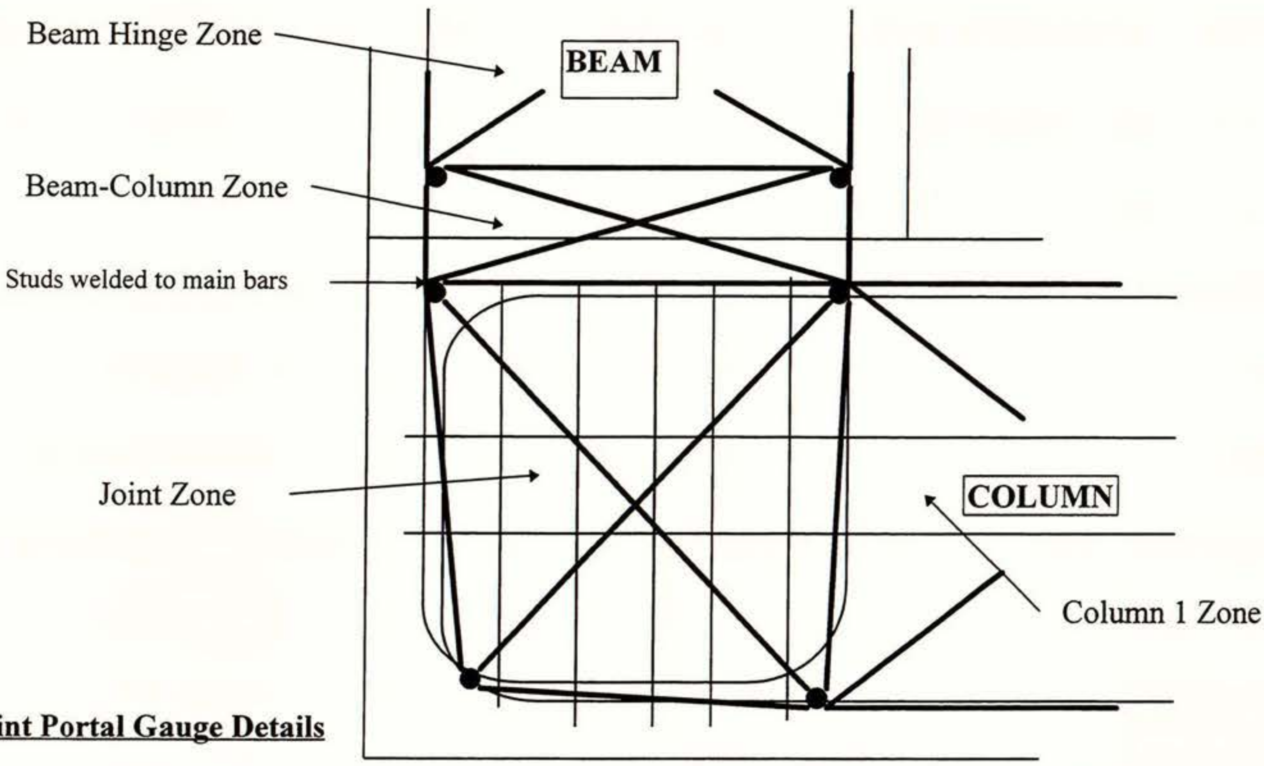


Beam Section



Knee Joint Portal Gauge Positions

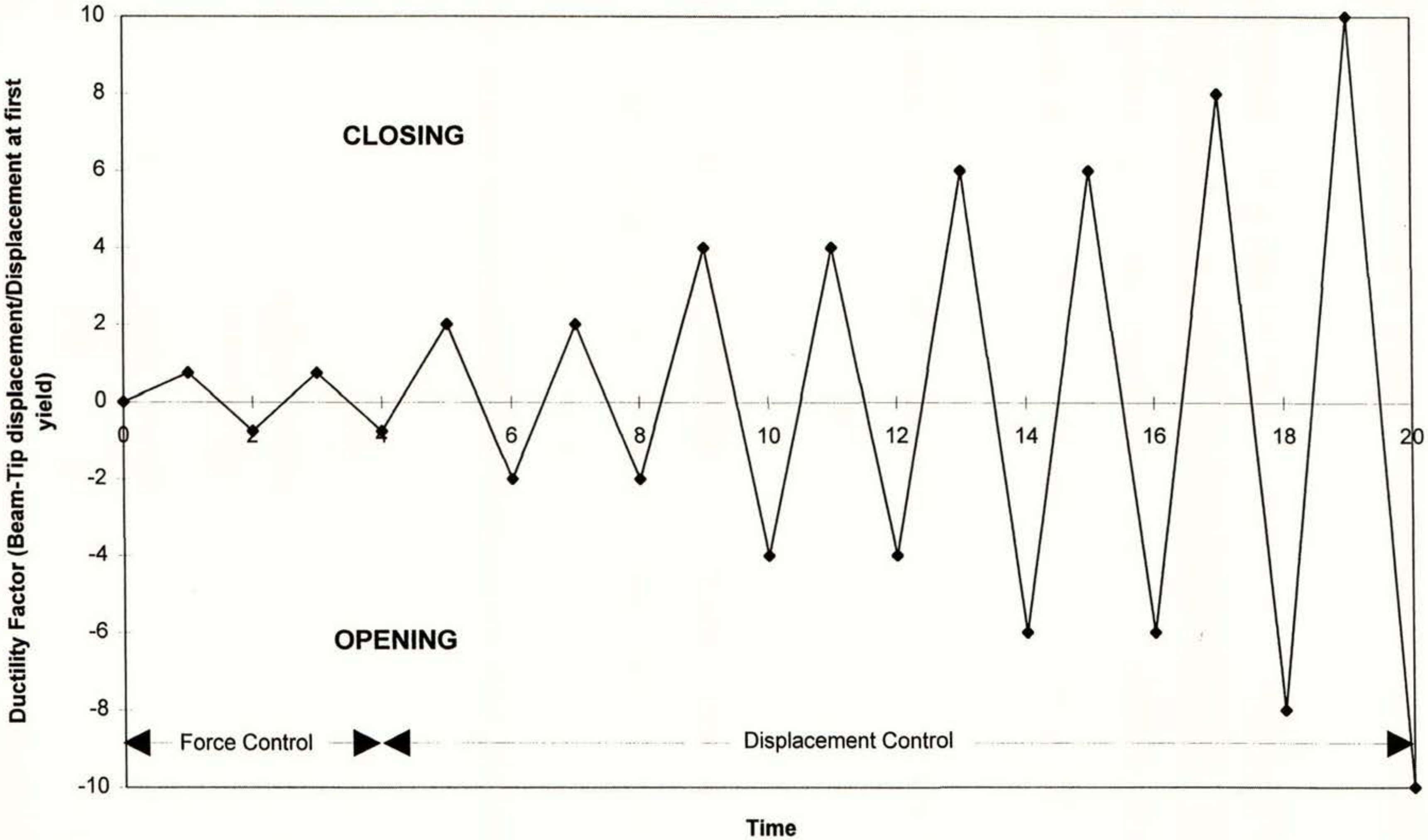
Column Section



Joint Portal Gauge Details

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

FIGURE 12: Loading Sequence in knee joint tests.



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 reaching a maximum of $0.95M_n$ under opening moments. However as the displacement cycles continued, the joint progressively failed in 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 13 and show a gradual reduction in load sustained as the displacement ductility increased.

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 14 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 and the measured deflection plots was a measure of the portal frame inaccuracy and the small flexure and shear deflections not measured near the ends of the beam and column. The shear deflections were larger in the closing direction and became greater than the flexural deformations at ductility 6. Like Mazzoni's (1991) 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. Figure 15 is a photograph of the joint at ductility -10 (opening), in which the destruction of the joint is obvious. The bent and buckled transverse joint reinforcement should be noted.

FIGURE 13: KNEE 1 Applied Force - Beam-Tip Deflection

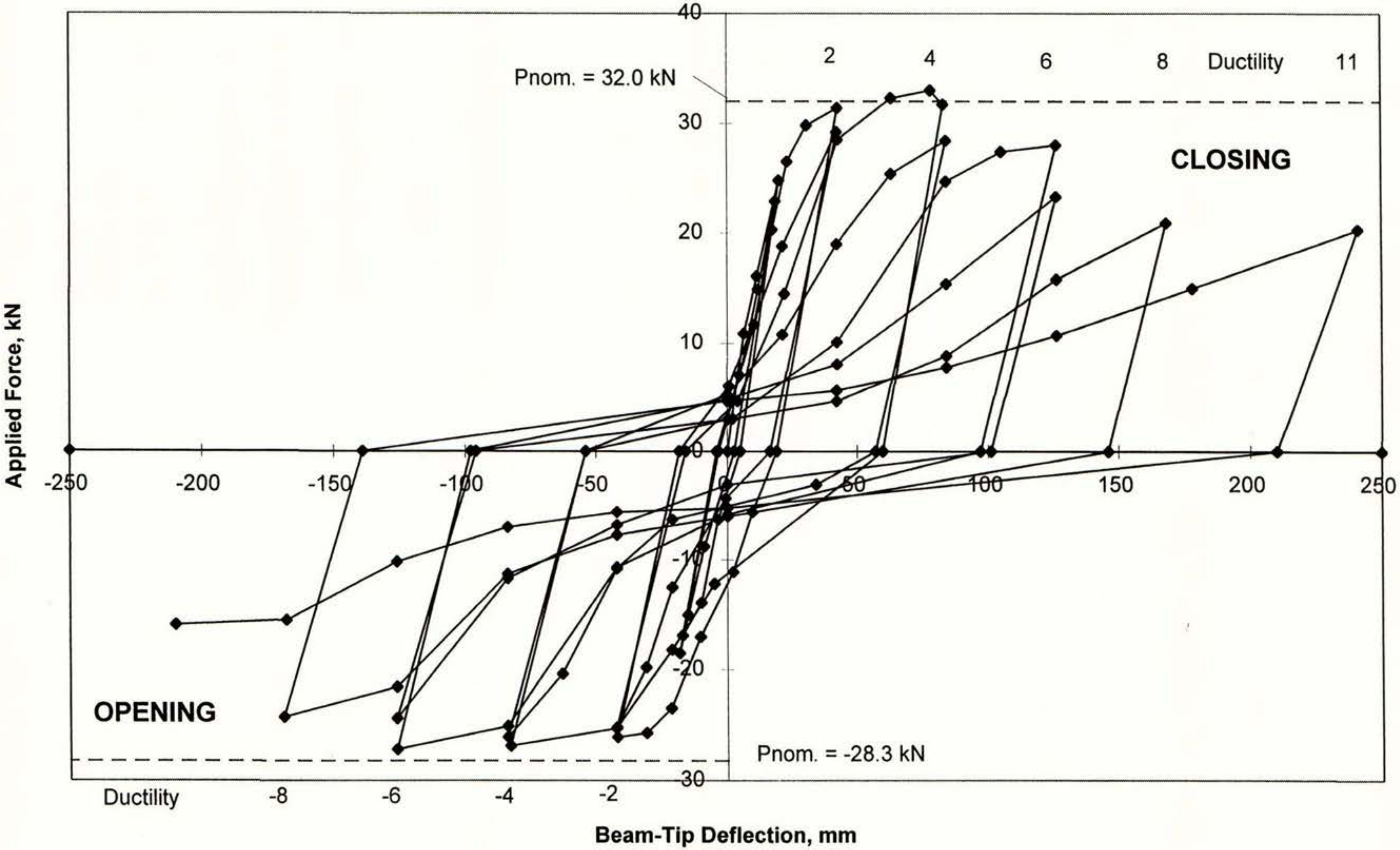
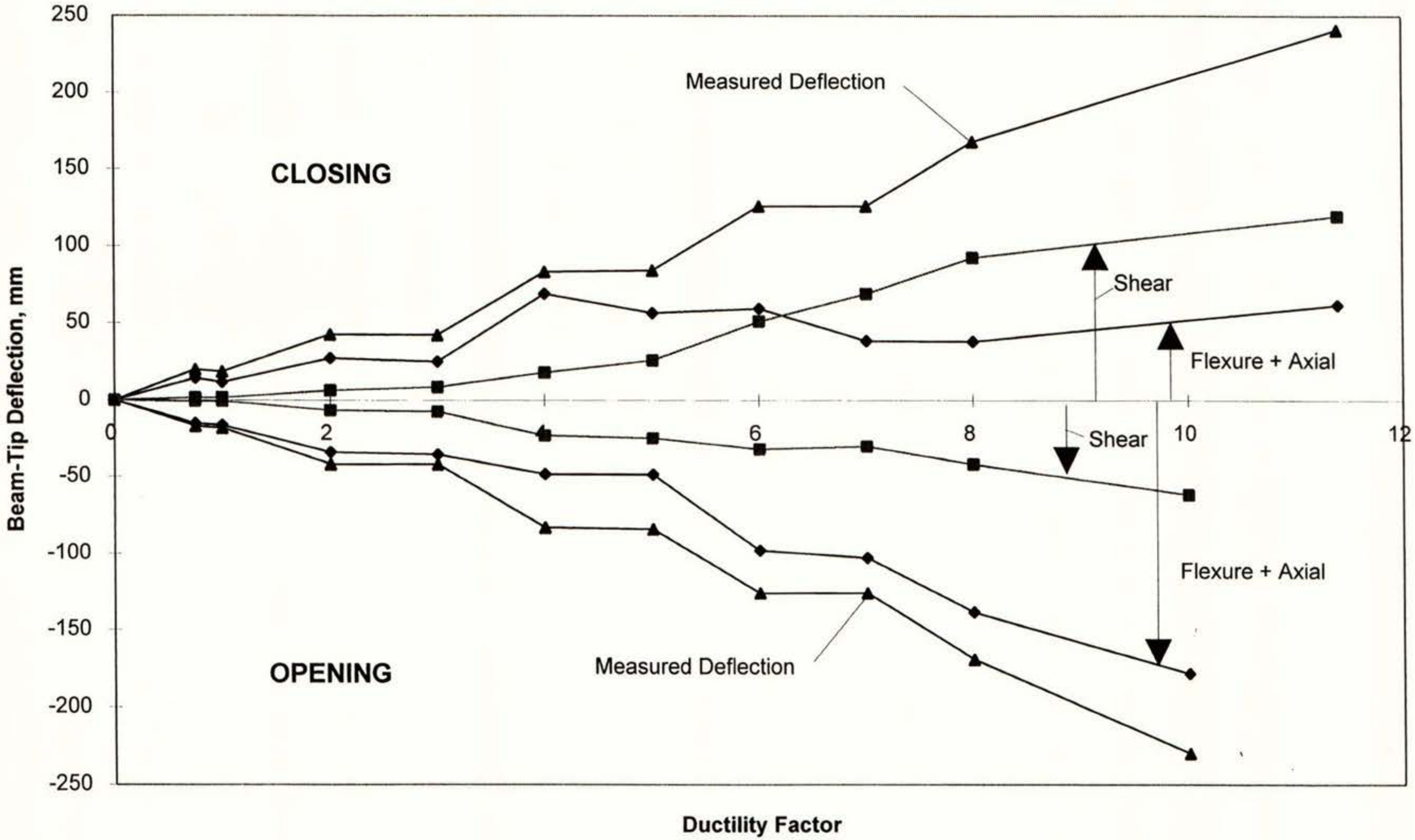


FIGURE 14: KNEE 1 Flexure+Axial, Shear deformations and Measured Beam-Tip Deflection at cycle peaks.



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3.2 Knee Joint 2: This joint with U-bars performed much better than the first unit. A beam plastic hinge continued to form throughout the test and the strength degradation was not as large, see Figure 16 for 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, see Figure 17, taken at the second closing cycle at ductility 6.

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 18 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 appear much better than "standard hooks" in small sections under cyclic loading. The moment 'efficiencies' (maximum test moment/nominal moment ratio, 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. Refer Ingham, Priestley and Seible (1994) for a fuller explanation.

The maximum horizontal joint shear stresses reached in knees 1 and 2 were nearly identical, the average being $0.094f_c^{'}$ (or $0.49\sqrt{f_c^{'}}$ MPa) closing and $0.077f_c^{'}$ ($0.40\sqrt{f_c^{'}}$ 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.

Chart1

FIGURE 16: KNEE 2 Applied Force - Beam-Tip Deflection

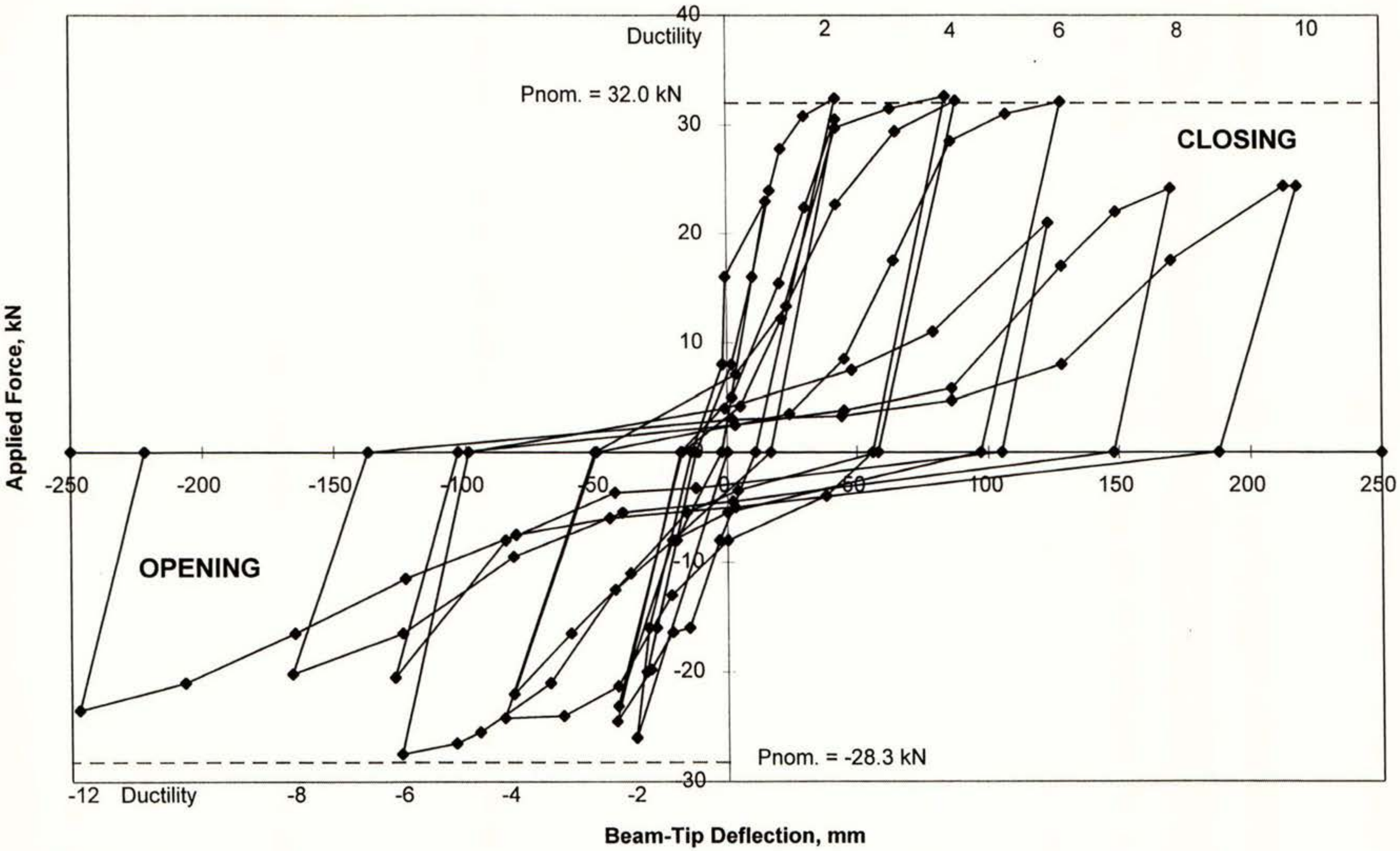

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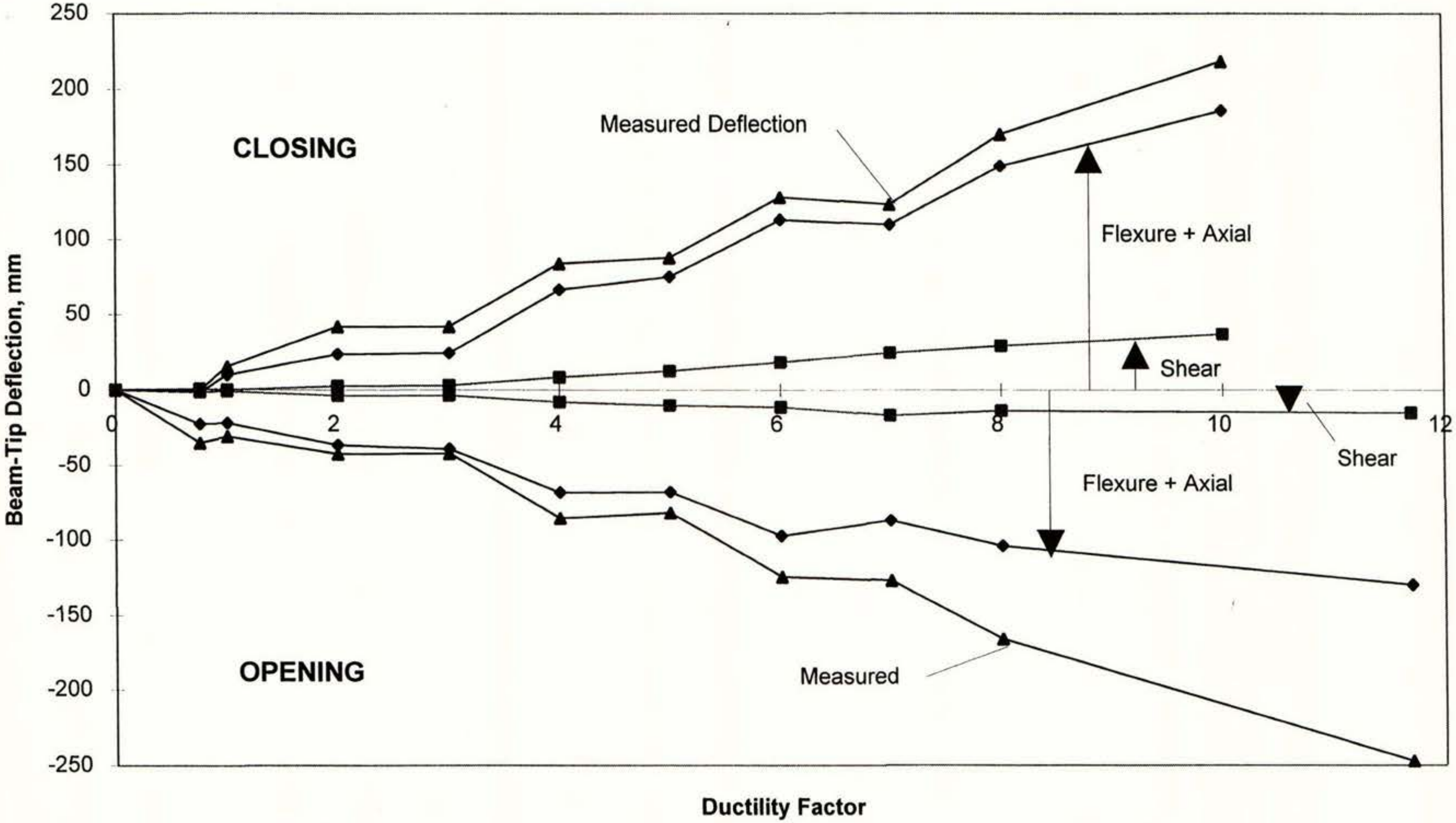
FIGURE 18: KNEE 2 Flexure +Axial, Shear deformations and Measured Beam-Tip Deflections at cycle peaks.



