Appendix A

Beam-Column Joint Database

The Beam-Column Joint Database is a collation of the results of tests performed on bridge beam-column joint specimens relevant to the purposes of this study. Many past tests have examined the behavior of joints as found in typical buildings but relatively few tests have been performed on joints with typical bridge geometry and loading conditions. The database was created to quantify their behavior and condense the lengthy reports into more manageable pieces. Geometric properties, loading protocol, and test results, both quantitative and qualitative, were of primary interest.

The records included within are from tests performed by California researchers for previous Caltrans-sponsored research projects. Several pertinent tests were not included due to a lack of shear stress-strain data needed for the purposes of this project. See Table A.1 for a list of both the included records and those investigations not considered here. The tests by Stojadinovic and Thewalt [Stojadinovic, 1995] and Mazzoni et al [Mazzoni, 1991] were not included because joint shear stress-strain data was not provided in the reports. Also, the specimens in the latter test were not as representative of bridge geometry as required for this project. The tests by Lowes et al [Lowes, 1995] were not included due to the extremely poor behavior of the as-built and the types of retrofit strategies employed.

Included after the summary table is a summary of the geometry of each of the specimens in the database, followed by a summary of the hysteresis plots. These summaries are included to aid the reader in finding a particular test either by hysteresis performance or by specimen geometry (e.g.

reinforcing layout or characteristics, tee or knee geometry). The ten database records follow the summaries.

Summary of Joint Tests

***Caltrans* Joint Local Deformations Project**

Designation	Type	Column	Researcher	Brief Description
Knee-Ing-RCT7	Knee	Rect.	Ingham, et al - UCSD - 1994	Redesign and test of an old non-ductile joint design
Knee-Ing-RND-4	Knee	Round	Ingham, et al - UCSD - 1994	Rebuild of non-ductile specimen: remove joint concrete, add haunch, ductile joint detailing
Knee-Ing-RND-6	Knee	Round		Ductile retrofit with R/C jacket and rounded beam stub on back of joint
DD-Maz-T-1	Dbl. Deck	Round	Mazzoni - UCB - 1997	Transverse loading of ductile double-deck bridge subassembly
DD-Maz-L-1	Dbl. Deck	Round		Longitudinal response of above specimen
DD-Maz-T-2	Dbl. Deck	Round		Transverse loading of ductile double-deck bridge subassembly with larger demands
DD-Max-L-2	Dbl. Deck	Round		Longitudinal response of above specimen
Tee-Nai-RD-A1	Tee	Round	Naito, et al. - UCB -1999	Baseline, designed to test current CalTrans specifications
Tee-Nai-RD-A2	Tee	Round		Headed reinforcement used instead of traditional joint detailing
Tee-Sri-RD-IC1	Tee	Round	Sritharan, et al. - UCSD - 1994	Baseline test of R/C cap beam in an investigation of P/S cap beams
Knee-Sri-RD-1	Knee	Round	Sritharan, et al. - UCSD - 1997	Exterior Knee joint of multi-column bent
Tee-Sri-RD-2	Tee	Round		Interior Tee joint of multi-column bent
Not considered in this Study:				
	Tee	*Rect.*	*Lowes & Moehle - UCB - 1995*	*One non-ductile(as-built) test, one mild steel and one prestressed retrofit*
	Knee	*Rect*	*Mazzoni, et al. - UCB - 1991*	*Two tests on knee joints designed per ACI 318-83 and 352-85; Retrofit of specimens*
	Knee	*Rect*	*Stojadinovic & Thewalt - UCB - 1995*	*Eight tests on bridge outrigger knee joint systems (as built, repair, retrofit, and new designs*