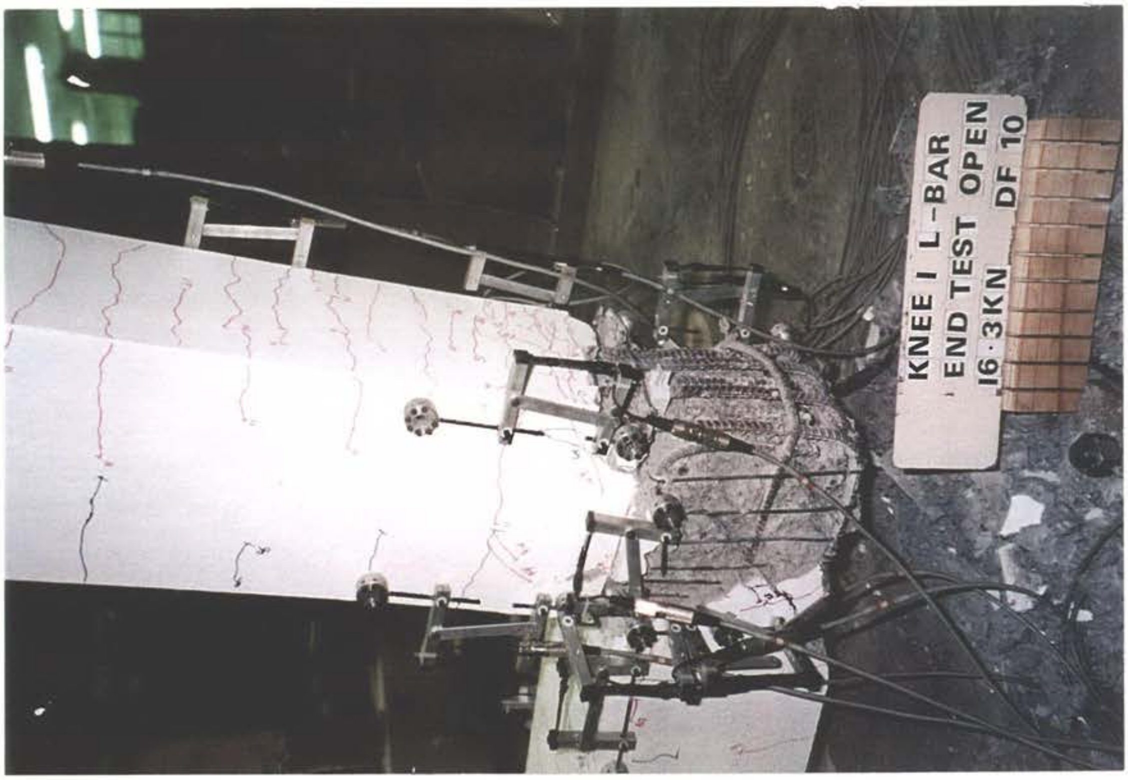


FIGURE 15: Knee 1 at end of test, opening ductility -10

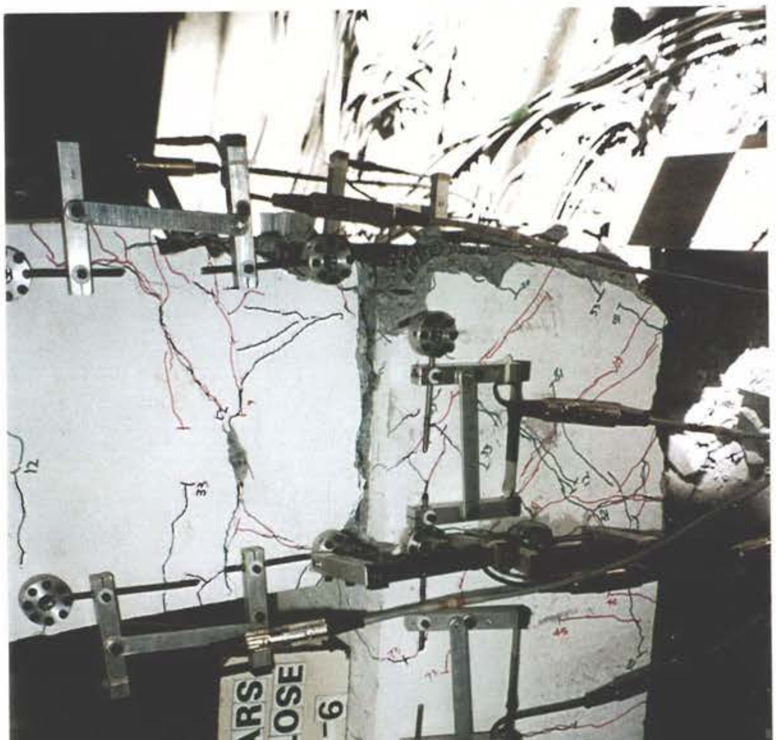


FIGURE 17: Knee 2 at closing ductility 6

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 right 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 19. 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 $0.75M_n$ peaks. 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 plastic hinge. This can be seen in Figure 20, 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

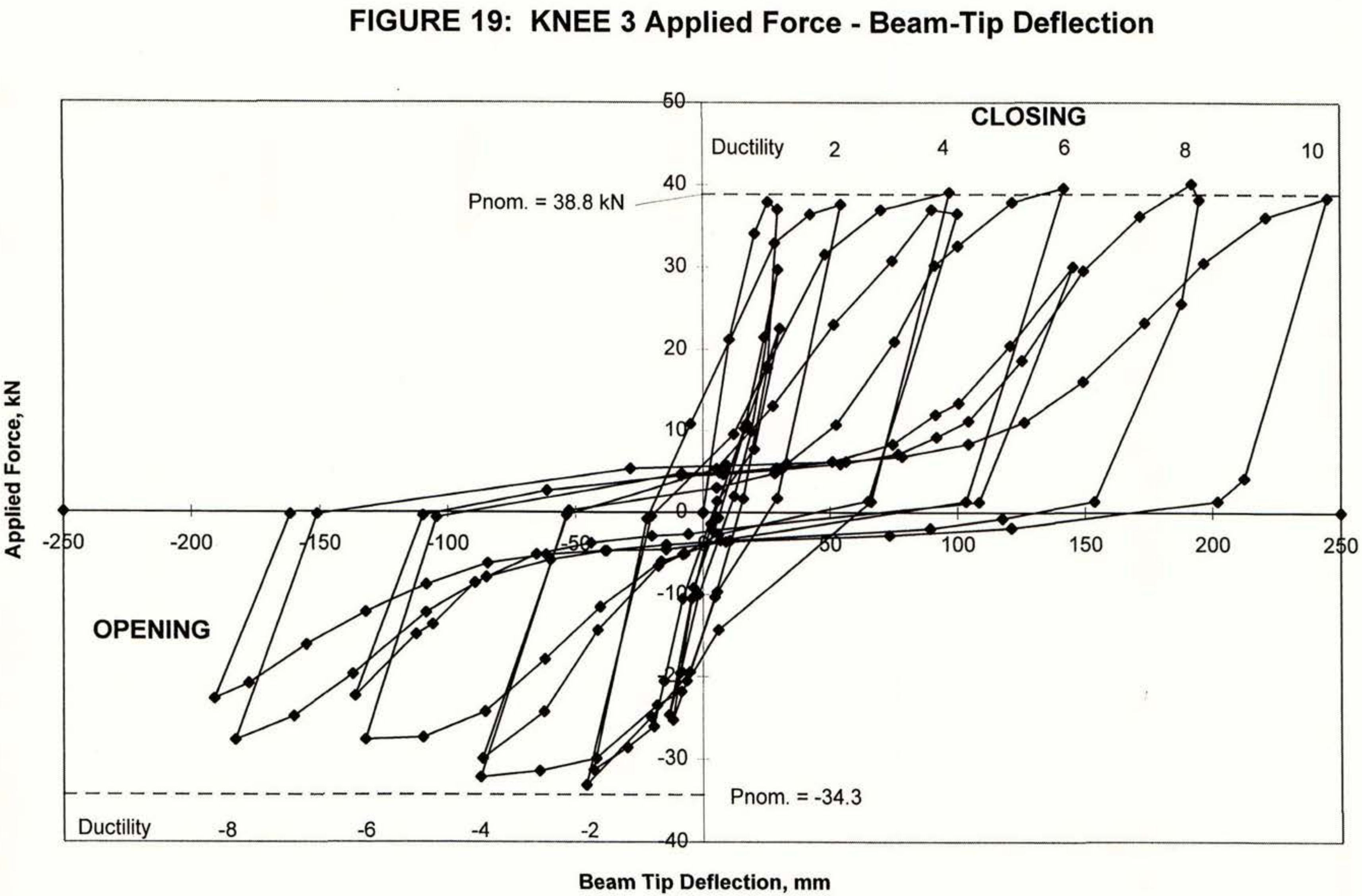


FIGURE 19: KNEE 3 Applied Force - Beam-Tip Deflection
CLOSING
Ductility 2 4 6 8 10
Pnom. = 38.8 kN
OPENING
Pnom. = -34.3
Ductility -8 -6 -4 -2
Applied Force, kN
Beam Tip Deflection, mm
50
40
30
20
10
0
-10
-20
-30
-40
-250
-200
-150
-100
-50
50
100
150
200
250

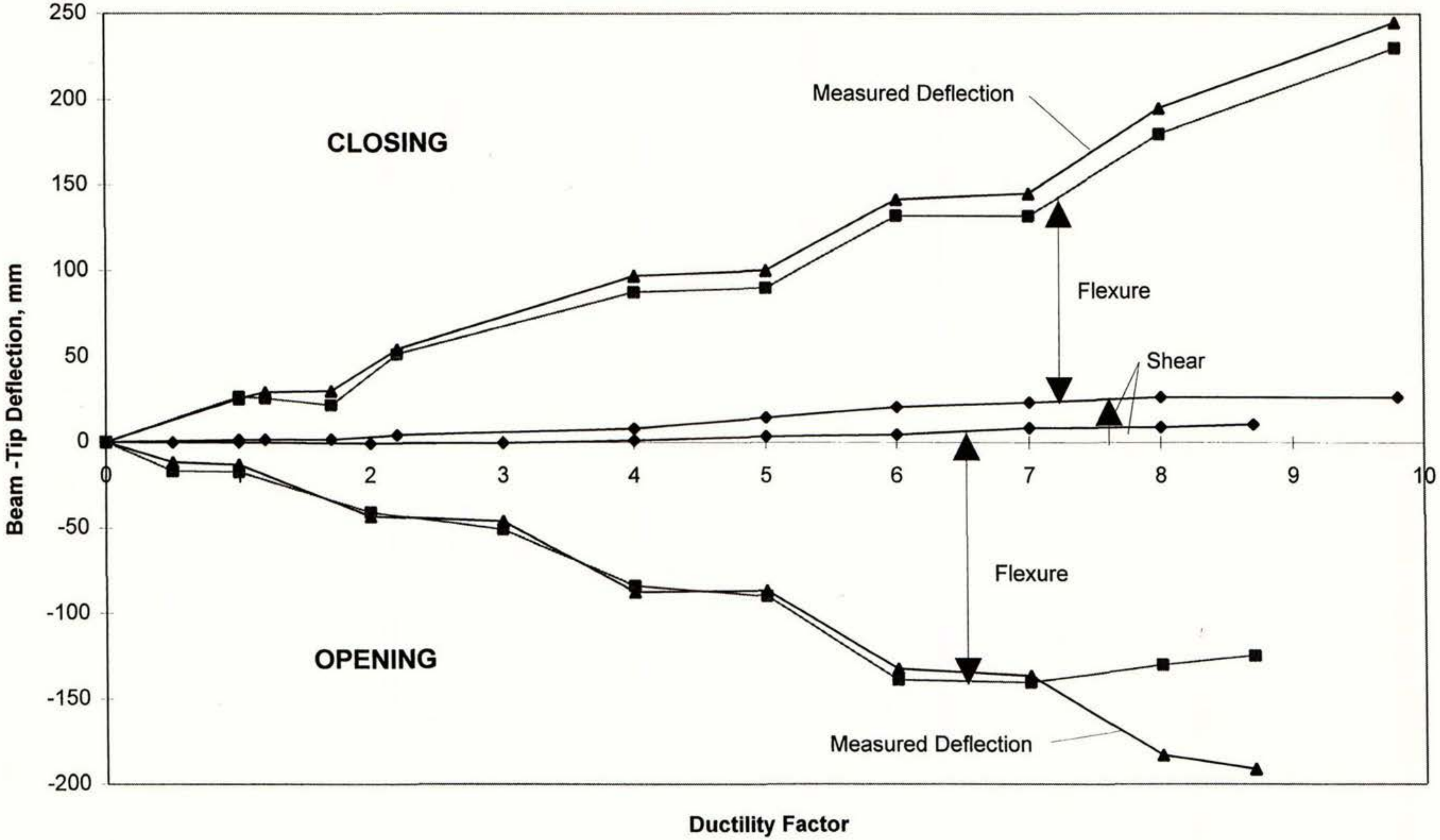
than yielding of the beam bars in the beam plastic hinge. This slip was the cause of nearly 150 mm of the beam-tip deflection in the ductility 6 cycles, while the beam hinge zone deflection was only causing about 25 mm of the tip deflection. Only when the very wide column face crack closed did the slipping stop and some strength was then able to be sustained by the beam.

The maximum joint shear stresses sustained in knee 3 were $0.095f_c'$ ($0.55\sqrt{f_c'}$ MPa) and $0.079f_c'$ ($0.46\sqrt{f_c'}$ MPa) under closing and opening conditions, respectively. Figure 21 shows the condition of knee 3 at opening ductility -6 for the second time.

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 22 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. Figure 23 shows the cycle peak deflections, both shear and flexure+axial which developed during testing. 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), while the beam hinge deflection was about 60% of the beam-column zone deflection at ductility 4 but reduced

FIGURE 20: KNEE 3 Flexural + Axial and Shear Deflections with measured beam-tip deflection at cycle peaks.



4-2

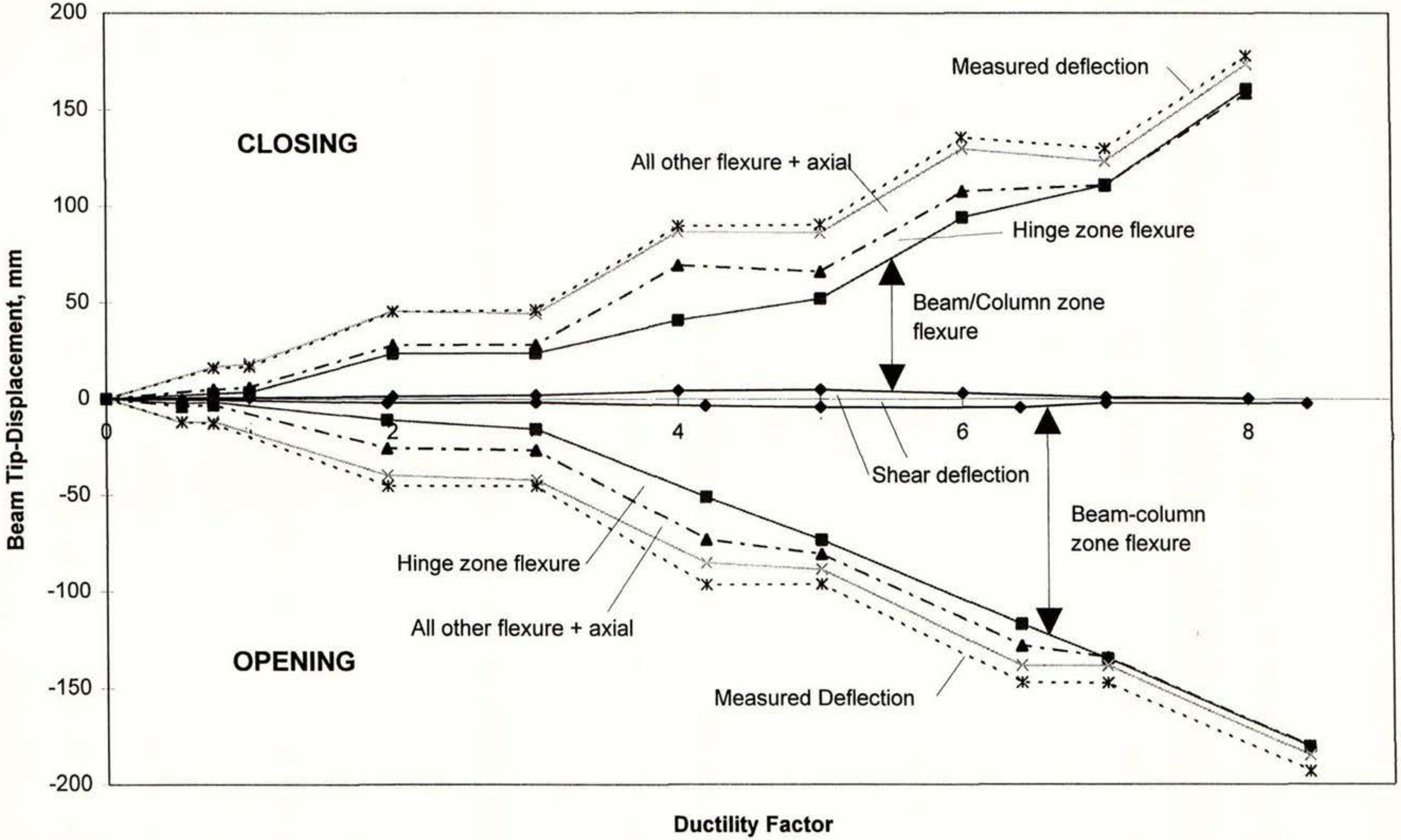
FIGURE 21: Knee 3 at opening ductility -6.

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 figure also shows that the shear deformations in this test were almost negligible.

Spalling of the beam cover next to the joint occurred in the second cycle to ductility 4, reducing the effective depth in the closing cycles. This spalling only happened over a 50 mm length measured out from the column face. The state of this joint at the first cycle at opening ductility -6 is shown in Figure 24.

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 25 show that the nominal closing and opening strengths were exceeded in both 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 about a 15% 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.

FIGURE 23: KNEE 4 : Flexure +Axial & Shear Deformations and Measured Beam-Tip Deflection at cycle peaks.



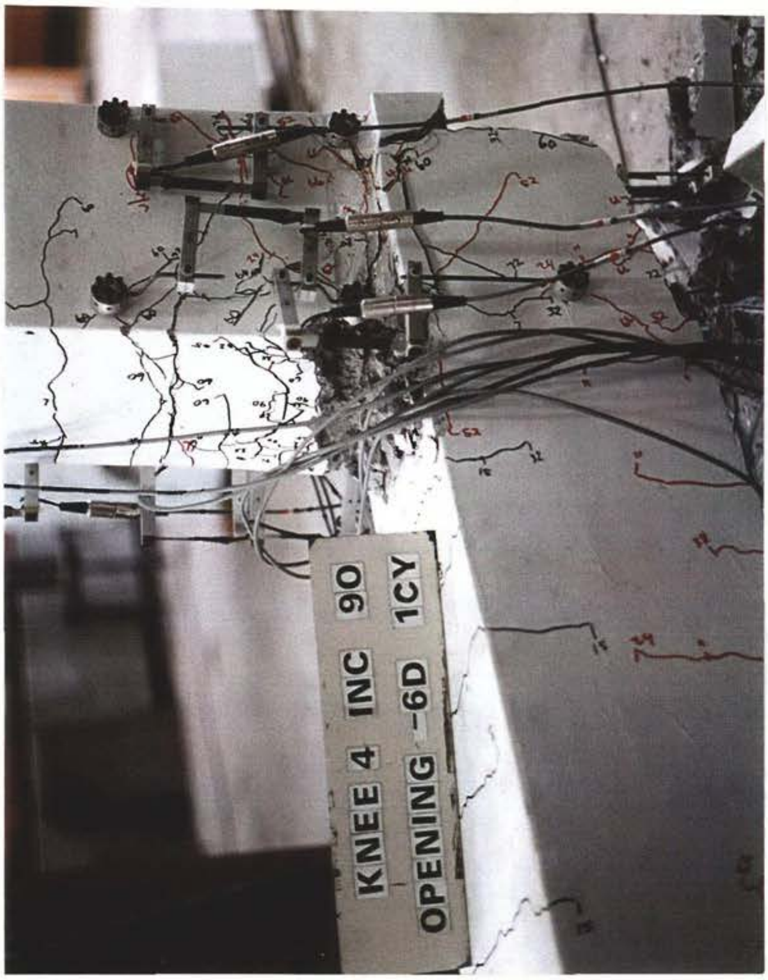


FIGURE 24: Knee 4 at opening ductility -6.

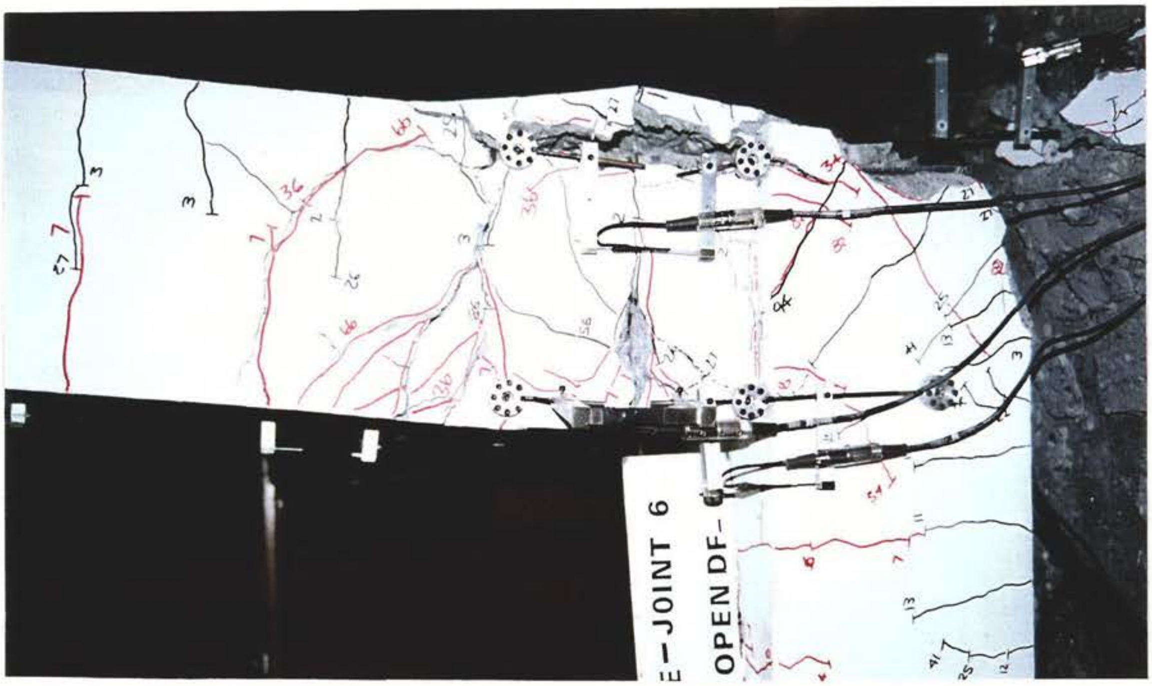
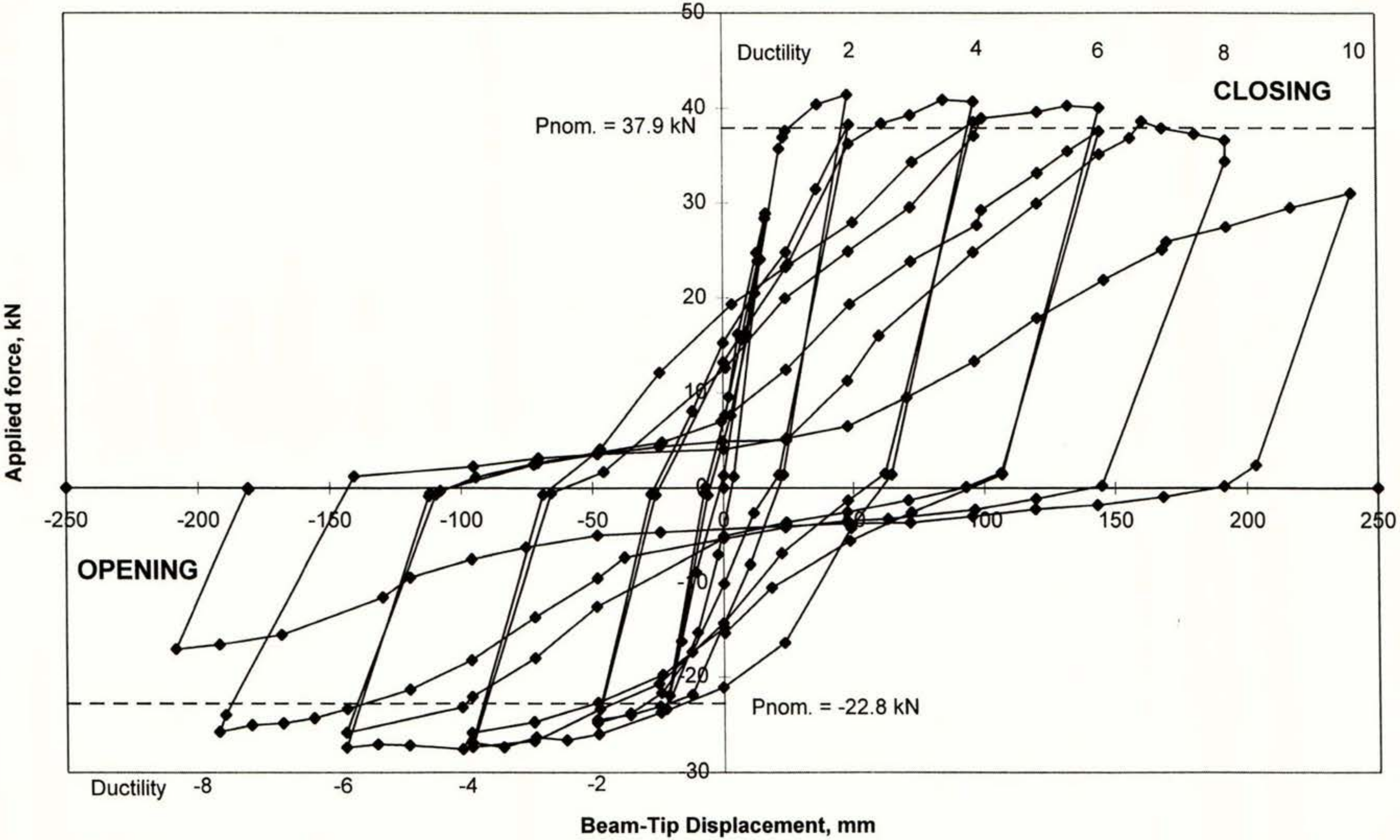


FIGURE 27: Knee 6 at opening ductility -6.

FIGURE 25: KNEE 6 Applied Force - Beam-Tip Deflection



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.

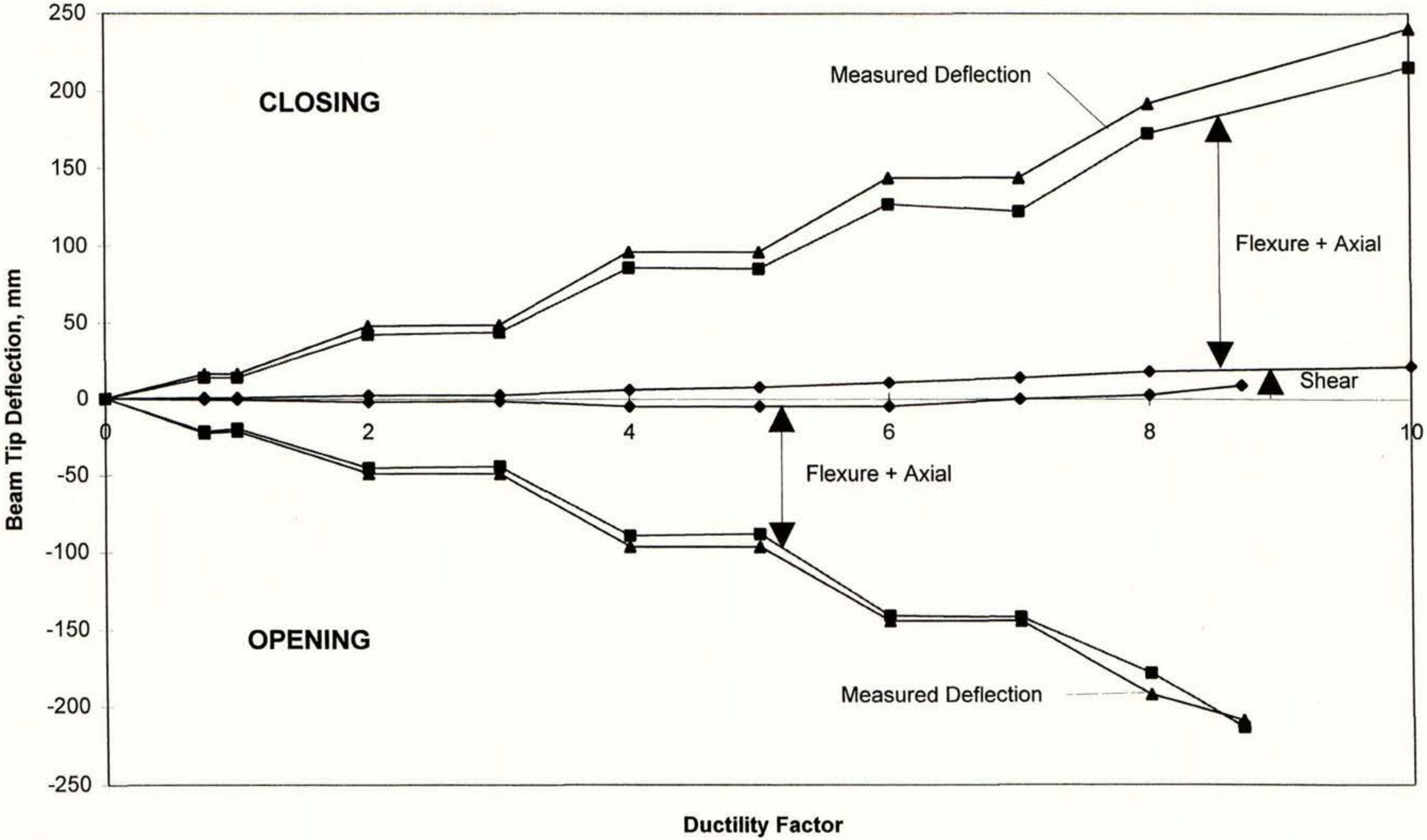
The core of the joint remained secure during this test, with only minor cracking evident in the side cover concrete. This is shown in Figure 26, where the shear deflections are very minor compared with the increasing flexural deflections throughout the test. In the ductility 8 cycle the beam-column zone 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 27 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 knee joint 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 previous joints. The maximum joint shear stresses reached in this test were $0.11f_c'$ ($0.64\sqrt{f_c'}$ MPa) and $0.073f_c'$ ($0.42\sqrt{f_c'}$ 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 members (250 x 200 mm beam, 250 mm square column). The problems of placing the large D20 bars and the 8 sets of horizontal joint ties in the

FIGURE 26: KNEE 6 Flexure + Axial & Shear Deflections compared with the measured beam-tip deflections at cycle peaks.



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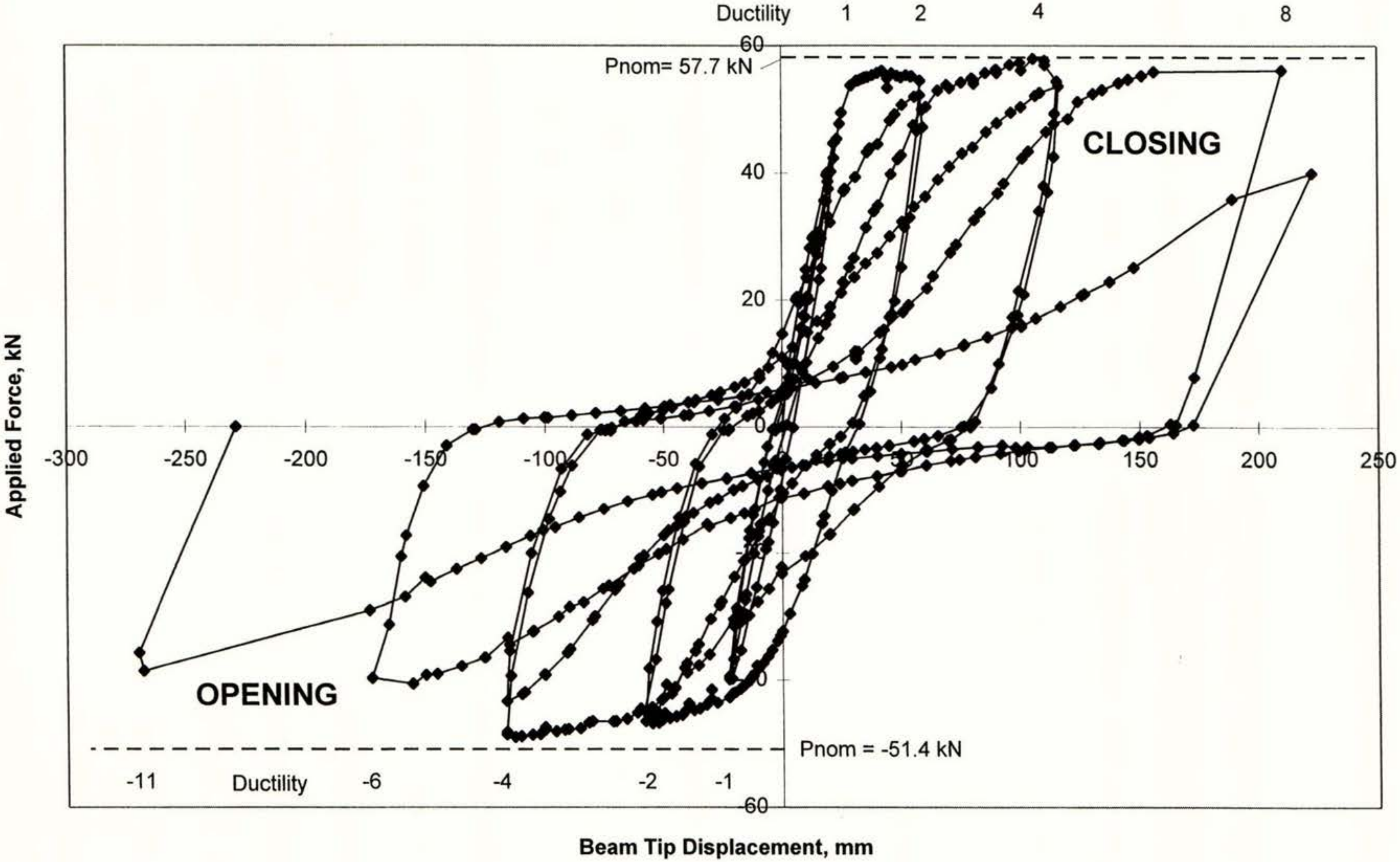
confined joint were time consuming and caused the cover to the main bars to increase to 23 mm, rather than the desired 20 mm.

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 28. 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 reached 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 cracks in both directions. The top and back of the joint also had numerous fine cracks. 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 cycles. The close spacing of the horizontal joint ties facilitated the splitting off 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

Chart22

FIGURE 28: KNEE 7 Applied Force - Beam-Tip Displacement



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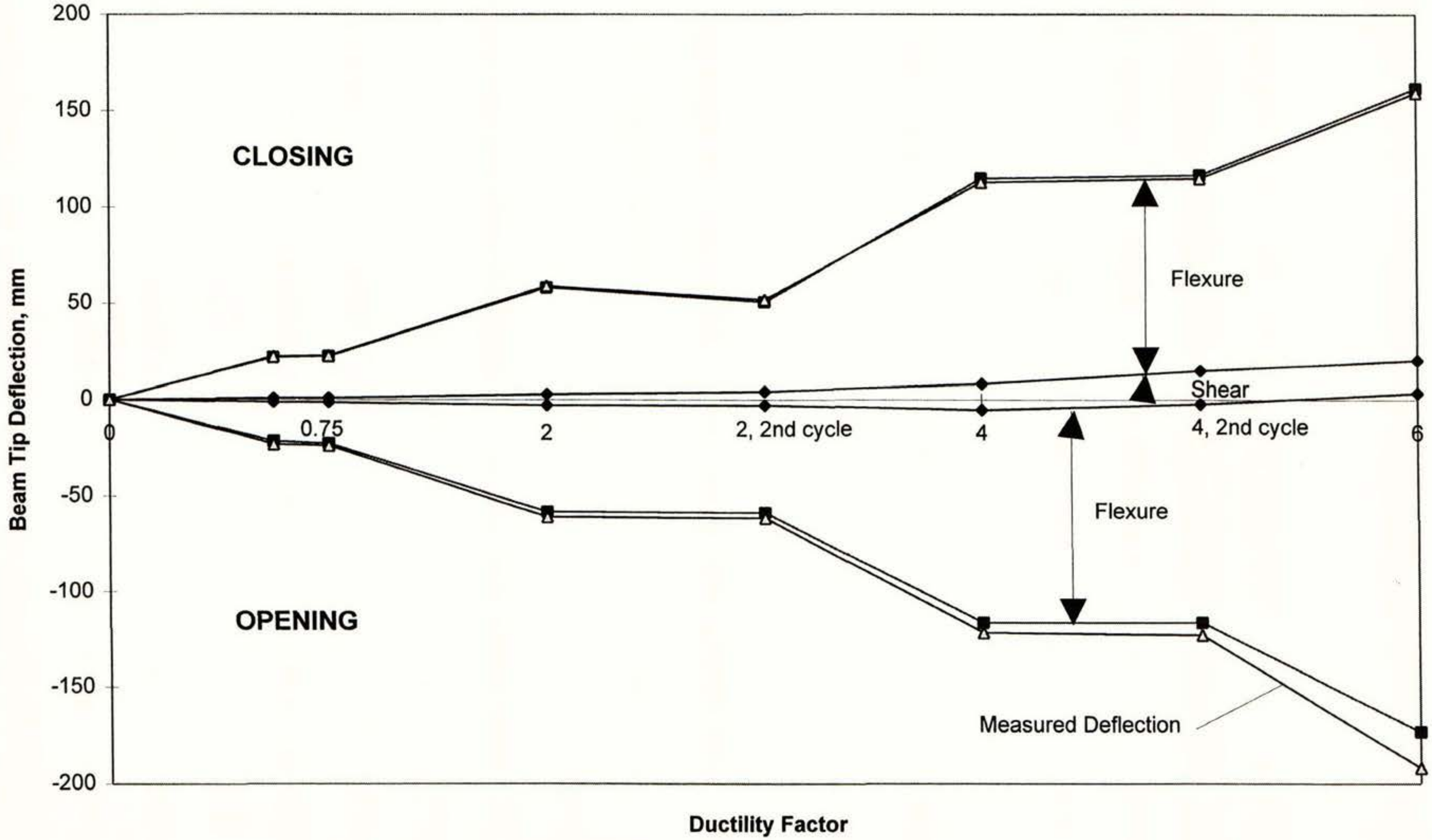
joint degradation. At this stage some slipping of the beam bars through the joint occurred, showing that the reduction in hook development length (L_{dh}) in this unit was causing bond failure within the joint. L_{dh} was only 80% of that required for "standard hooks" although this unit used U-bars. The standard 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. Figure 29 shows the calculated beam-tip deflections due to shear and flexure+axial at the cycle peaks. The deformations in the cycle at ductility factor greater than 6 are not included, as two of the portal gauges proved unreliable, due to disruption by the spalling cover concrete.

Figure 30 shows the joint at opening ductility -6, first cycle, with major cover loss to the top, back and sides of the joint.

The horizontal joint shear stresses reached in knee 7 were $0.101f_c^{'}$ ($0.72\sqrt{f_c^{'}}$ MPa) closing and $0.084f_c^{'}$ ($0.60\sqrt{f_c^{'}}$ 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 the joint.

FIGURE 29: KNEE 7 Flexural & Shear deflections at cycle peaks compared with measured beam-tip deflections.



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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. The loading sequence was reversed from that used previously, with the first cycle to $0.75M_n$ in the OPENING direction. Figure 31 is the applied force versus the beam-tip deflection plot, which shows that 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.

Beam hinging occurred initially and 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 opposite 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.

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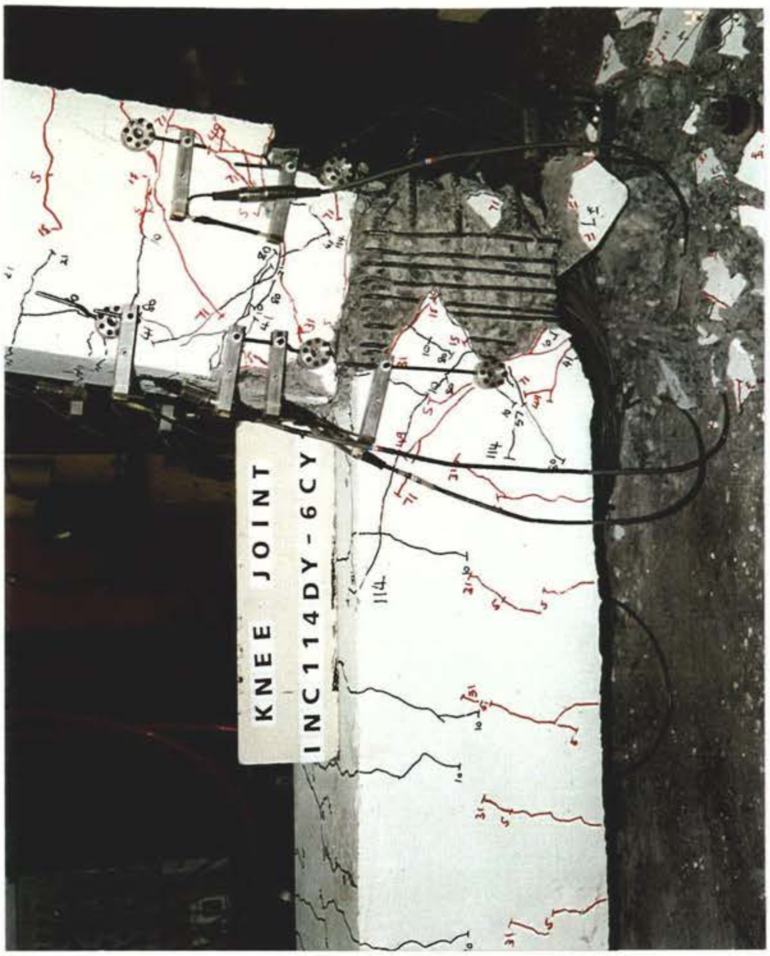


FIGURE 30: Knee 7 at opening ductility -6, first cycle.

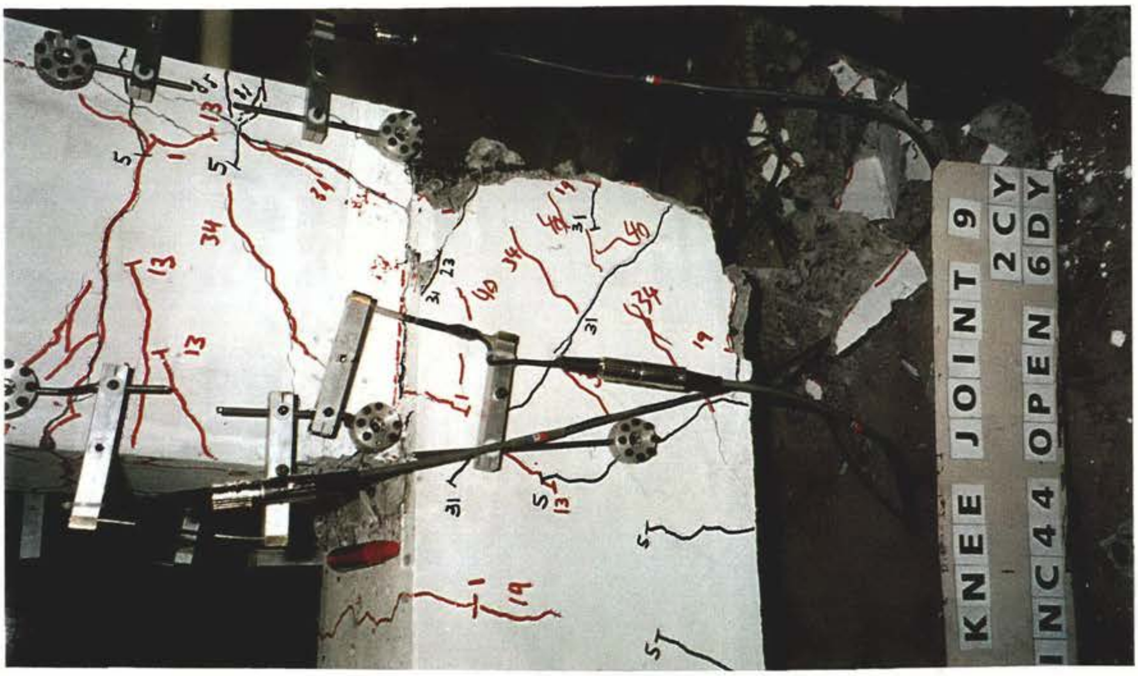
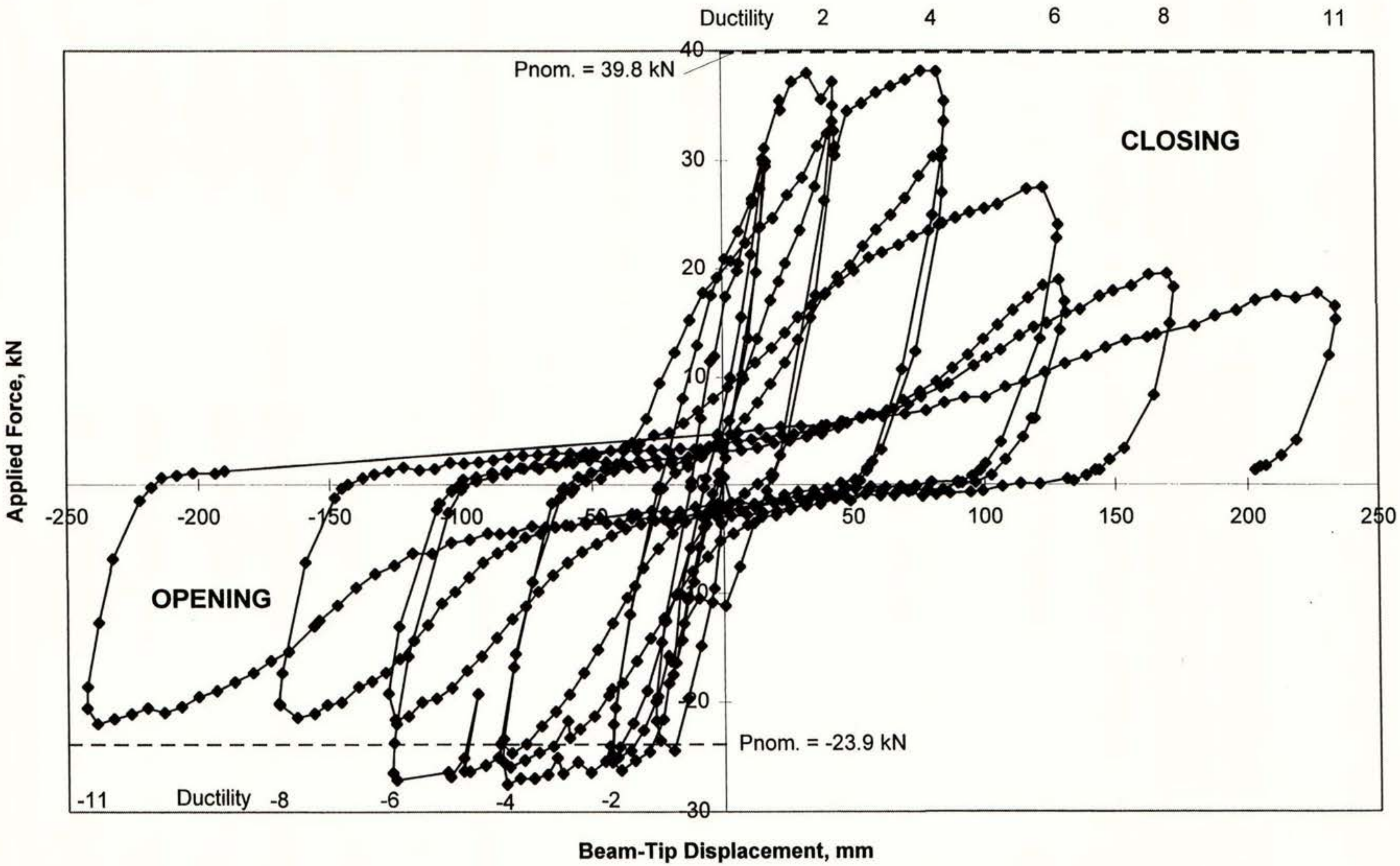


FIGURE 32: Knee 9 at second cycle to opening ductility -6.

FIGURE 31: KNEE 9 Applied Force - Beam-Tip Deflection



LS

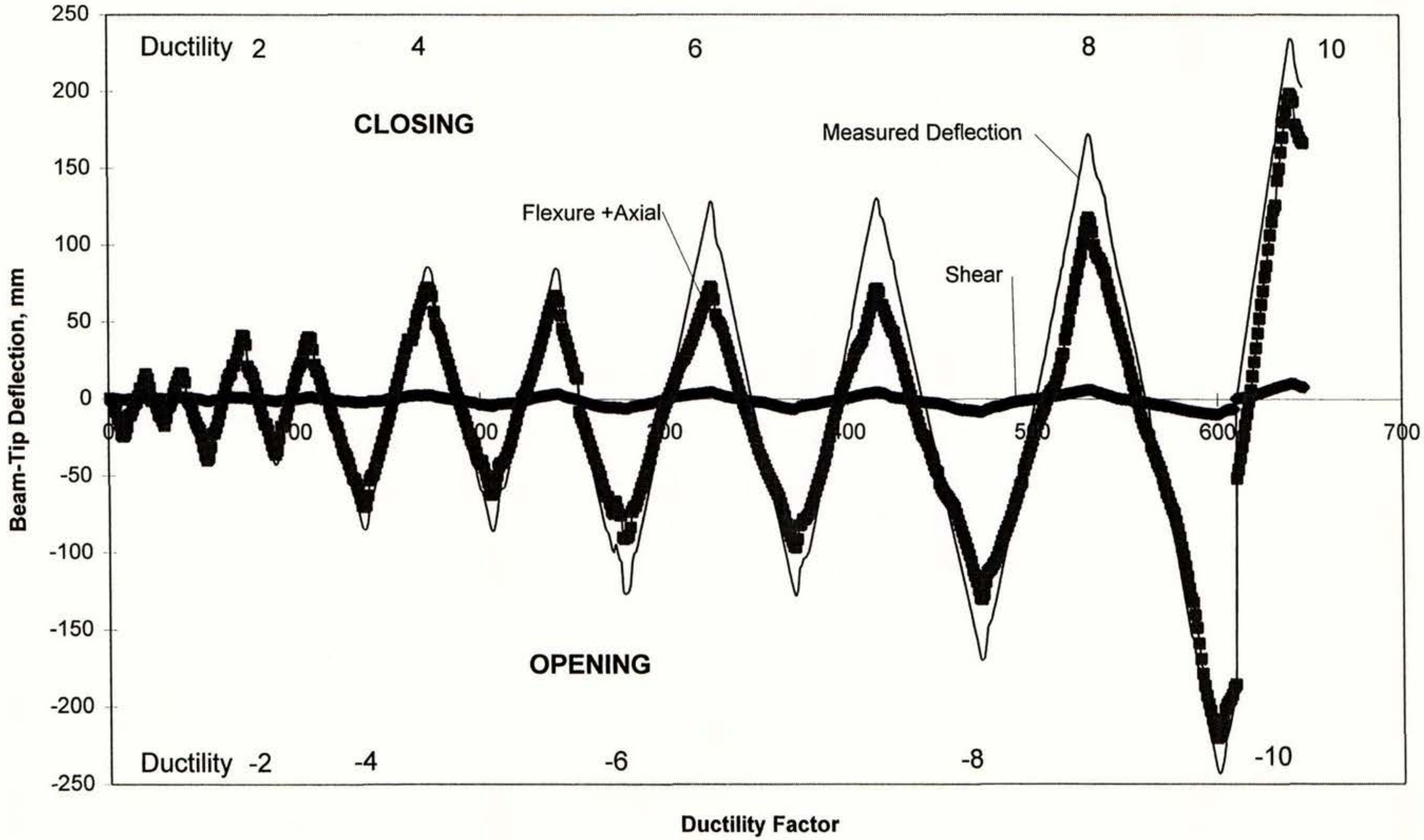
In the ductility ±6 cycles the remaining top and back joint cover fell off and up to 20 mm of bar slip was apparent at the column/joint interface, in the closing direction. In the second opening cycle to ductility -6 the moment reduced to 90% of M_n, due to the loss of cover concrete at the outer column face causing a decrease in beam 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. Figure 32 shows the joint's condition at opening ductility -6, second cycle.

Figure 33 shows the calculated flexural plus axial beam-tip deflections, along with the shear deflections, compared with the measured beam-tip deflections throughout this test.

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. In knee 10 the two D16 transverse bars were provided within the 90-degree bends in the three joint corners, see Figure 10. These extra bars had been omitted from knee 9.

FIGURE 33: KNEE 9 Calculated Flexural + Axial, Shear & Measured Beam-Tip Deflections



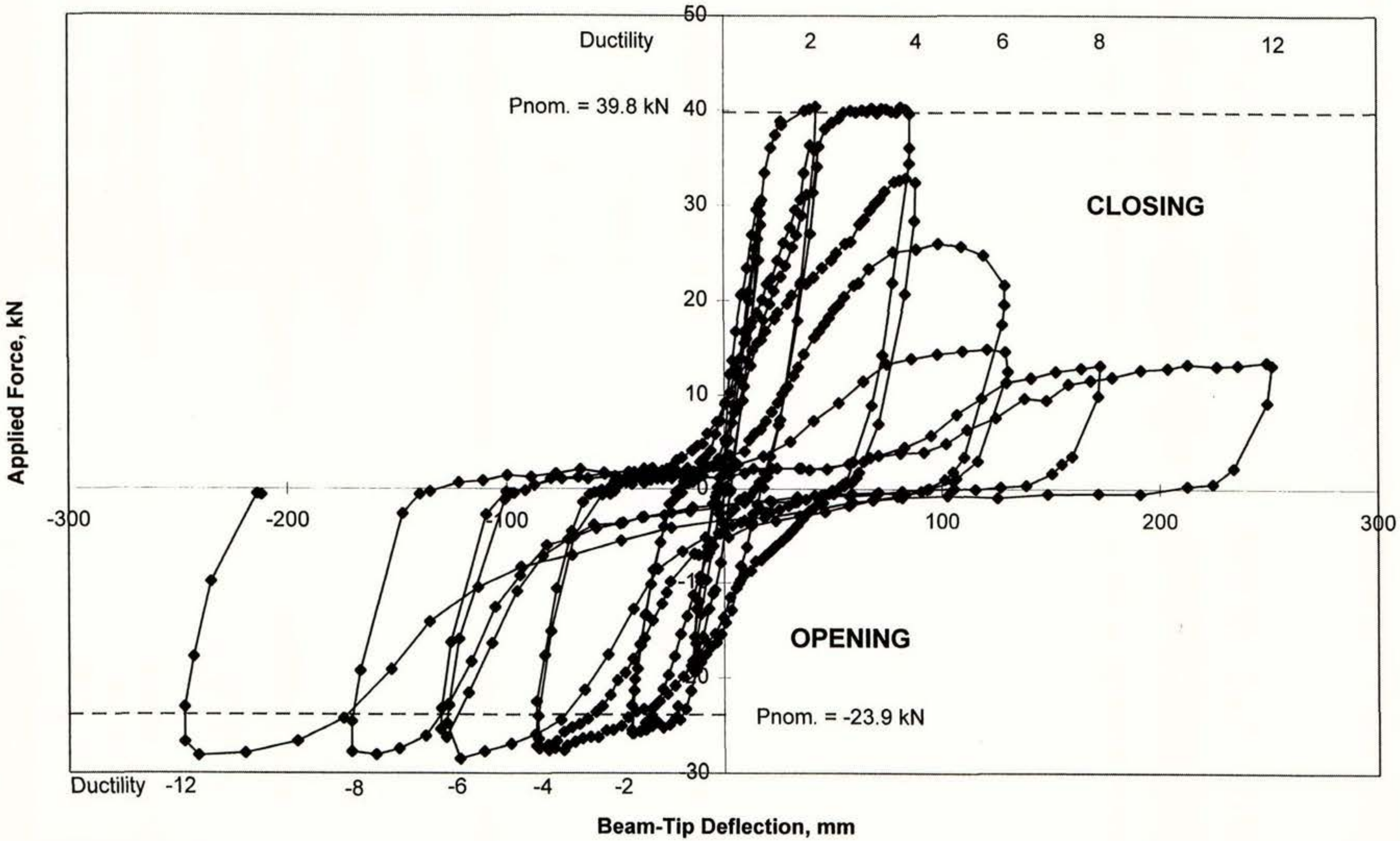
The applied force versus beam-tip deflection hysteresis loops are reproduced in Figure 34. Performance under opening bending moments was excellent, with the nominal strength being exceeded up to and at ductility -10 displacements. However under closing conditions there were strength reductions at the 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.

Cracking occurred in the beam and column in the elastic cycles with one crack across the top of the joint about 100 mm in from the column face under closing moments. This crack spread to the sides of the joint and moved up to the column face at a slope of about 45-degrees. In the ductility -2 opening first cycle three new cracks appeared across the joint's top and 3 splitting cracks formed on the top, which lined up with the main reinforcing. Splitting cracks also formed around the outer 90-degree bend on the sides of the joint. One diagonal joint crack formed about 100 mm in from the re-entrant corner, spreading from the centreline of the beam to the centreline of the column.

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

FIGURE 34: KNEE 10 Applied Force- Beam Tip Deflection



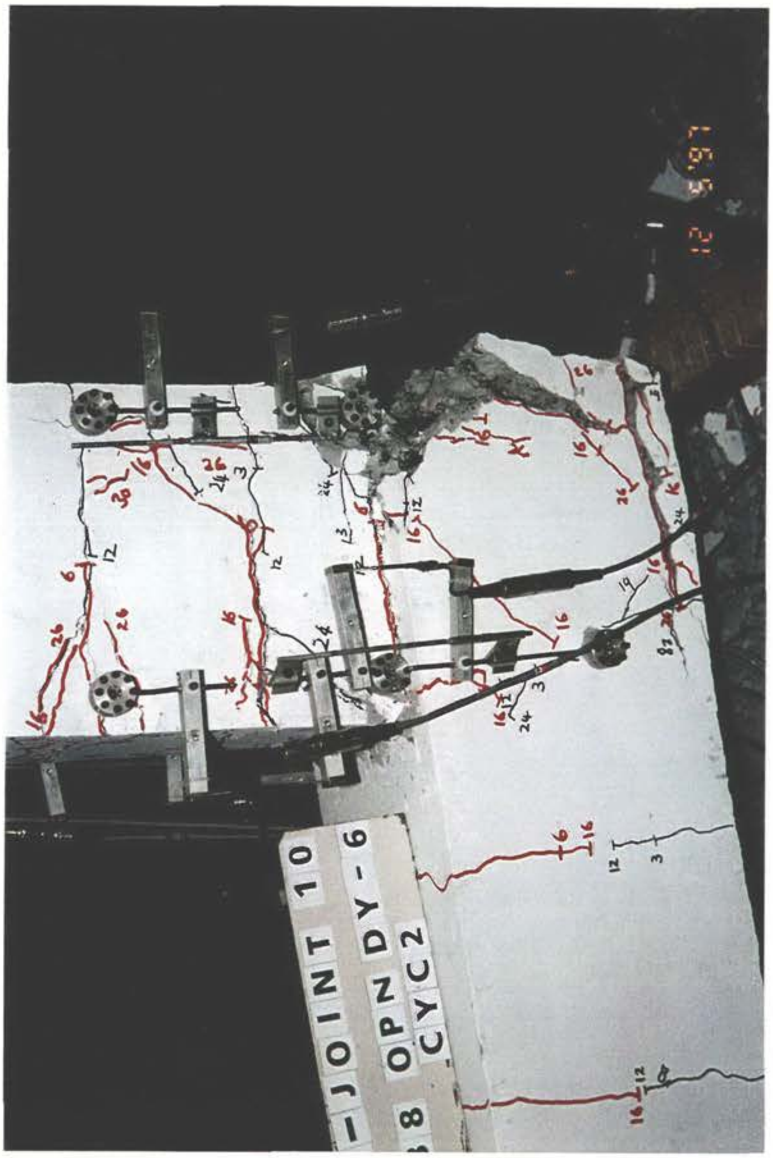


FIGURE 35: Knee 10 at second cycle to opening ductility -6.

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. Figure 35 shows the knee joint at the second opening cycle to ductility -6.

As in knee joint 9, the shear deformations were very small and they are shown together with the flexural and axial deflections calculated at the beam-tip, along with the measured beam deflection in Figure 36.

Figure 37 shows the applied force versus the beam hinge zone deflection loops, while Figure 38 shows the force-deflection loops for the 250 mm long beam zone out from the 200 mm long hinge zone. Both zones show substantially more plastic hinge behaviour under opening conditions than under closing actions. In Figure 39 the beam-tip deflections due to distortion of the short beam-column zone are plotted against the applied force. This shows that much of the closing deformation was due to bar slip, as one end of the portal gauge was fixed to a column bar while the other end was attached to the adjacent beam bar.

Chart21

FIGURE 36: KNEE 10 Flexural+Axial, Shear and Measured Beam-Tip Deflections

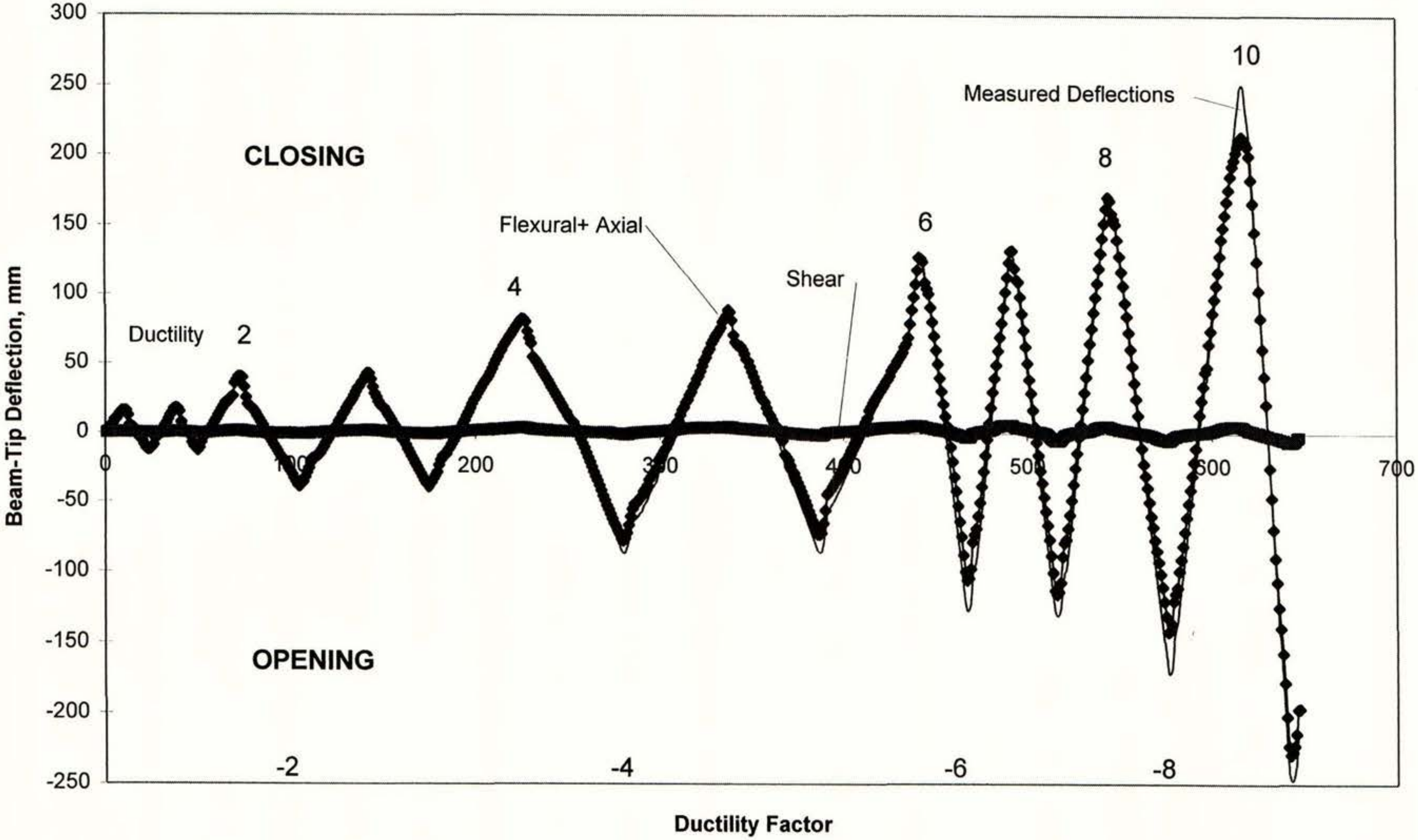


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FIGURE 37: KNEE 10 Applied Force-Beam Hinge zone Deflection calculated at the beam-tip.

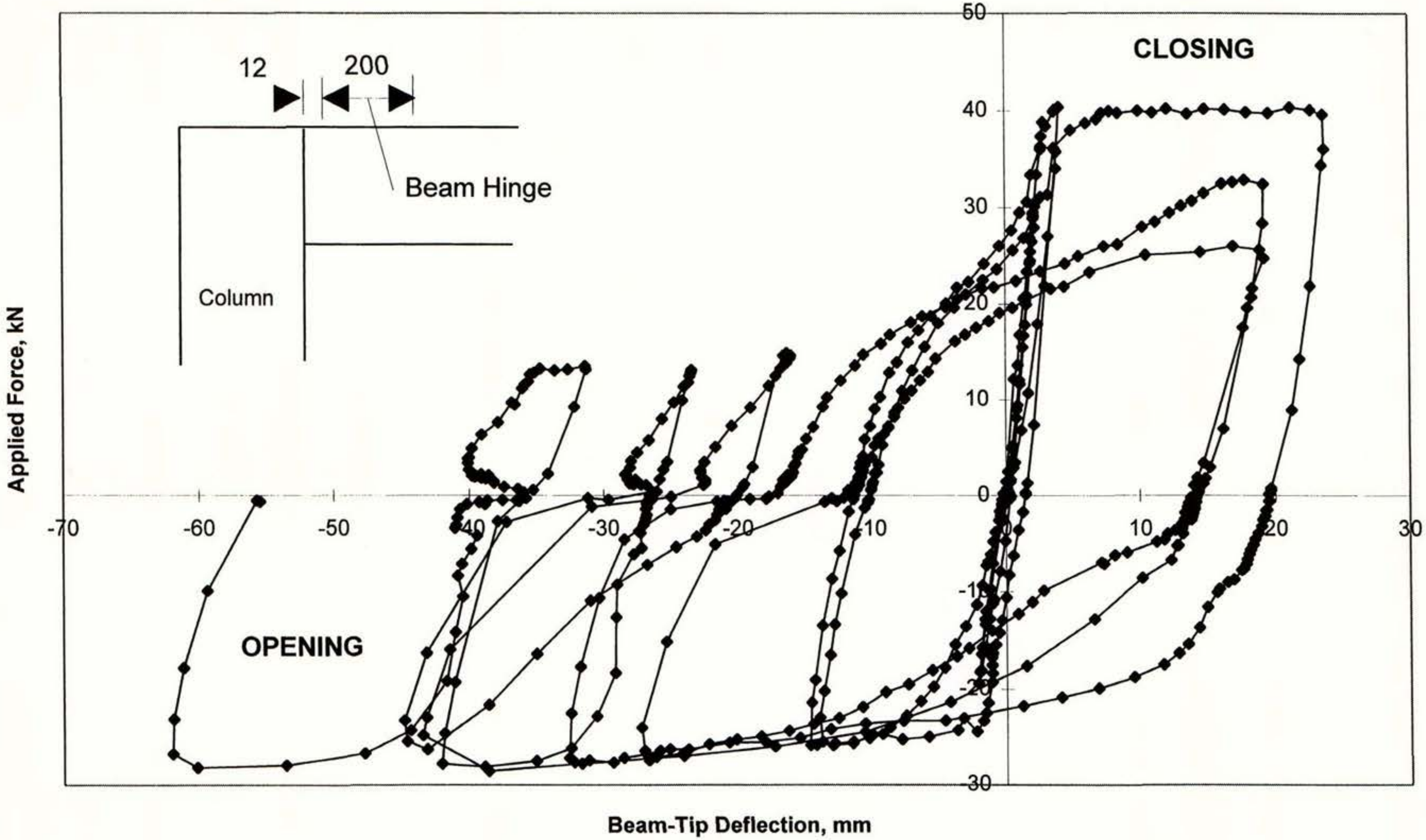

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FIGURE 38: KNEE 10 Applied Force - Beam zone 2 Deflection, 212 mm out from the column face, calculated at the beam-tip.

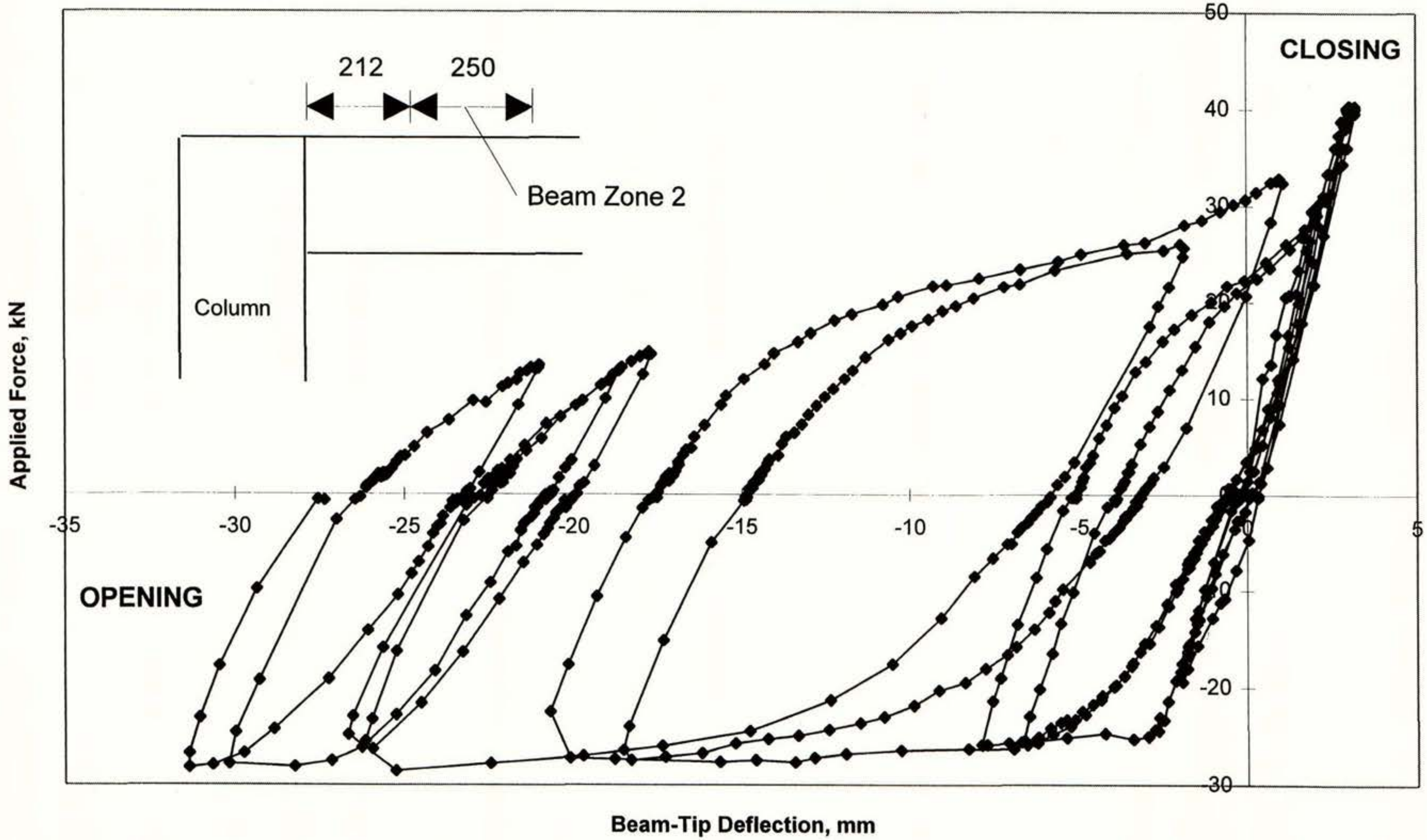
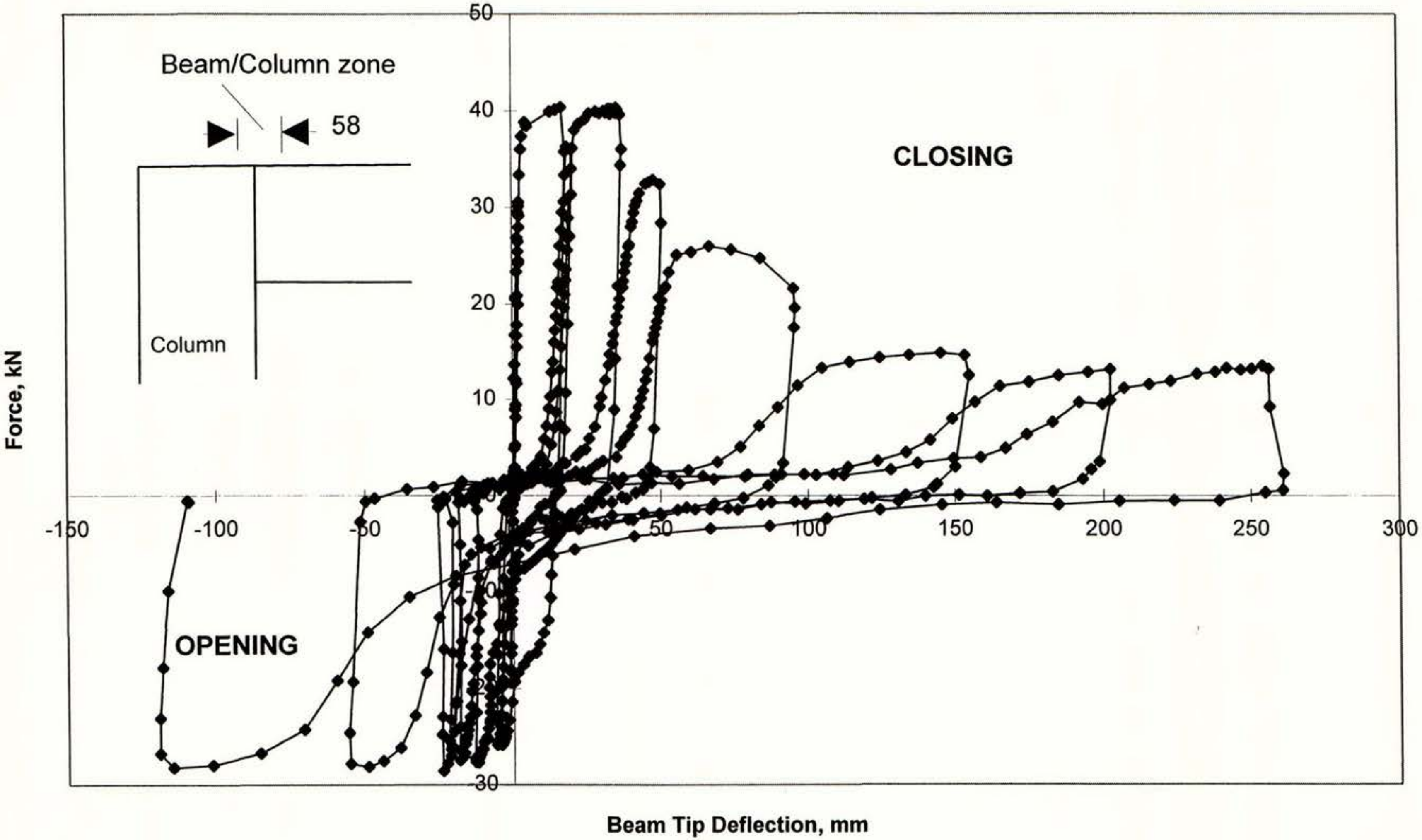


FIGURE 39: KNEE 10 Applied Force- Beam/Column zone deflection calculated at the beam-tip.



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4. RESULTS OF 1960's DESIGNS

4.1 Knee Joint 5: This joint had D16 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 force of only 67% of M_n was attained.

Continued pump pressure resulted in the load reducing to 58% of M_n (15 kN jack force) at an approximate displacement ductility of one at about 25 mm beam-tip deflection.

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. Thus 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 when compared with the poor performance of similar unreinforced (for shear) joints, studied in Europe in the 1970's, under monotonic forces, Megget (1994). The full force-displacement loops are shown in Figure 40.

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. Figure 41 shows the proportions of beam-tip deflection made up from joint shear, flexure and beam-column zone flexure at the cycle peaks.

The horizontal joint shear stresses were low at $0.072f_c^{'}$ ($0.42\sqrt{f_c^{'}}$ MPa), closing and $0.04f_c^{'}$ ($0.23\sqrt{f_c^{'}}$ MPa), opening. Priestley (1996) 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. 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. Figure 42 is a photo of knee 5 at closing ductility 4, first cycle, showing severe joint damage and negligible cracking in the beam or column.

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 earlier 1960's joint, in that the maximum strengths reached under closing and opening moments

FIGURE 40: KNEE 5 Applied force - Beam-tip Deflection

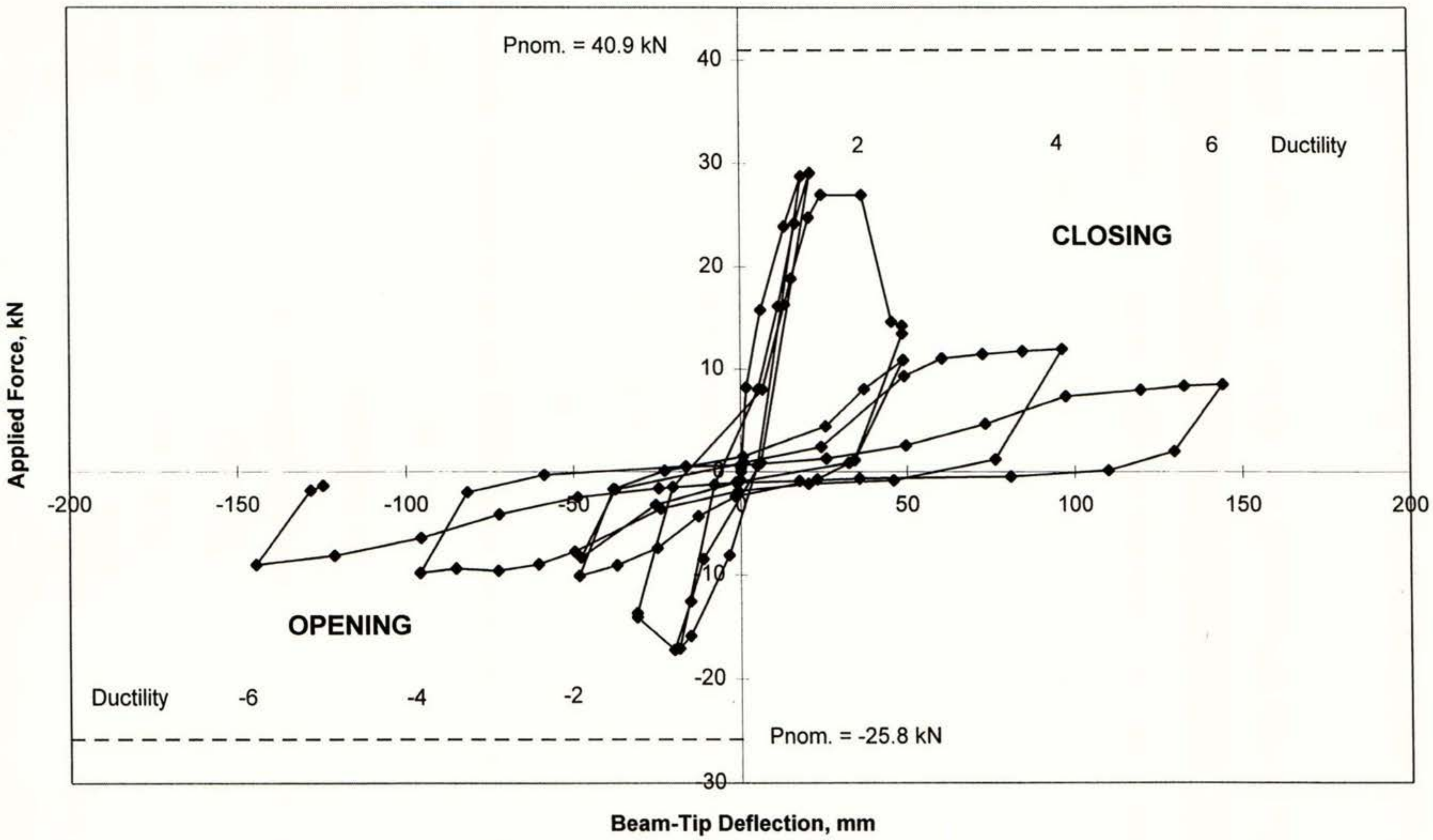
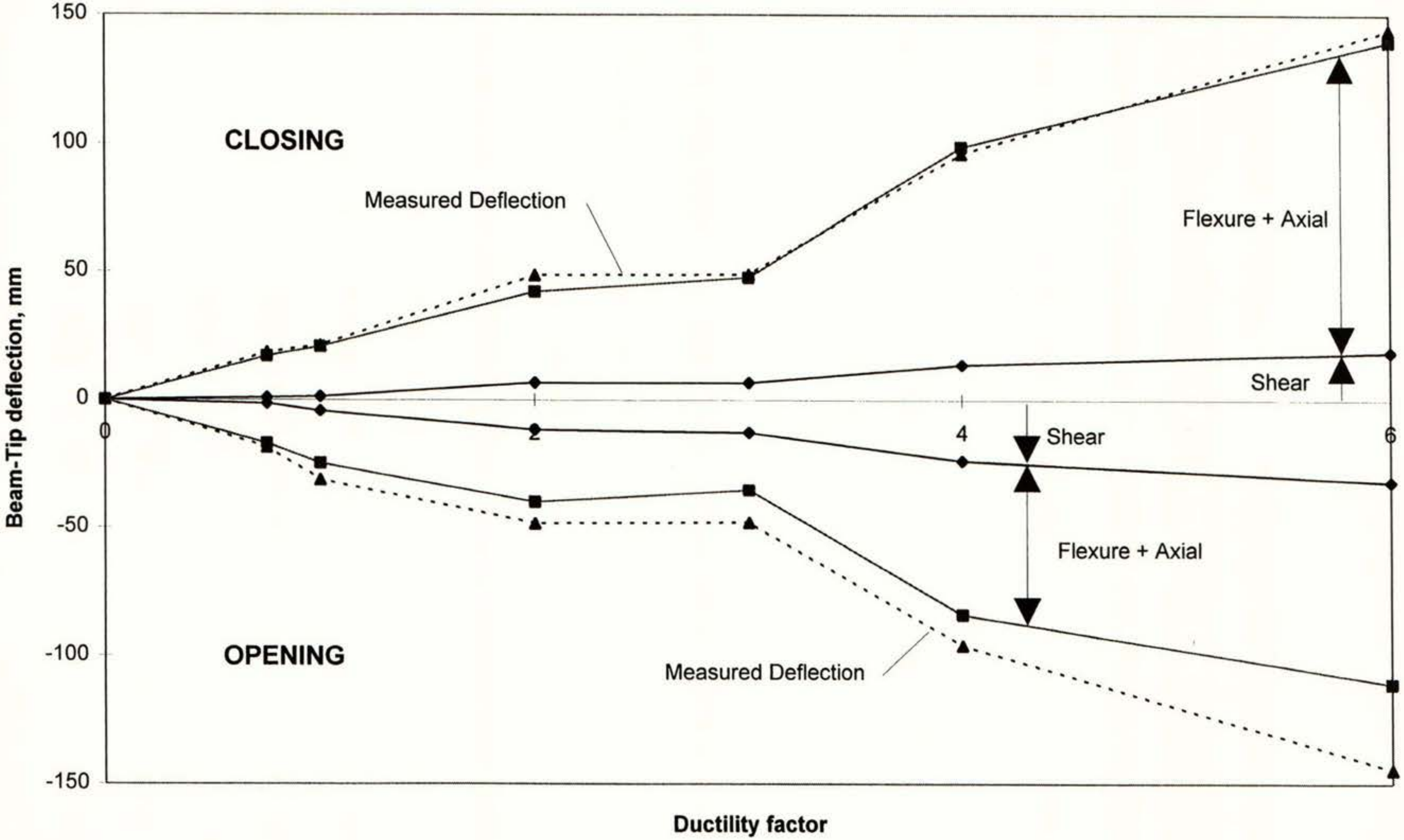


FIGURE 41: KNEE 5 1960's Design, Flexure+Axial & Shear Deformations and Measured Beam-Tip Deflections at cycle peaks.



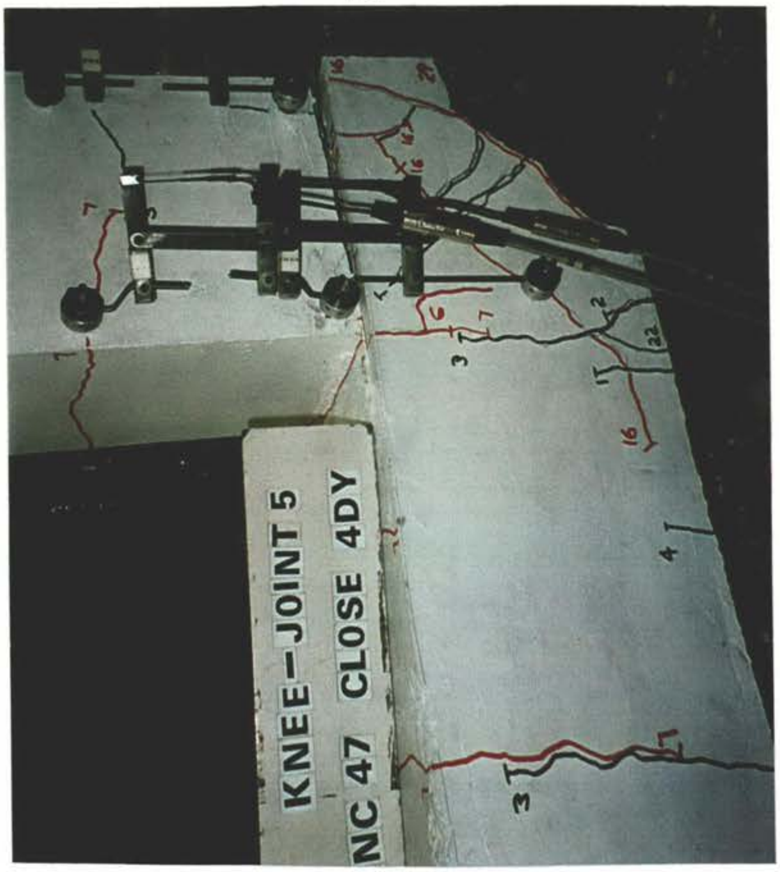


FIGURE 42: Knee 5 (1960's design) at closing ductility 4.

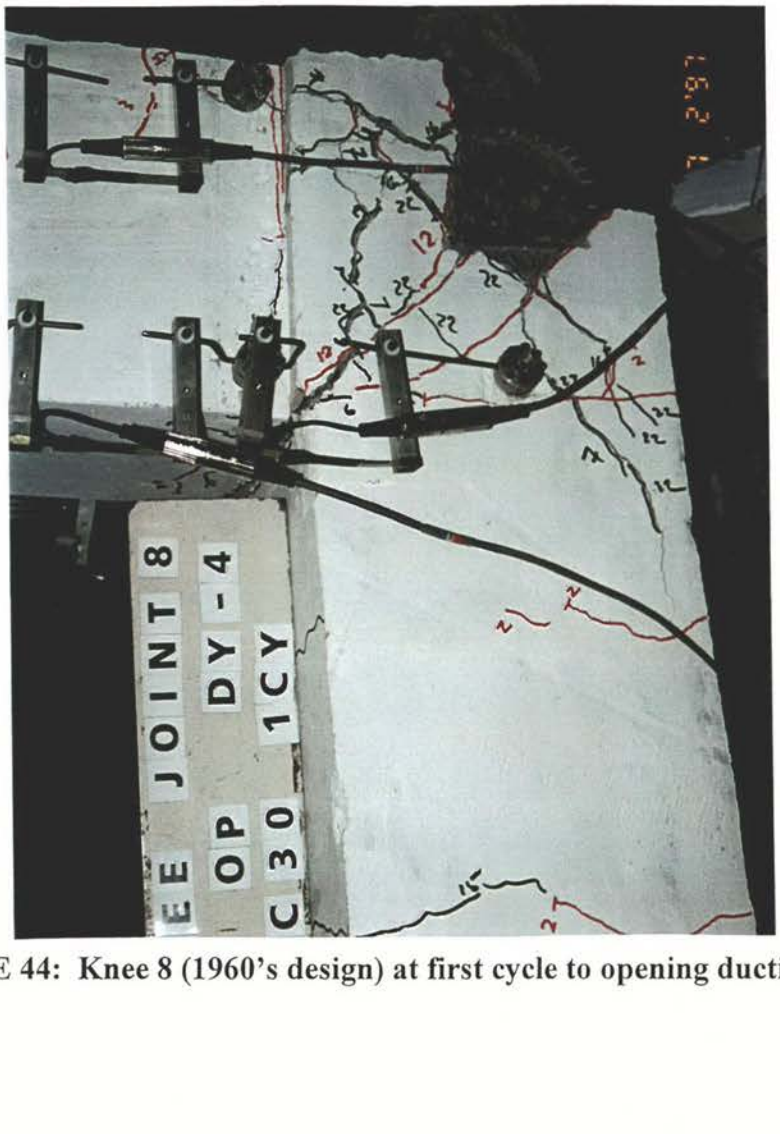


FIGURE 44: Knee 8 (1960's design) at first cycle to opening ductility -4.

were only 81 and 84% 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 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 cycle to ductility ±4, as shown by the force-displacement loops in Figure 43. 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 44 at opening ductility -4, first cycle. Note the lack of flexural cracks in the beam hinge zone, confirming the elastic behaviour recorded in the hinge region.

Figure 45 shows that the majority of the beam-tip deflection was due to flexure and bar slip, rather than shear in the joint, which accounted for approximately 25% of the deflection once joint failure had occurred. The maximum joint shear stresses reached were $0.063f_c^{'}$ ($0.40\sqrt{f'_c}$ MPa) and $0.038f_c^{'}$ ($0.24\sqrt{f_c^{'}}$ MPa) under closing and opening conditions, respectively. These stresses were a

Chart1

FIGURE 43: KNEE 8, 1960's Design, Applied Force - Beam-Tip Deflection

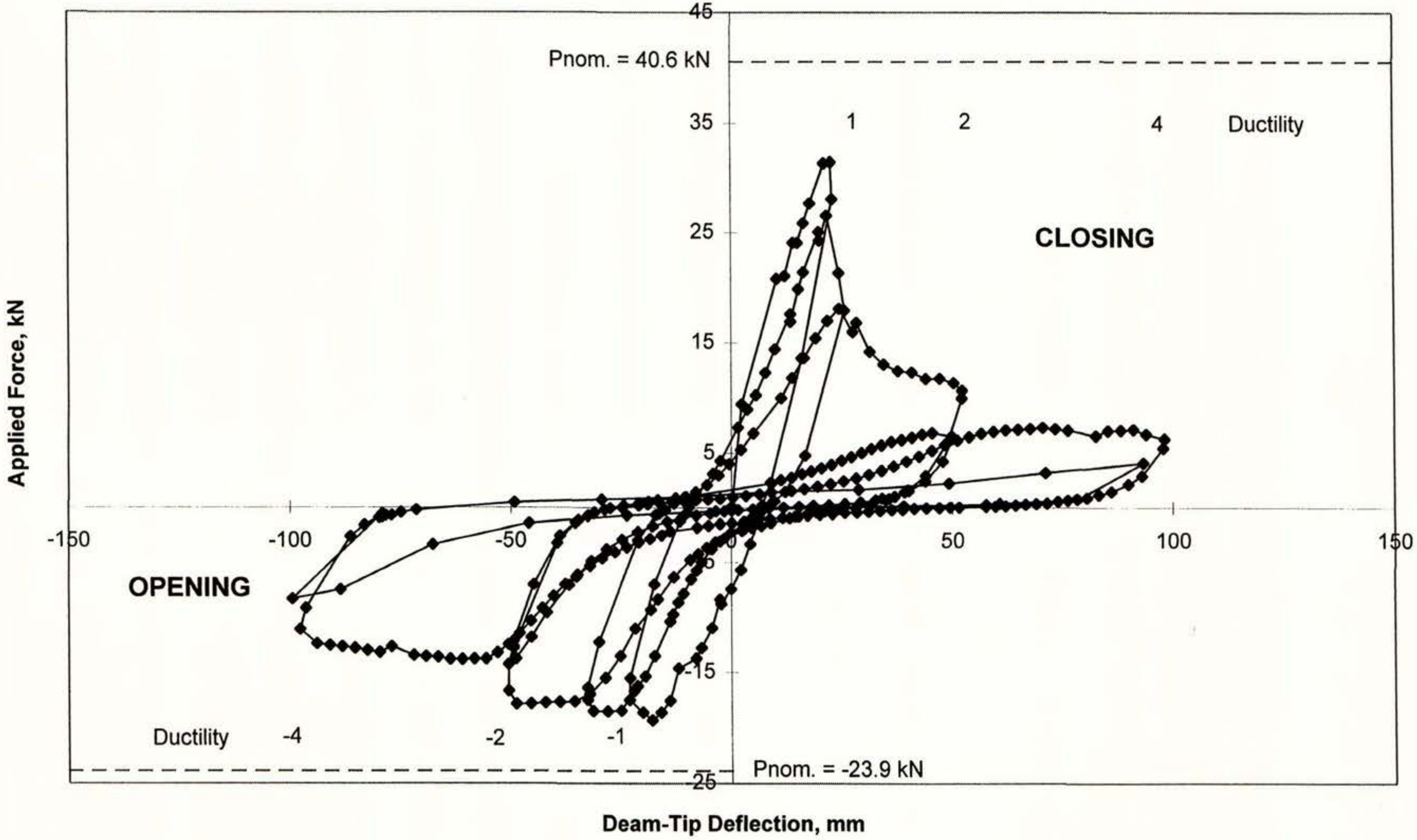


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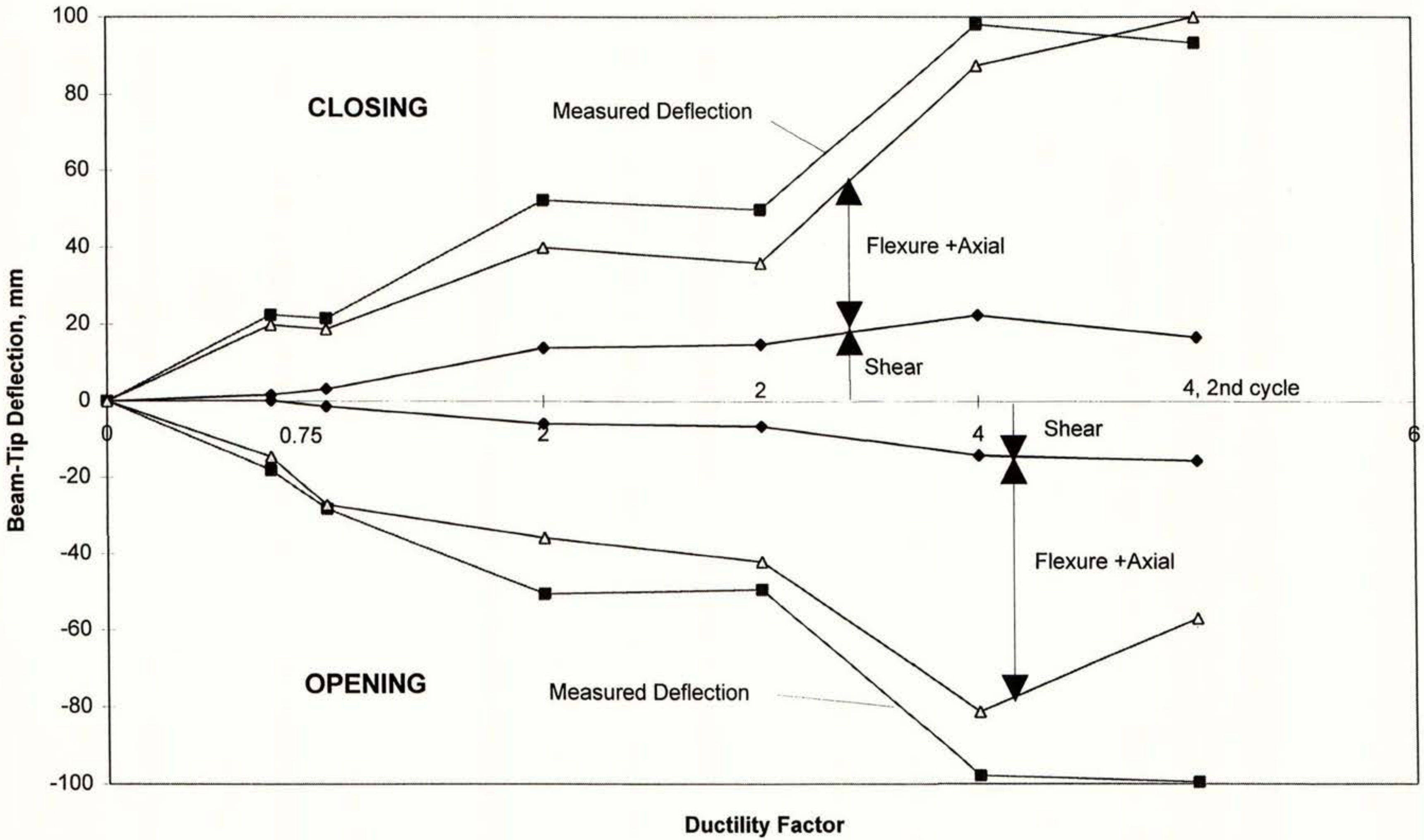
Chart19

FIGURE 45: KNEE 8 Flexure+Axial & Shear Deformations and Measured Beam-Tip Deflectionsat cycle peaks.



75

little less than those recommended by Priestley (1996) for unreinforced exterior joints. It was expected that the shear stress limit for knee joints would be less than the equivalent exterior joint, which has a column above the joint adding some confining effects to the joint zone.

5. PART B KNEE JOINTS WITH ANCHORAGE PLATES

This part consisted of 3 knee joint tests, where anchorage plates were welded to the beam and column principal reinforcing in an effort to improve the anchorage and shear performance of these small joints after the conventional details, described in Part A, which usually failed due to loss of anchorage of the main bars within the joint.

5.1 Knee Joint 11: This joint had a 50x50x6 mm anchorage plate welded to each beam and column main bar (D16). The plate had an 18 mm diameter hole in its centre and the bar was fillet welded around one side of the plate with a 6 mm leg weld. The plates were positioned at the end of each bar within the 20 mm concrete cover on the top and back of the joint, see Figure 46. The outer column bars were cranked inwards by 25 mm over the beam depth to allow their plates to pass inside the top beam bar plates so as to preclude an unreinforced section across the joint's diagonal, as occurred in Wallace's (1996) tests. This joint had only 3-6ϕ tie-sets within the joint depth and supposedly 1-D10 U-bar as vertical joint steel. All other beam and column reinforcement details were identical to earlier units but there were neither diagonal bars across the re-entrant corner nor transverse bars in the joint's corners.

The horizontal joint ties represented 112% of the A_{jh} area required using the actual value of f_{yh} and f_y= 300 MPa. The vertical joint U-bar amounted to 103% of the A_{jv} area required using the actual f_{yh} and f_{yv} values. However the D10 U-bar was inadvertently left out of knee 11 and thus there was no vertical joint shear steel.

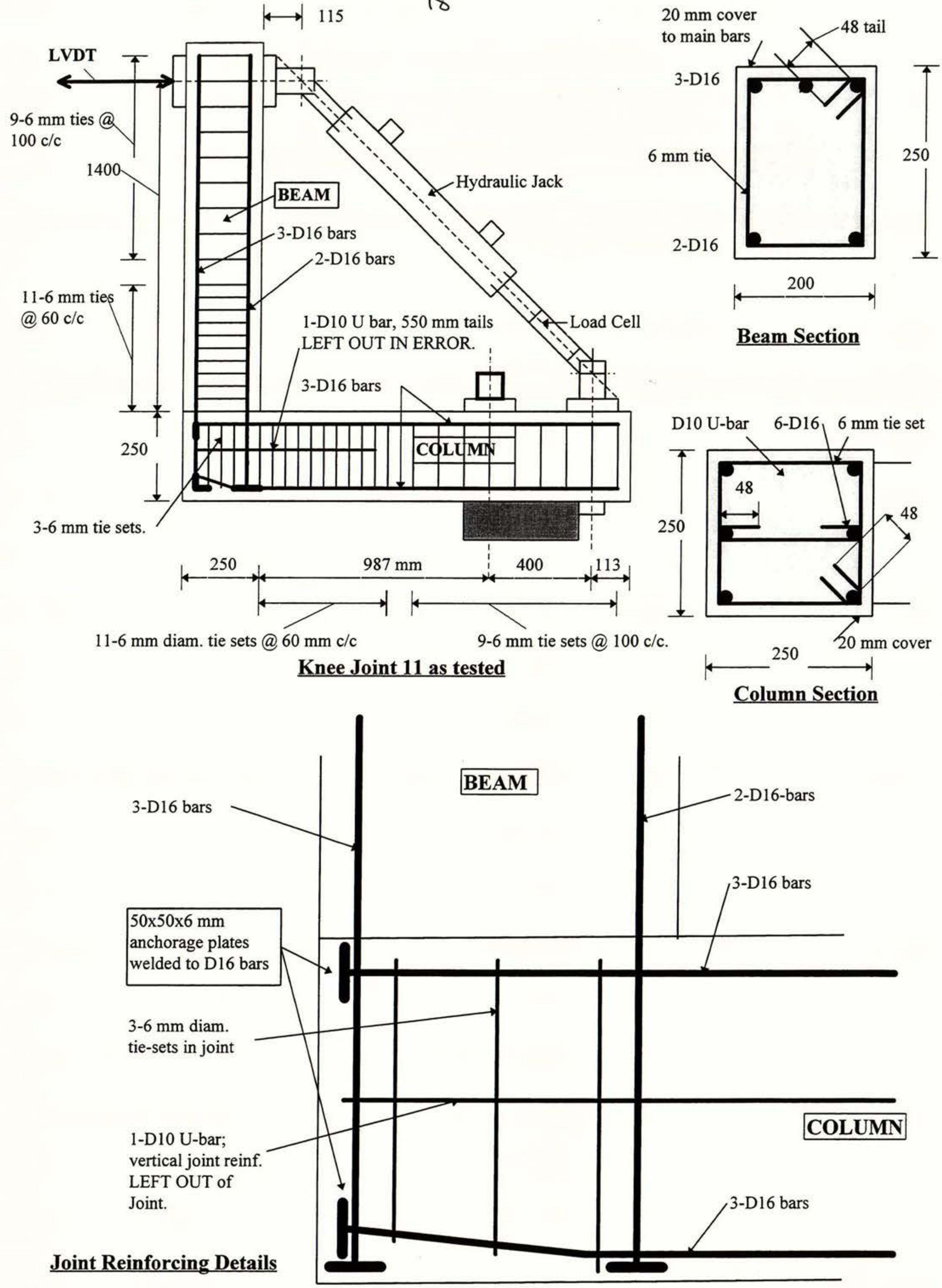


**Figure 46: Knee Joint 11 with anchorage plates.
3 Transverse joint ties included.**

5.2 Knee Joint 12:

This joint contained anchorage plates on the end of the main beam and column bars in the same way as in knee 11. However in knee 12 there were no transverse joint ties horizontally and no D10 U-bar vertically. Figure 47 shows the reinforcing details of knee joint 12. It was expected that this joint would behave poorly and fail due to shear in the joint zone, in a similar way to the 1960's designed joints.

5.3 Knee Joint 13:

This joint contained main beam and column **U-bars**, each with an anchorage plate welded just before the 90-degree bend. In the beam bar's case this was just before the bend at the bottom of the beam, while on the column bars it was positioned just before the bend on the inner side. The middle top beam bar was anchored with a standard 90-degree hook, with no anchorage plate at its end; see Figure 48 for the reinforcing details. The aim was to improve the diagonal strut across the joint by positioning the anchorage plates near the ends of the opening moment joint strut. Like knee 11 this knee had 3-6ϕ tie-sets horizontally and no vertical joint steel whatsoever, the hoped for improved diagonal strut reducing the requirement for vertical joint shear reinforcing.

The concrete compressive strengths and reinforcing steel yield stresses for knee joints 11,12 and 13 are given in Table 3.

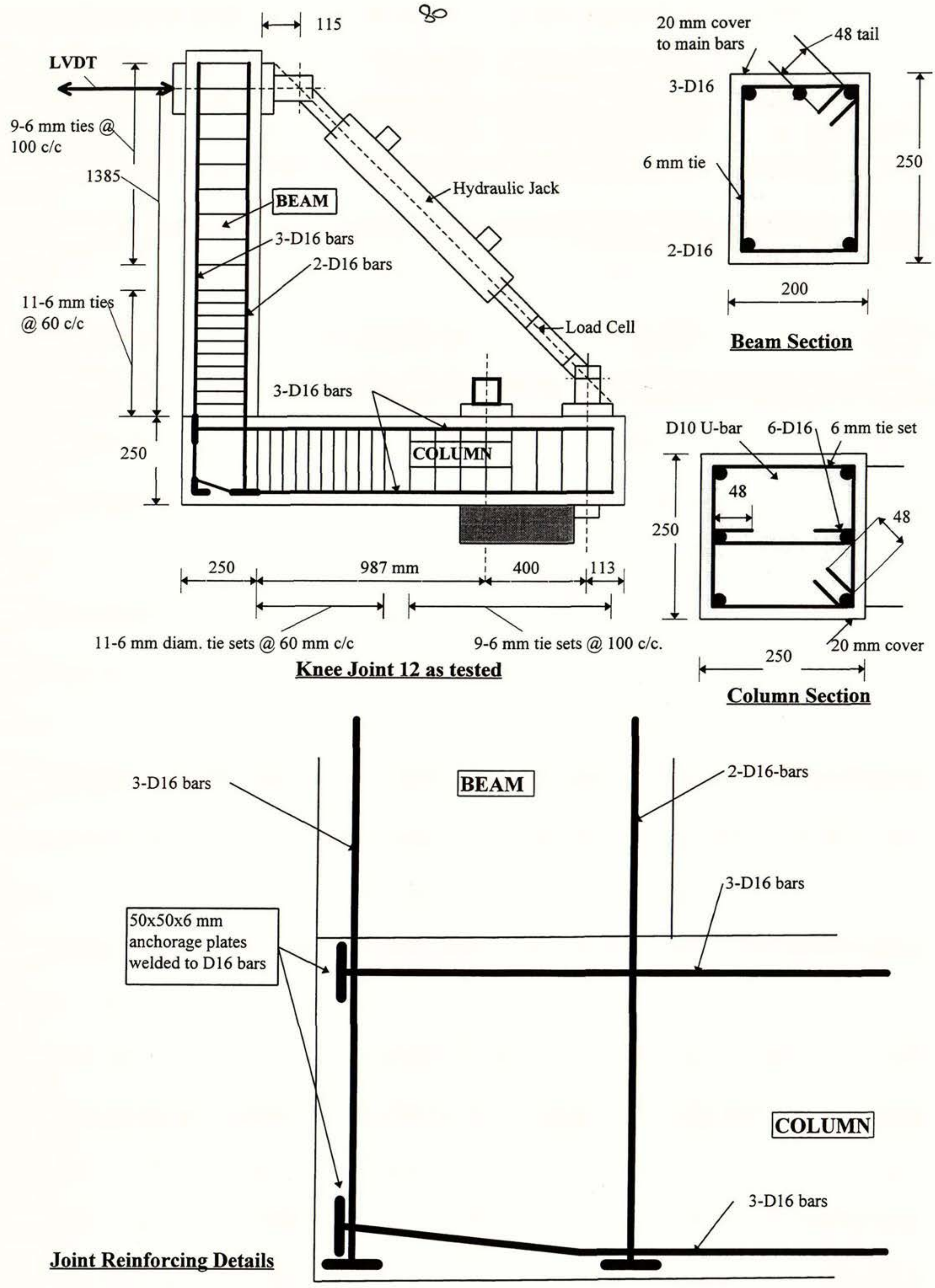


Figure 47: Knee Joint 12 with anchorage plates and no joint shear reinforcing.

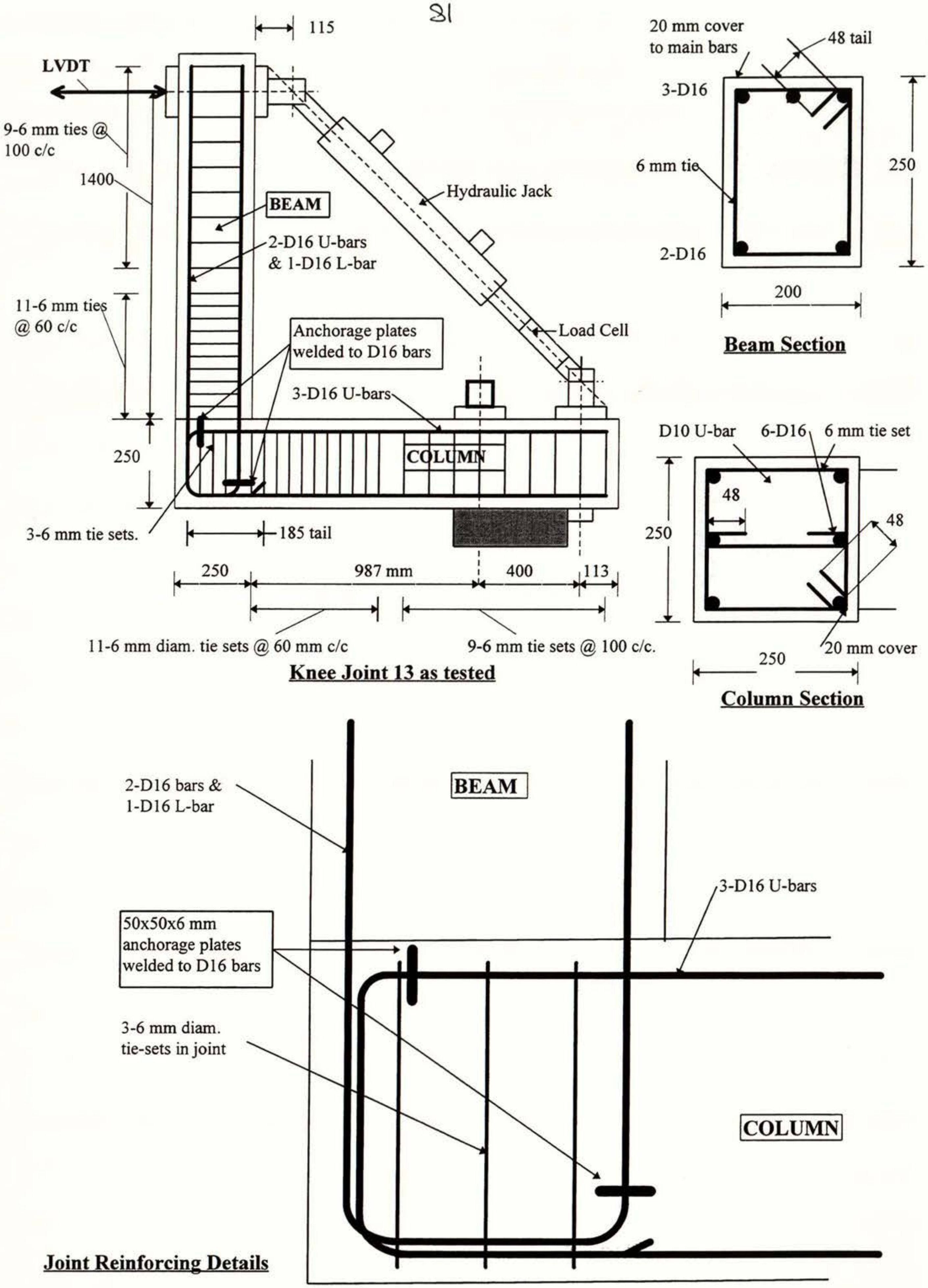


Figure 48: Knee Joint 13 with an anchorage plate on each U-bar. 3 tranverse joint ties.

TABLE 3: Material Properties of knee joints 11, 12 and 13 with anchorage plates.

Knee Joint	f_c' (MPa) Concrete	Bar type, diameter (mm)	f_y (MPa) Main bars	Bar type, diameter (mm)	f_{yh} (MPa) Joint Ties	Bar type, diameter (mm)	f_{yv} (MPa) Vertical Jt. U-bars
11	26.8	D16	333	6ϕ	322	D10	337
12	27.7	D16	333				
13	36.9	D16	320	6ϕ	322		

6. BEHAVIOUR OF KNEE JOINTS WITH ANCHORAGE PLATES

6.1 Knee Joint 11:

The applied force-beam displacement plot is shown in Figure 49, from which it can be seen that the nominal strength in both directions was not reached. The actual maximum values of M_{test}/M_n sustained were 0.84 and 0.90 under closing and opening moments, respectively. Shear failure in the joint occurred in the first cycle to ductility 2 in the closing direction.

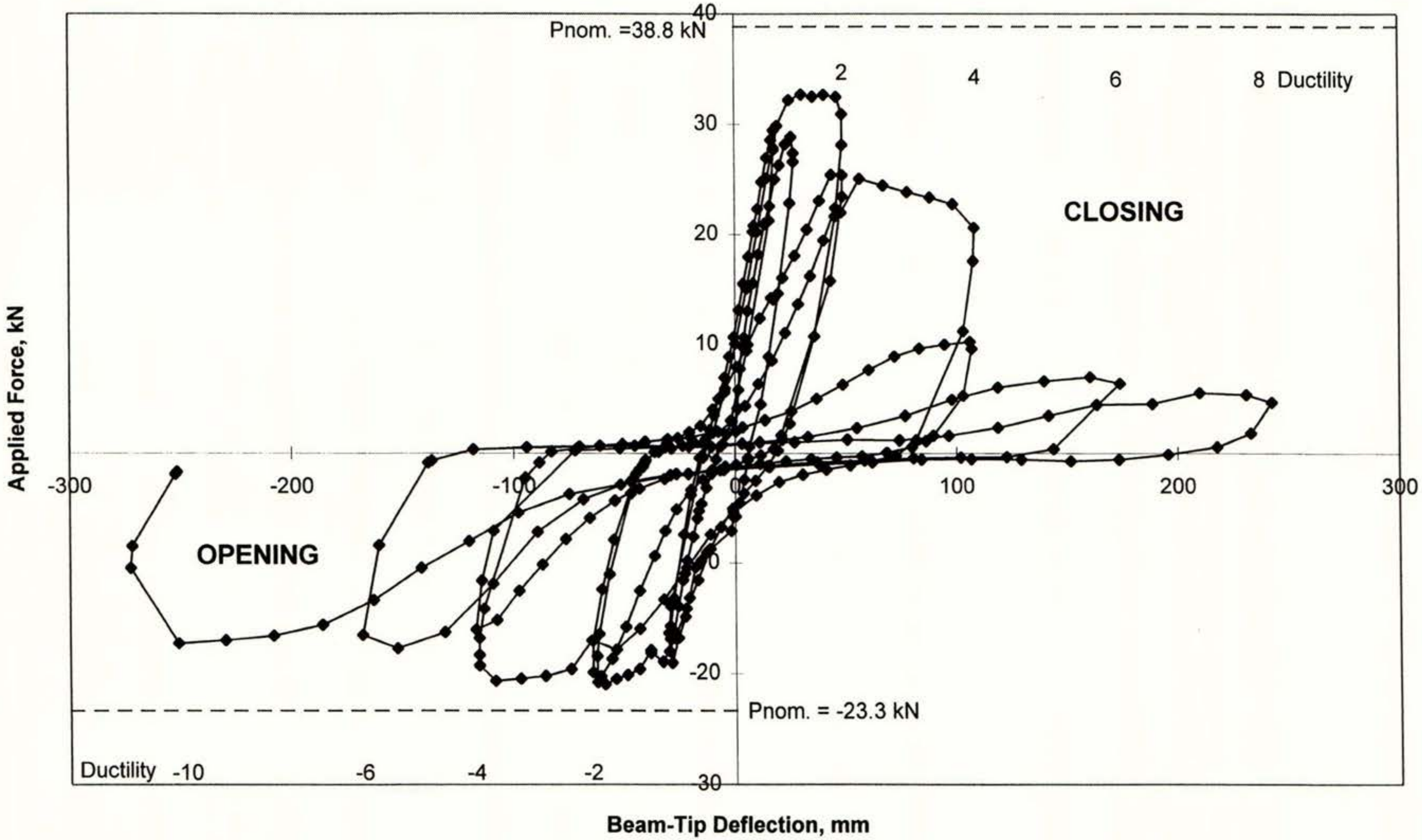
In the 'elastic' first cycle to closing $0.75M_n$ two diagonal joint cracks formed, one going from near the re-entrant corner to the outer corner. Beam and column flexural cracks also formed as expected. No diagonal joint cracks formed in the opposite direction in the opening cycles to $0.75M_n$.

During the first closing cycle to ductility 2 several more diagonal cracks formed with little extension of the beam hinge cracks, indicating a joint shear failure before any yielding had occurred in the hinge region.

While attempting to reach M_n and a ductility of -2 in the opening direction, two diagonal cracks formed across the joint (in the opposite direction to the first cracks), with the main crack being a splitting crack following the lines of the beam and column bars around the outer corner.

A moment of $0.90M_n$ was reached in the first cycle and was repeated in the second cycle to opening ductility -2, the joint cracks continued to open and it was obvious that some bar slip was occurring in the joint, as the column face crack widened but no beam bar yielding occurred. At closing

FIGURE 49: KNEE 11 Applied Force - Beam-Tip Deflection
Straight bars with anchorage plates.



ductility 2, for the second time, the moment attained had reduced to about $0.65M_n$, as the joint continued to disintegrate.

Spalling of the cover concrete (back and sides) occurred in the first closing cycle to ductility 4 and in the second cycle to the same ductility a substantial reduction in sustained moment occurred ($\cong 0.25M_n$), due to almost total loss of concrete around the top beam bars, including their anchorage plates.

In the opening direction the strength performance was better, with nearly $0.9M_n$ being reached in the first ductility 4 cycle, but reducing to about $0.70M_n$ in the second cycle.

The ductility ±6 cycles produced a further reduction in sustained strength in both directions with buckling of the top joint tie due to the lateral force being applied to the column outer bars by the anchorage plate on the top beam bars pulling out of the joint. Figure 50 shows the joint's condition at opening ductility -6, first cycle, noting extensive joint degradation.

In the final cycle to ductility ±8 there were again slight reductions in the strength sustained. The beam top and column outer bars were completely free of concrete through the joint during these cycles.

The behaviour of this joint would have been improved a little with the addition of the specified vertical U-bar through the middle of the joint, but it is doubtful whether M_n would have been reached in both directions. The joint exhibited a brittle nature with negligible ductile behaviour; such detailing would only be recommended for non-seismic conditions.

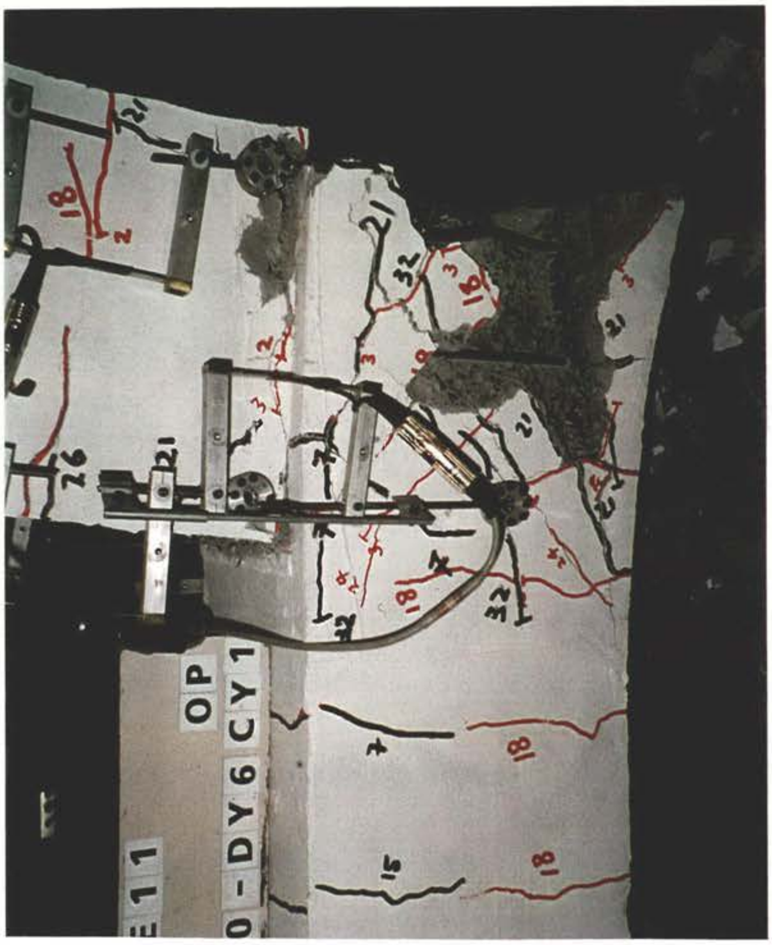


FIGURE 50: Knee 11 with anchorage plates at first cycle to opening ductility -6.

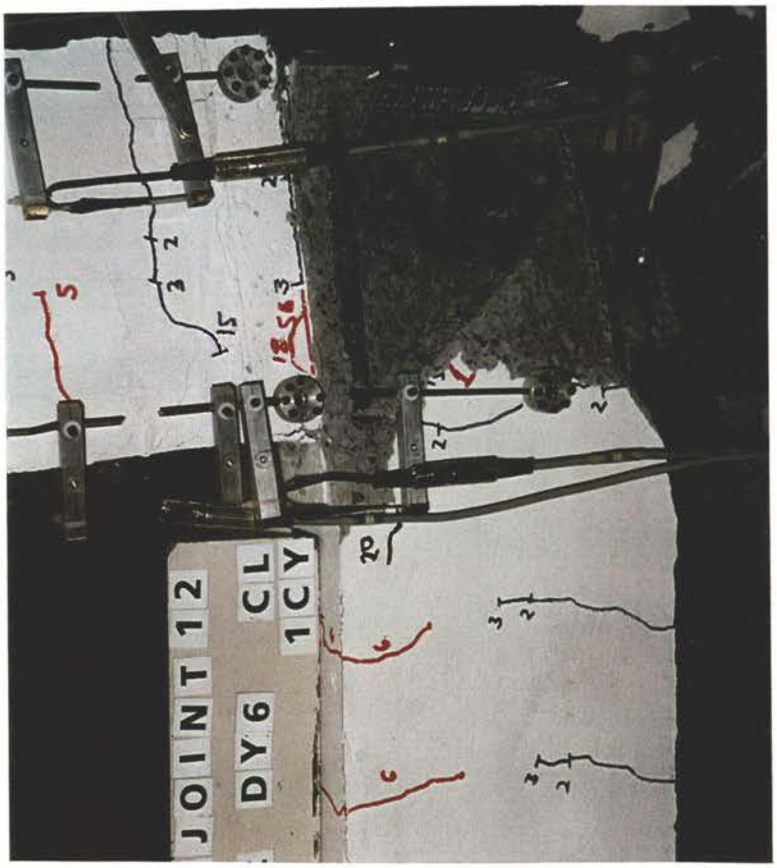
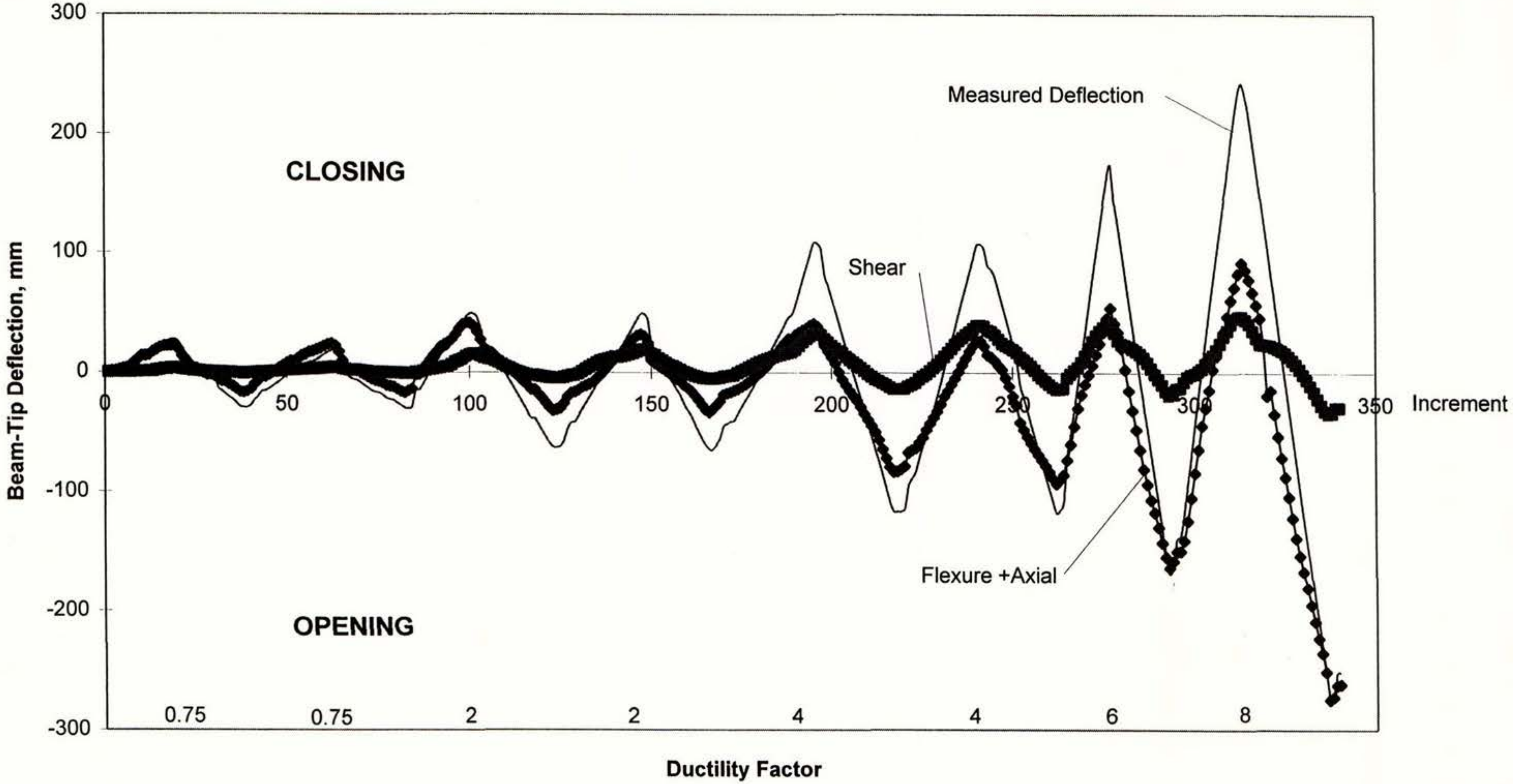


FIGURE 53: Knee 12 with anchorage plates at first cycle to closing ductility 6.

FIGURE 51: KNEE 11 Flexural+Axial & Shear Deformations and Measured Beam-Tip Deflections



The joint shear deformations were almost equal to the flexural deformations in this knee joint, as shown in the accumulated flexural and shear deflections calculated at the beam-tip in Figure 51.

6.2 Knee Joint 12:

The maximum flexural strength of knee 12 with no joint shear ties was a little better than knee 11, in that the closing and opening M_{test}/M_n ratios reached were 0.96 and 0.95, respectively. These strengths were reached in the first cycle to ductility ±2. However in the following repeat cycle to ±2 ductility there was a substantial decrease in closing moment (to ~$0.60M_n$), while the opening strength reduced slightly to about $0.85M_n$. In the following closing cycles the moment sustained progressively decreased till from the second cycle to ductility 4 onwards the moment reached was only approximately $0.15M_n$. The opening strength behaviour was better but with a progressive decrease in moment carried till in the final cycle to ductility -9 the moment had reduced to about $0.50M_n$. The force-displacement loops are shown in Figure 52. There was no joint cracking till during the first cycle to ductility 2 in the closing direction. In that cycle three diagonal cracks formed, two extending almost from the re-entrant corner to the outer corner. Numerous small cracks also appeared on the top of the joint. Opposing diagonal cracks formed in the opening cycle to ductility -2.

Joint destruction occurred in the ductility ±4 cycles with side and top cover spalling from the outer corner. The anchorage plates on the outer column bars initiated this because there were no ties to restrain the bars after the cover had spalled. The diagonal joint cracks were 2-3 mm wide and there was negligible flexural crack extension along the beam hinge or column. The joint cracks seemed to extend through the full joint width. The lack of any transverse or vertical joint ties meant a premature diagonal tension failure occurred in this joint. The whole outer half of the joint was attempting to break off and the concrete continued to disintegrate during every subsequent cycle. By the end of the ductility 4 cycles the outer beam and column bars were totally exposed and obvious bar slip was occurring under closing moments. The opening performance was better than in the closing direction because the inner column bars and bottom beam bars were still reasonably anchored in the concrete, allowing stresses near yield strength to be attained.

As the cycles progressed the joint shear deflection continued to increase, especially under closing moments. From the ductility 4 cycles on the opening joint shear deformation accounted for about half of the total beam-tip deflection. This can be seen in the obvious joint shear deformation in Figure 53 taken at ductility 6, first cycle, closing. Figure 54 shows the calculated shear and flexural+axial deformations together with the measured beam-tip deflection throughout the test. At the test's end any closing resistance was due solely to the anchorage plate on the top beam bars pulling the column outer bars inwards (the beam plate was resting on the side of the column bar).

From this test it was apparent that some transverse joint ties and vertical shear reinforcing are required in such joints, even though it was expected that the anchorage plates would improve the behaviour of the diagonal joint concrete strut. The transverse ties also confine the joint core concrete, reducing the tendency for the joint to dilate during reversing load cycles.

6.3 Knee Joint 13:

The flexural strength behaviour of this joint with U-bars and anchorage plates was considerably better than both joints 11 and 12. The maximum M_{test}/M_n values reached were 1.07 and 1.20 under closing under closing and opening moments, respectively. The closing nominal moment was

FIGURE 52: KNEE 12 Applied Force - Beam-Tip Deflection
Straight bars with anchorage plates & no tranverse joint ties.

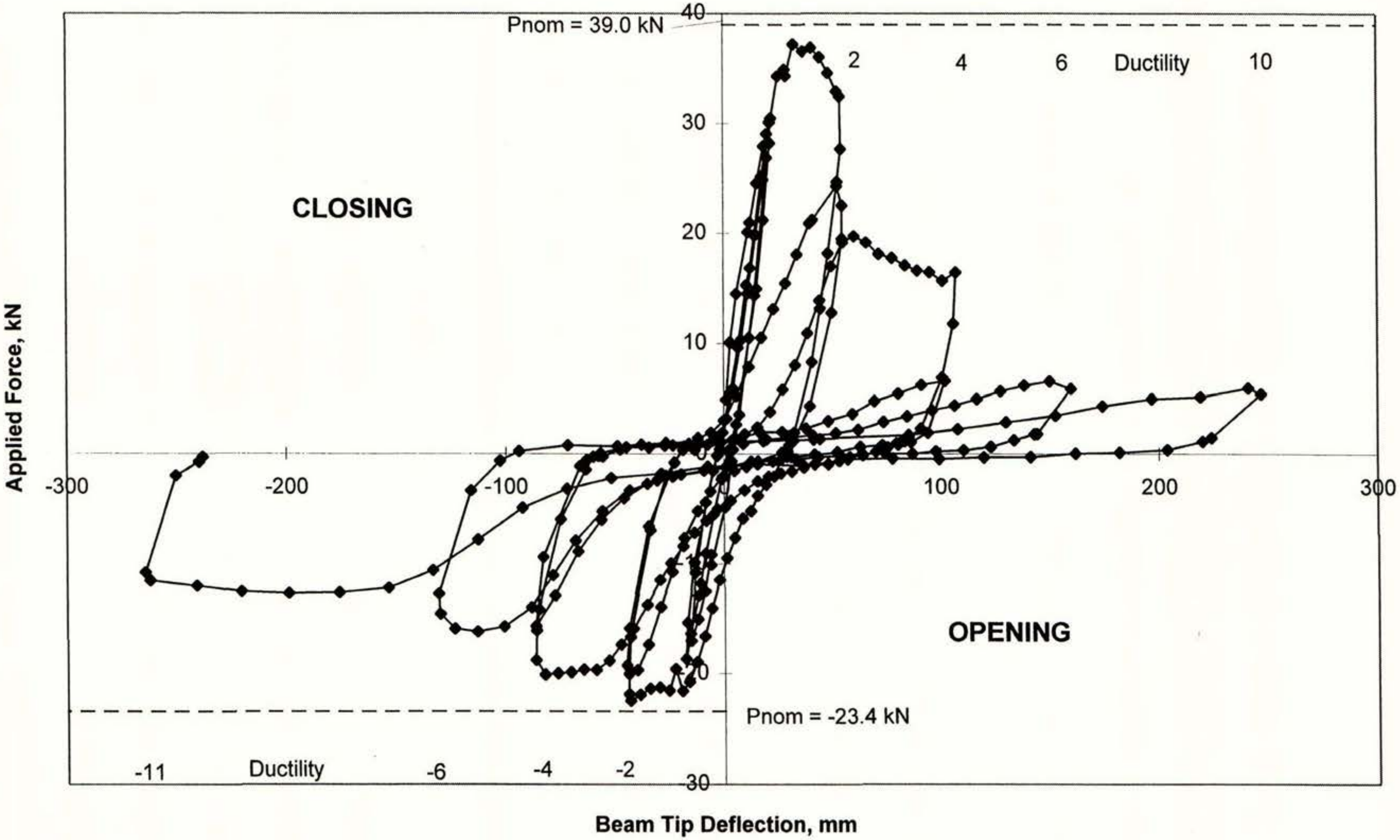
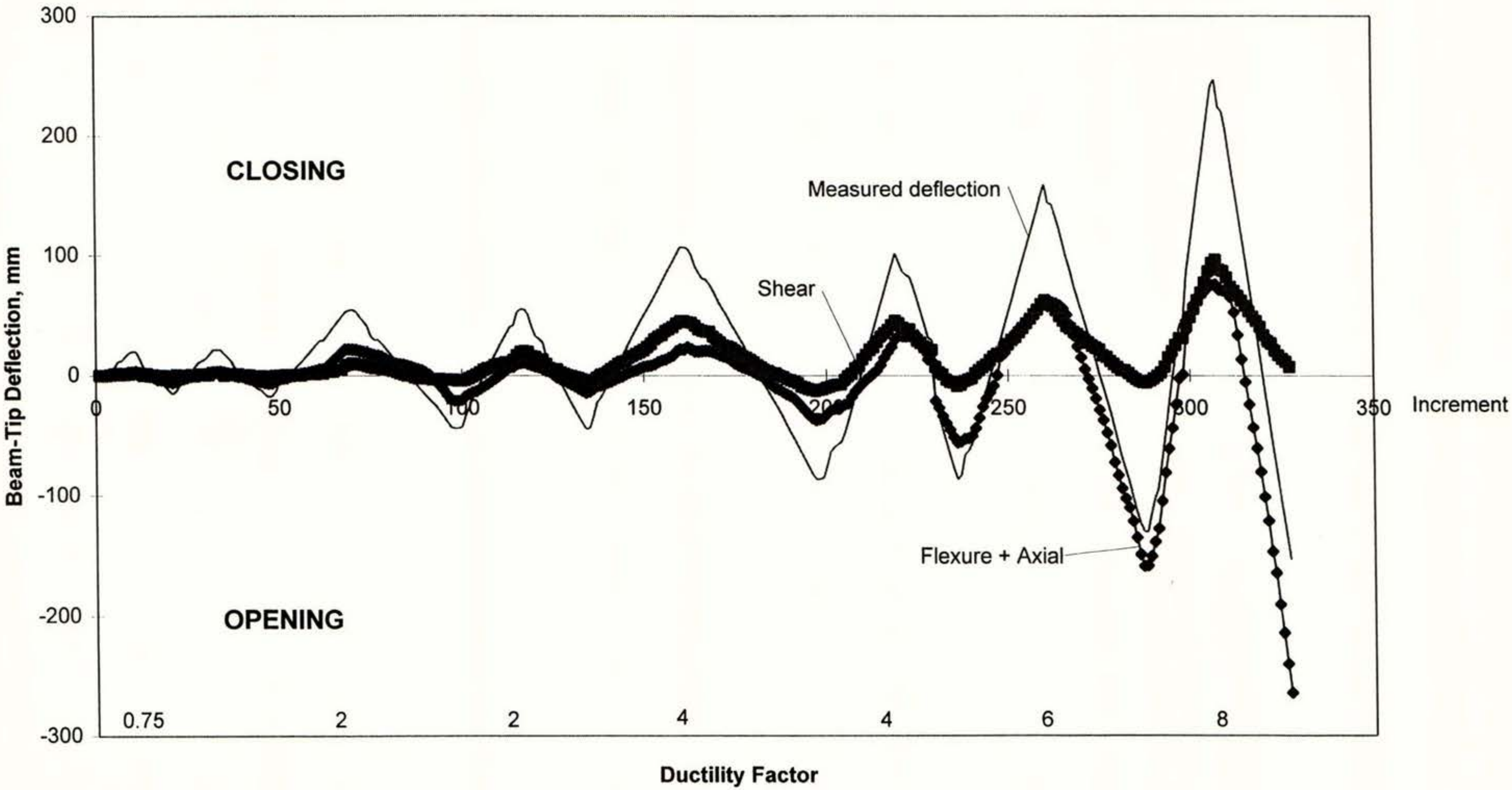


FIGURE 54: KNEE 12 Flexure +Axial & Shear Deformations and Measured Beam-Tip Deflections



exceeded in the first cycles to ductility 2 and 4, while the opening M_n was exceeded in all cycles at ductility -2, -4, -6 and -8. It was only in the closing cycles to ductilities 4 (second cycle), 6 and 8 that the strength reduced to about 70% of M_n and a distinct loss of stiffness occurred at small ductilities in the later cycles. This performance is shown in Figure 55; the applied force versus beam-tip deflection loops for knee 13.

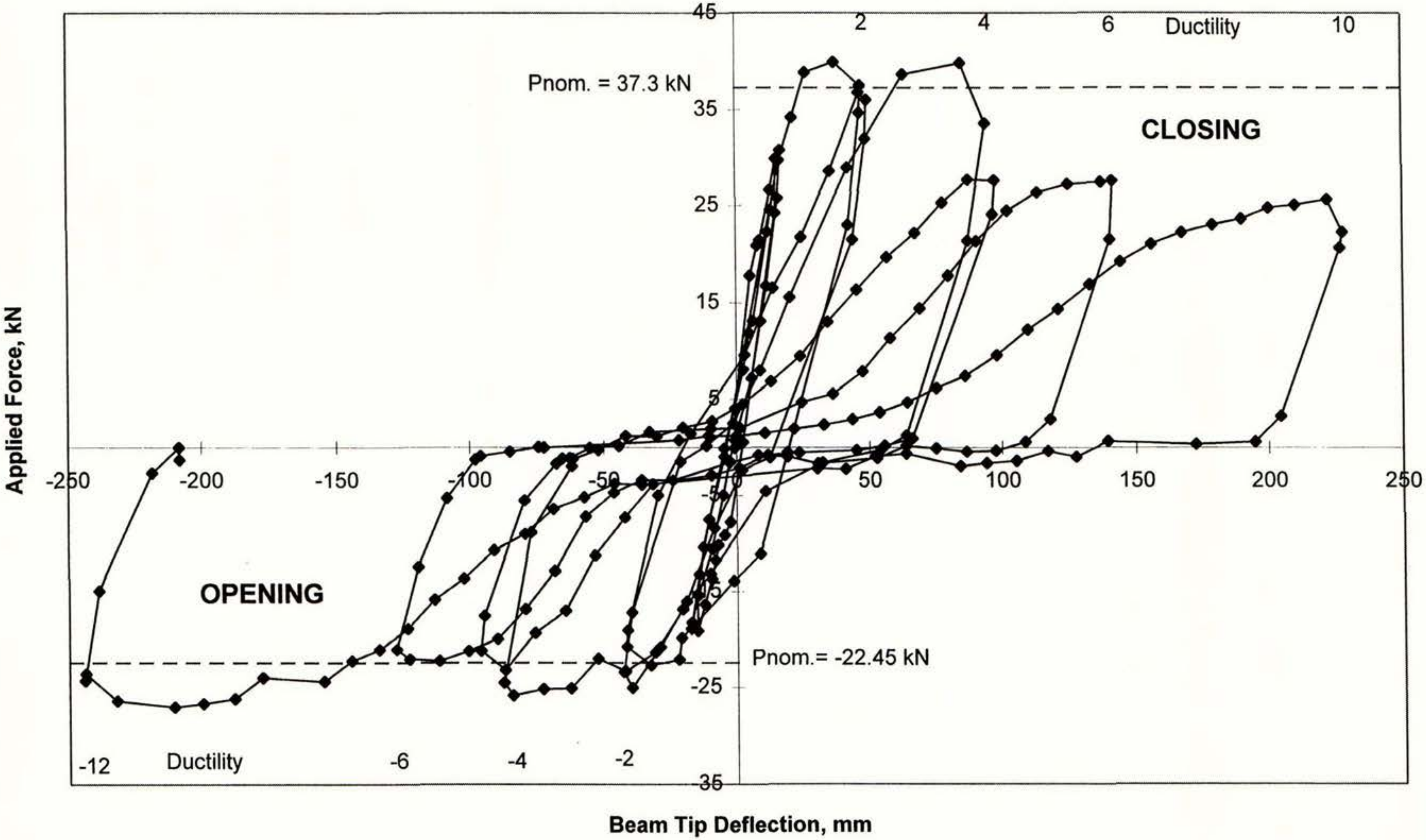
Initial cracking occurred in the beam and column but two small cracks formed across the top of the joint near the inner column face in the first cycle to $0.75M_n$ in the closing direction. During the second cycle to $0.75M_n$ cracks formed from the inner and outer column faces approximately along the line of the inner column bar within the joint, but there were no diagonal joint cracks. The beam and column cracks continued to lengthen across the respective sections.

Splitting cracks around the outer column bars occurred in the joint during the first opening cycle to ductility -2. One of these cracks extended across the top of the joint at about the column centreline position. The splitting cracks extended around the column bar's 90-degree bend during the second closing cycle to ductility 2.

During the ductility ±2 cycles yielding was occurring in the beam hinge zone over a length of about 100 mm out from the column face. The second cycle in each direction showed good repeatability in strength sustained, although there was some reduction in stiffness at low applied force levels.

The first diagonal joint crack appeared during the first closing cycle to ductility 4, while the outer splitting cracks were now 2 mm wide and the outer corner was threatening to spall off. The column face crack was about 6 mm wide at the top of the joint, showing that some slip of the beam bars was then occurring. Three beam cracks were opening but they had not widened beyond that reached in

FIGURE 55: KNEE 13 Applied Force - Beam-Tip Deflection
Main reinforcing U-bars with one anchorage plate each.



the previous cycle to ductility 2. However a new beam hinge crack formed 50 mm out from the column face (about 1 mm wide at the outer face).

In the first opening ductility -4 cycle the back and top joint cover was detached and an opposing diagonal joint crack formed. This crack was very fine (less than 0.5 mm wide). The strength attained dropped to about 75% of M_n in the second closing cycle to ductility 4, as the and back joint cover fell off and crushing of the beam's inner cover concrete occurred, reducing the lever arm considerably. Also there was considerable anchorage loss around the U-beam bars resulting in slipping and reduction of the maximum strength able to be reached by the ductility 4 displacement, ≅92 mm. The beam reinforcing was strain-hardening at this point, compensating for the decrease in lever arm caused by the spalling of the top joint cover. Figure 56 shows a photograph at this point in the testing (second cycle to opening ductility -4). The moment reached in the second opening cycle to ductility -4 was about 85% of that reached in the first cycle. The joint's back cover continued to spall to a position level with the bottom of the beam. The anchorage plate on the beam bars appeared to be anchoring the bars well, at least on the inside of the bar where little damage to the core concrete was occurring.

The single cycle in both directions to ductility ±6 displacement resulted in little further damage to the joint and the same strengths were reached as in the previous cycle (ductility ±4) but there was a stiffness loss at low load levels, due to some bar slip.

The reversed cycle to ductility ±10 produced very similar behaviour, with the same strength ($0.67M_n$) being sustained in the closing direction and the test's maximum opening strength of $1.20M_n$ being reached in this cycle. This was due to strain-hardening of the bottom beam bars and

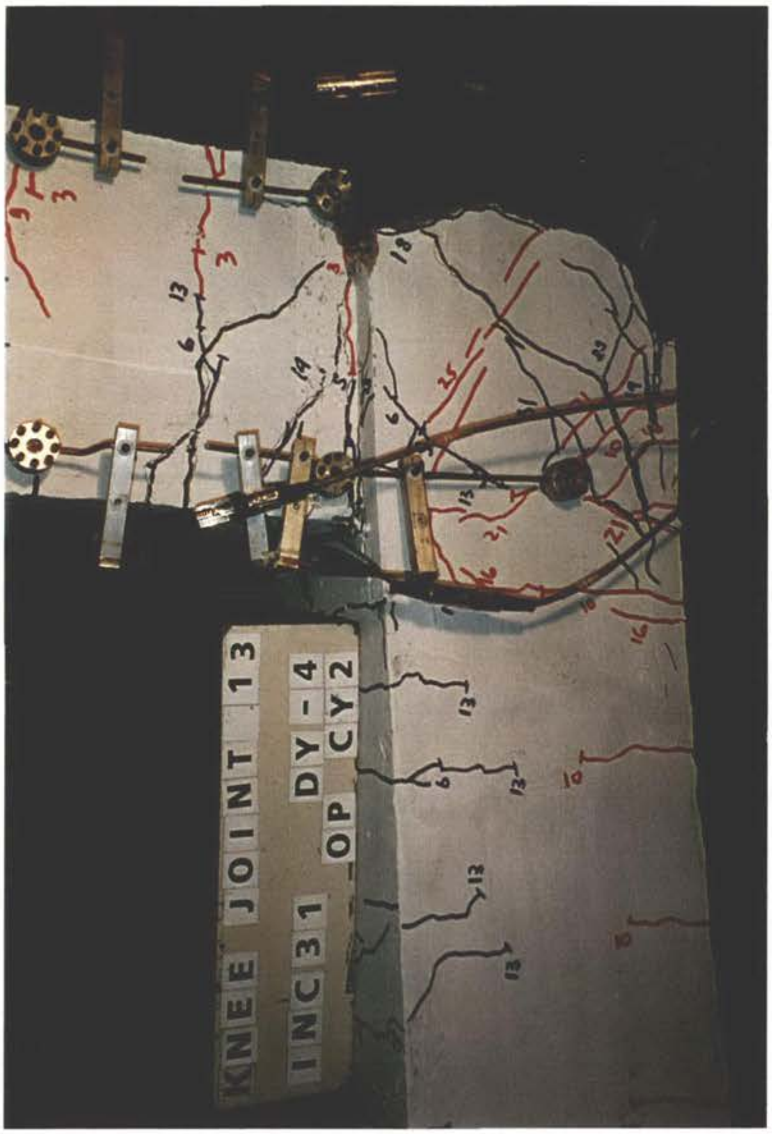
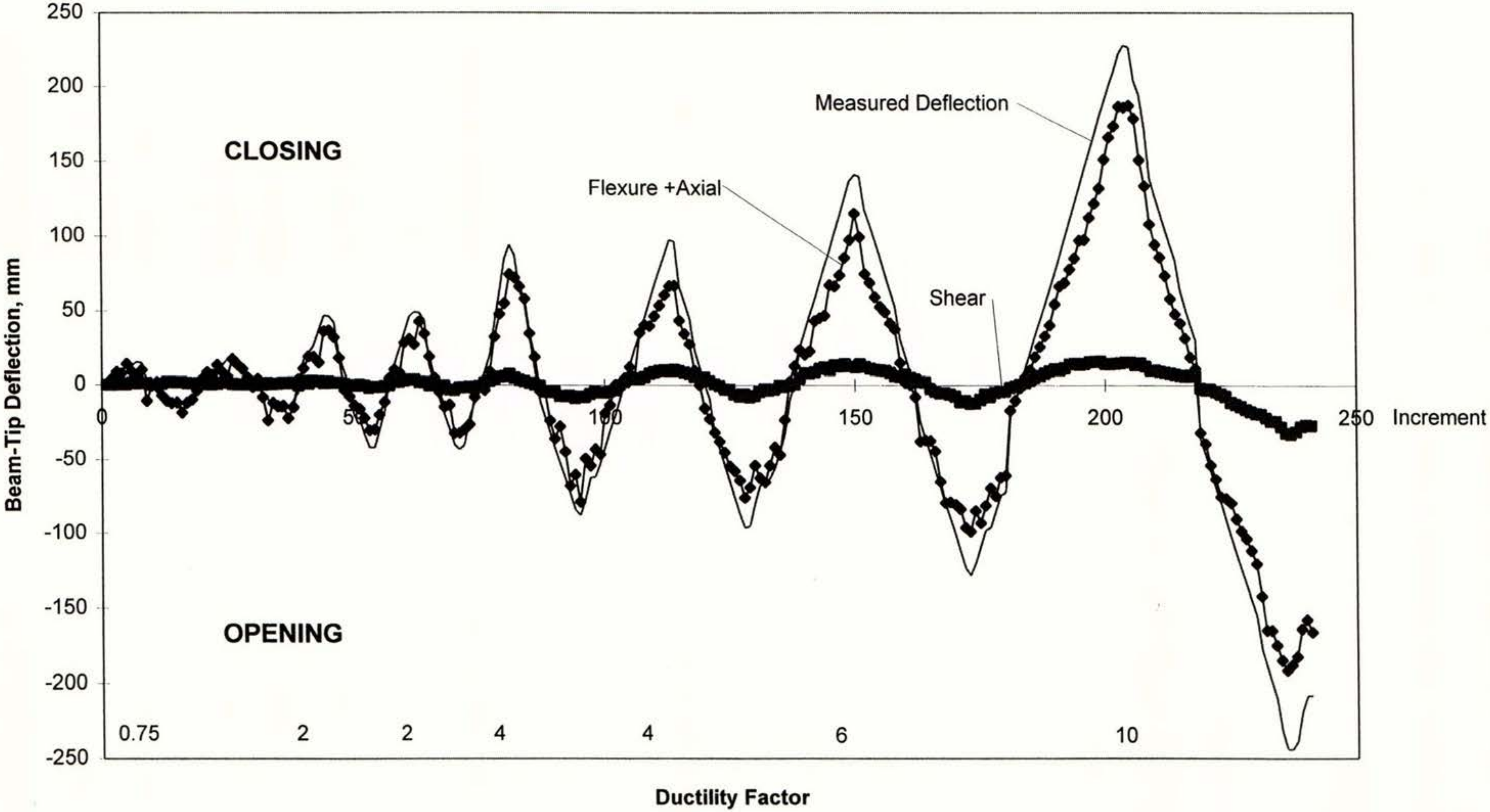


FIGURE 56: Knee 13, U-bars & anchorage plates, at second cycle to opening ductility -4.

Chart20

FIGURE 57: KNEE 13 Flexure +Axial & Shear deformations and Measured Beam-Tip Deflections



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good anchorage of these bars in the lower joint zone, which was unaffected by the top and back joint cover spalling. Several more diagonal joint cracks formed during this extreme ductility cycle.

Knee joint 13 performed very well, especially under opening moments and it was only the loss of cover on the top and back of the joint which resulted in a decrease of 30% in the sustained moment in the closing direction during the second cycle to ductility 4. The joint core remained secure with only minor joint cracking and no loss of the joint side cover, with shear deformations amounting to about 10% of the total flexural and shear deformations. This is shown in Figure 57, which is the calculated summation of the flexural plus axial and shear deformations together with the measured beam-tip deflections throughout the test.

7. DISCUSSION

7.1 Joint Shear Stresses:

Figure 58 shows the opening and closing maximum joint shear stresses, v_{jh} plotted against $\sqrt{f_c^{'}}$ for knee joints 1 to 13, together with the other small knee joint tests found in the literature. Also drawn are the lines $v_{jh} = 0.37\sqrt{f_c^{'}}$ and $0.5\sqrt{f_c^{'}}$ (MPa), which represent the maximum joint shear stress for unreinforced joints and the approximate average joint shear stress for current New Zealand designs, respectively.

The joint shear stress limit specified in NZS3101:1995 (13) is $0.2f_c^{'}$, which is approximately equivalent to $1.10\sqrt{f_c^{'}}$ (MPa) for the concrete strength used in these tests (30 MPa). Therefore it seems that the Standard's limit is unlikely to be reached in small knee joints, even with the Standard's maximum amount of main reinforcement fitted.

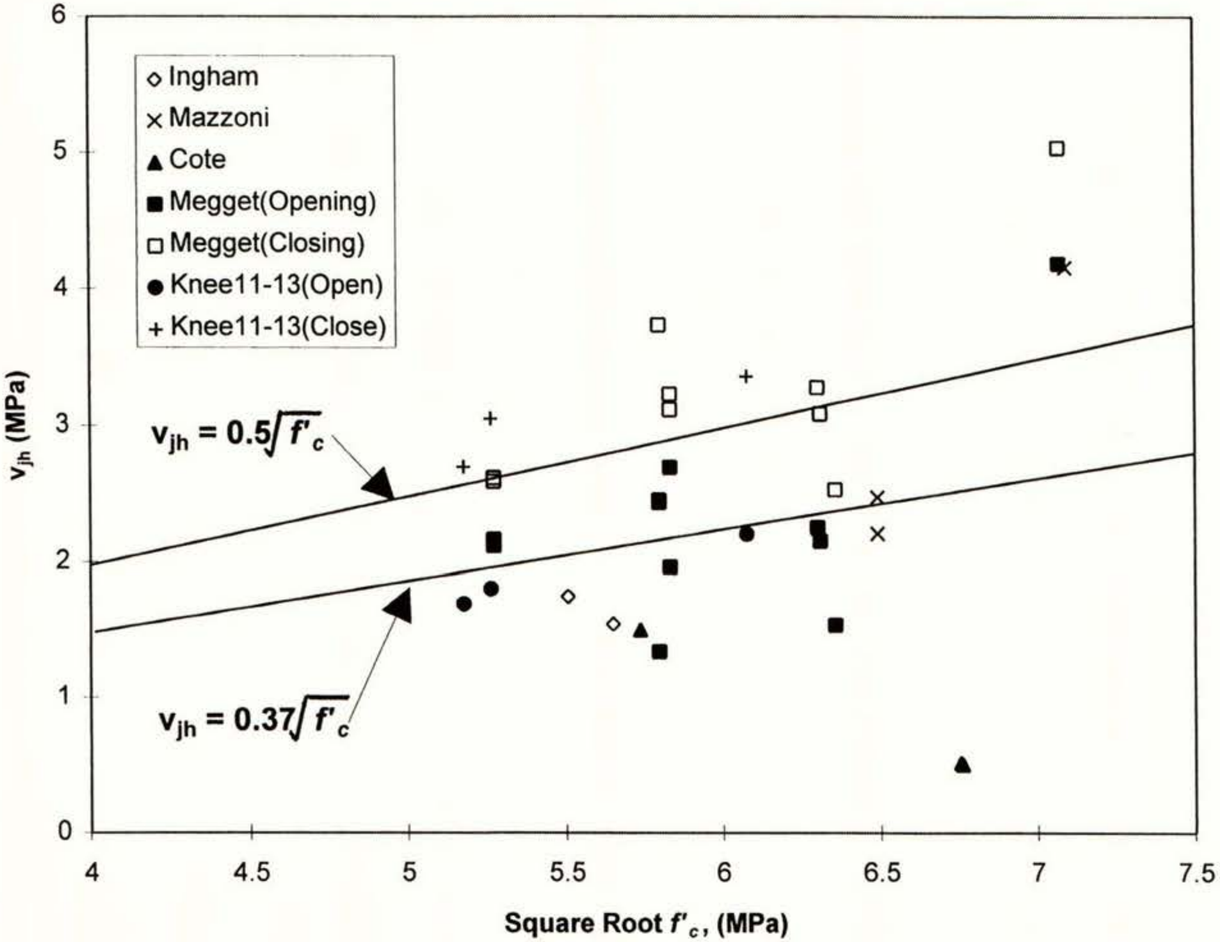

Figure 58: Maximum Joint Shear Stress versus Square Root of Concrete Compressive Stress for all cyclic loaded beam-column knee joint tests.

8. CONCLUSIONS

1. The seismic design requirements for an exterior joint's transverse shear reinforcement in the NZ Concrete Standard (NZS3101:1995) gave satisfactory joint shear behaviour for small reinforced concrete knee joints under cyclic loading. A joint shear failure resulted when the amounts of transverse joint reinforcing were reduced below those specified. In some tests the specified amount of vertical joint shear steel seemed to be the minimum required for shear failures not to occur. It may be advisable to increase the amount of vertical tranverse reinforcing in small knee joints, if excellent ductile behaviour is required.

2. The anchorage of main beam and column bars in joints using continuous U-bars produced better cyclic behaviour (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.

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

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

5. The maximum joint shear stresses sustained were about $0.1f_c'$ for small knee joints designed to the 1995 Concrete Standard. This is half of the specified maximum in the Standard. It is virtually impracticable to design for $0.2f_c^{'}$ joint shear stresses in small exterior joints, due to the limit on principal beam bar reinforcement ratios, $p_{max} = \dfrac{f_c^{'} + 10}{6f_y}$, but p_{max} not to be greater than 0.025.

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 became the predominent component of the joint's rotation.

7. Joints with anchorage plates on straight main bars did not reach the nominal strength in either direction and continued to degrade at higher beam deformations. The loss of the top and back joint cover caused anchorage failure and bar slip at low force levels to occur.

8. A joint with U-bars and one anchorage plate welded near the 90-degree bend performed much better than the straight bars with end plate; the nominal strength being exceeded in both directions up to displacement ductilities of ±4.

9. Knee joints designed to the 1960's Code of Practice (14) failed to reach their nominal strength in both opening and closing directions and the strength sustained at ductility ±2 displacements were less than half of the corresponding nominal moment. The maximum closing joint shear stresses sustained were about $0.4\sqrt{f_c^{'}}$ MPa for these unreinforced joints (no horizontal or vertical transverse ties). The corresponding maximum stress for opening moments was $0.24\sqrt{f_c^{'}}$ MPa , which approximately agrees with Priestley's (1996) recommendation for unreinforced joints with beam bars bent *away* from the joint.

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APPENDIX 1

A1.1 KNEE JOINT 14 DESIGN:

This extra knee joint was added to the testing programme in an attempt to improve the closing moment behaviour by adding two D12 diagonal bars across the tension diagonal and anchoring them next to the outer beam and column bars. Figure A1 shows the reinforcing details of knee 14. The extra 2-D16 transverse bars within the 90-degree bends in some of the earlier units were not fitted to knee 14. Otherwise this knee joint was similar to knee 9, with 3-6ϕ horizontal tie-sets and 1-D10 U-bar vertically within the joint.

The beam and column main bar (D16) yield stress was 325 MPa, while the concrete compression cylinder stress, f_c' was 32.4 MPa.

A1.2 RESULTS:

Figure A2 is the applied force versus the beam-tip deflection, which shows 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.

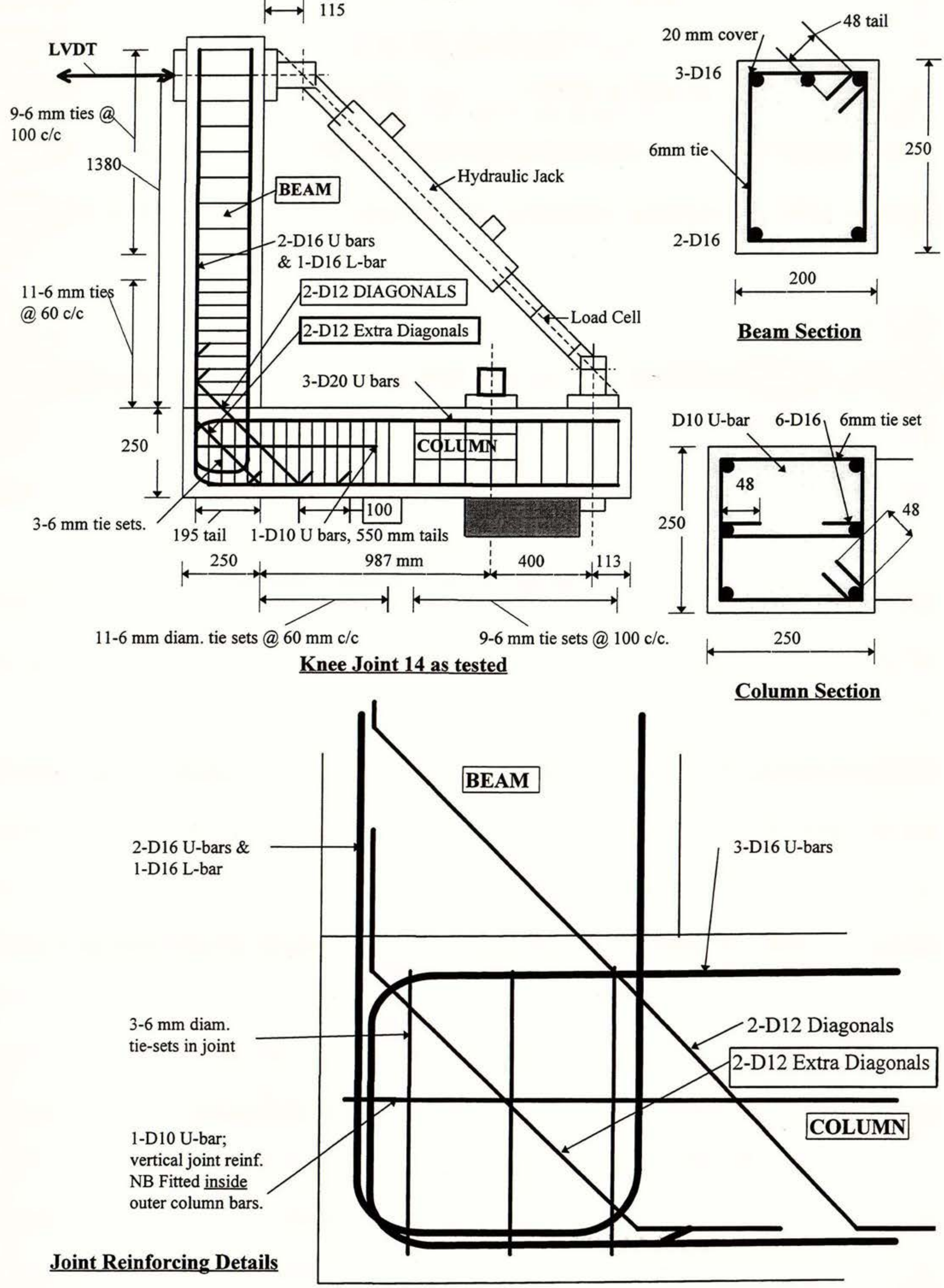

Figure A1: Knee Joint 9 designed to 1995 Standard plus extra diagonal bars through joint.

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 centreline of the column bars around the 90-degree bend.

Figure A3 is a photo of the joint at the first closing cycle to ductility 6, with the outer corner loose and the outer beam cover near the column face spalling. Figure A4 is at the first opening cycle to ductility -8 when all the top and back joint cover had fallen off and the inner beam hinge cover had crushed and then spalled.

In Figure A5 the addition of the flexural plus axial deformations along with the beam and joint shear deformations are plotted with the measured beam-tip deflections throughout the test. It can be seen that the shear deformations are negligible, as confirmed by the photographs (Figs. A3 and A4). There was excellent agreement between the summation of the flexural+axial and shear

FIGURE A2: KNEE 14 Applied Force - Beam Tip Deflection

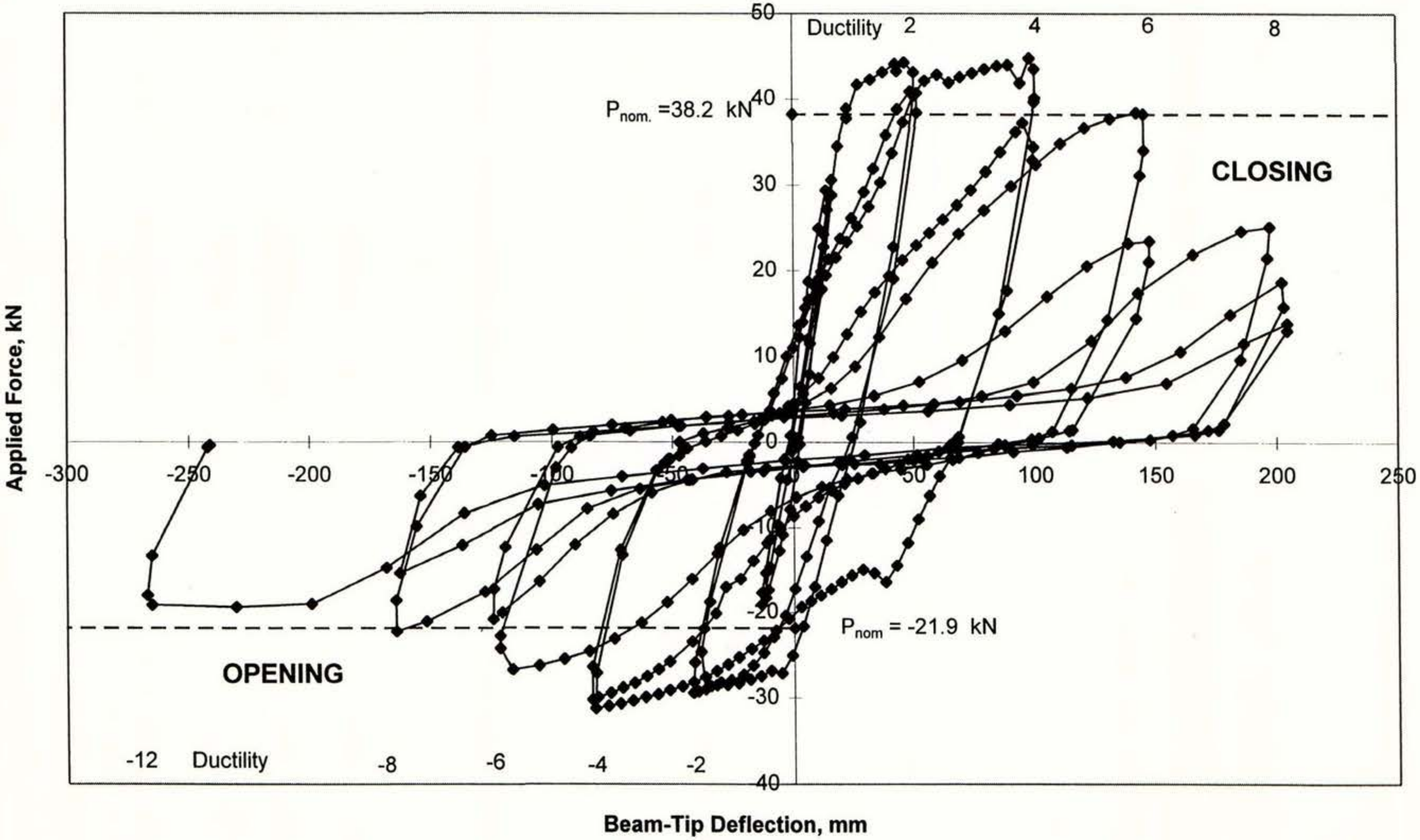

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FIGURE A3: Knee 14 at first cycle to closing ductility 6.

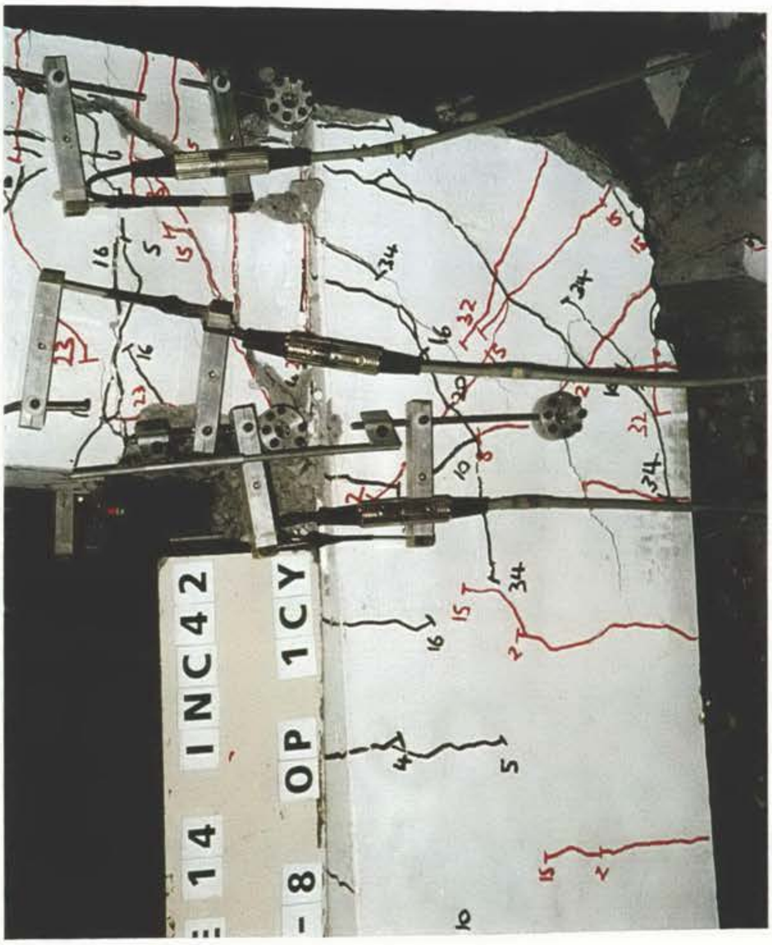


FIGURE A4: Knee 14 at first cycle to opening ductility -8.

deformations and the beam-tip measurements. Figure A6 is the applied force versus beam hinge deflection calculated as deflection at the beam-tip and shows the good hysteretic energy absorption in the cycles up to and including ductility 4.

FIGURE A5: KNEE 14 Flexure+ Axial & Shear deformations and Measured Beam-Tip deflections

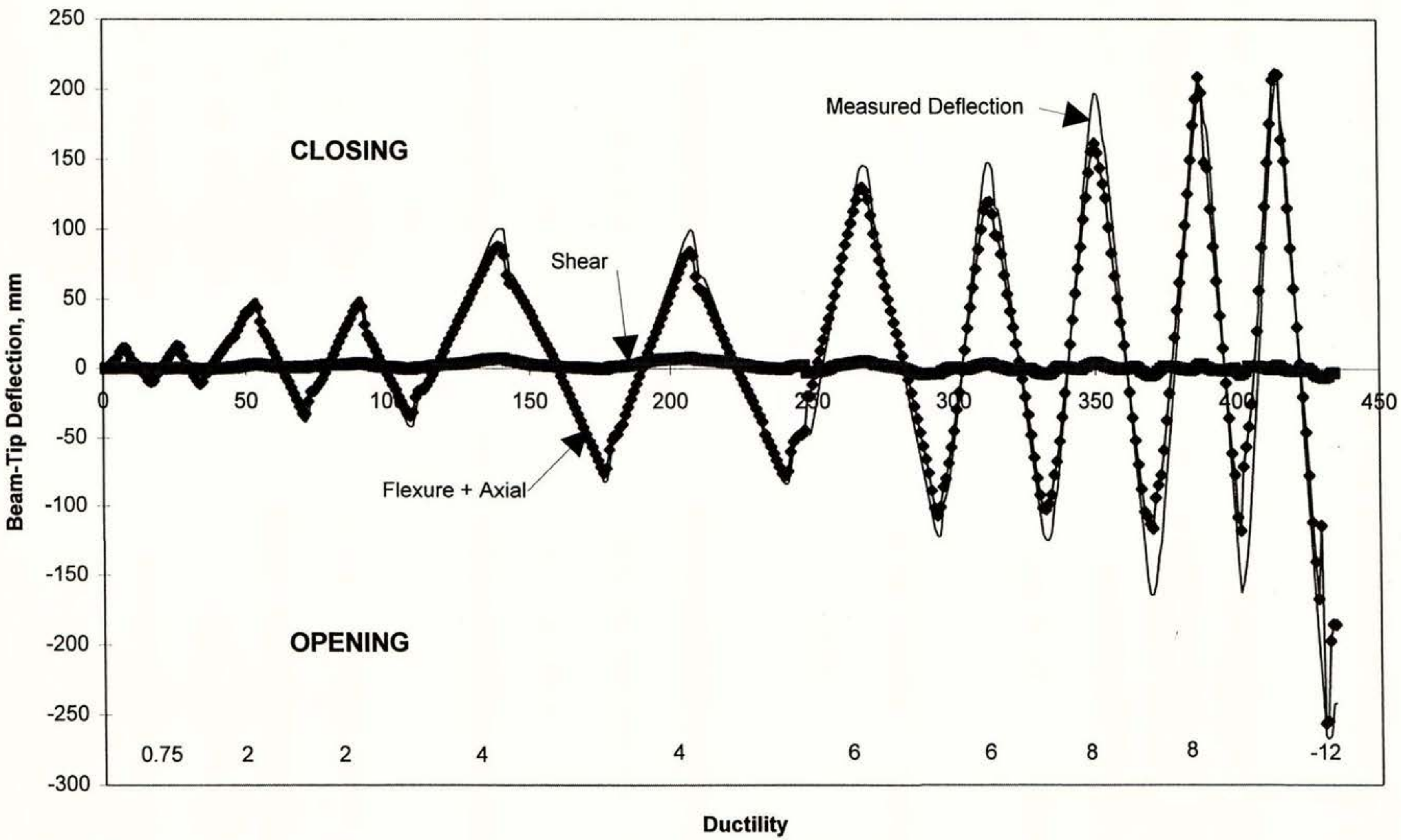
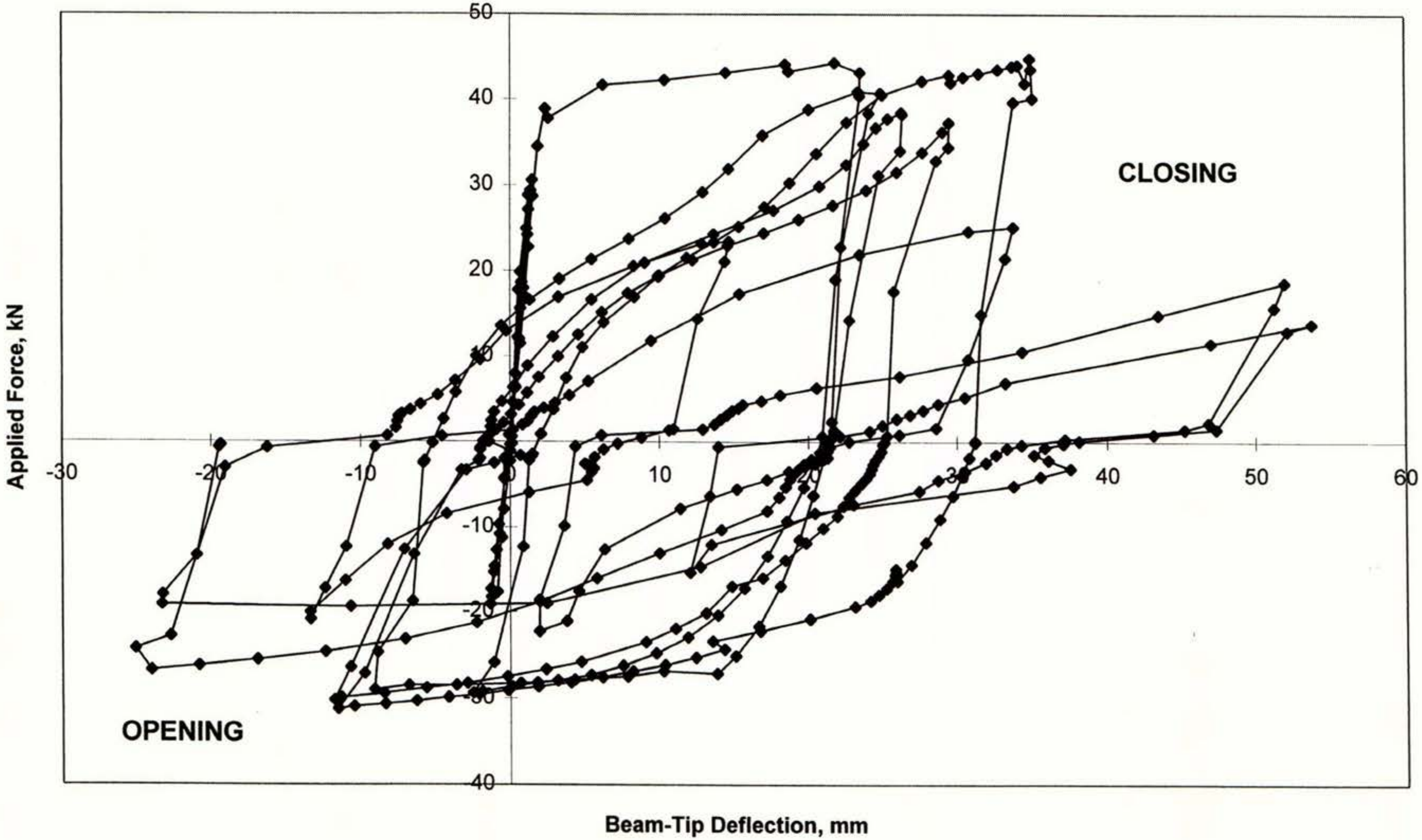


FIGURE A6: KNEE 14 Beam Hinge zone Applied Load - Beam-Tip Deflection



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