Table A.1 Beam-Column Database Records

SUMMARY OF JOINT GEOMETRIES

Caltrans Local Deformations Project

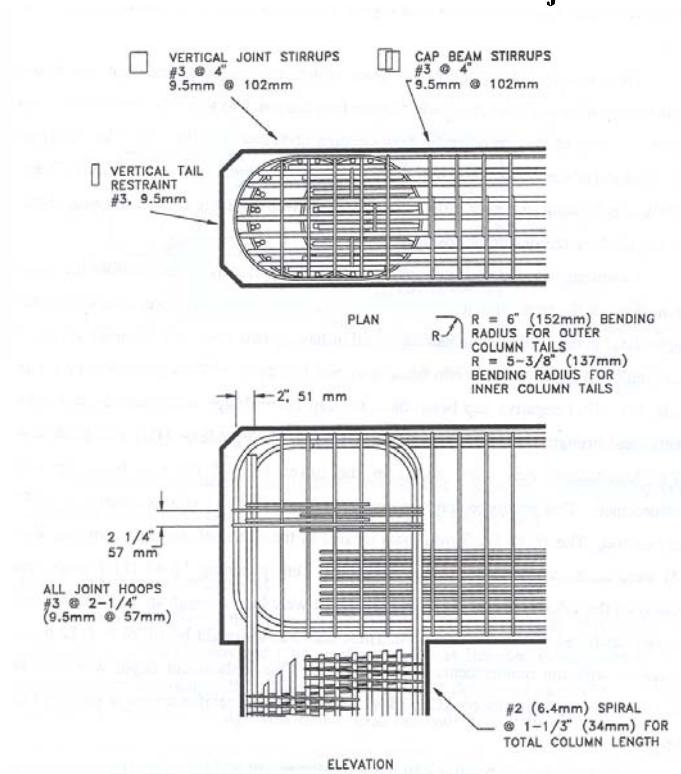


Figure 1 - Knee-Ing-RCT-7

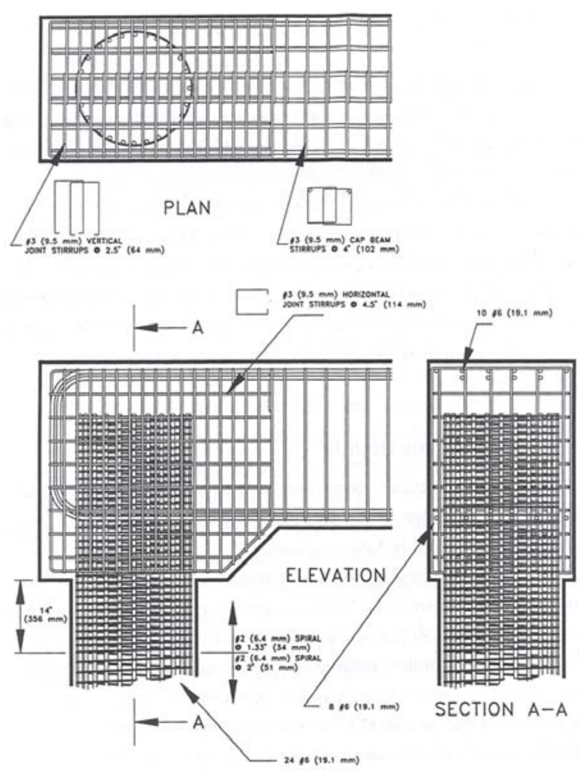


Figure 2 - Knee-Ing-RND-4

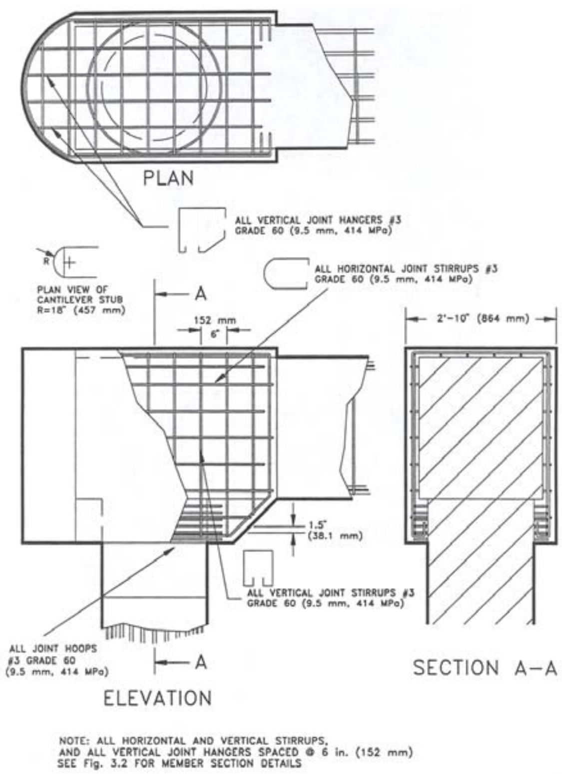


Figure 3 - Knee-Ing-RND-6

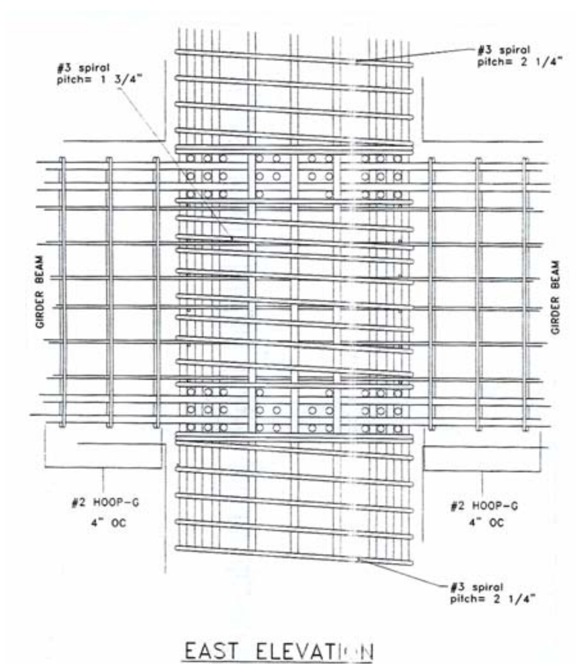


Figure 4 - Longitudinal Elevation of DD-Maz-1

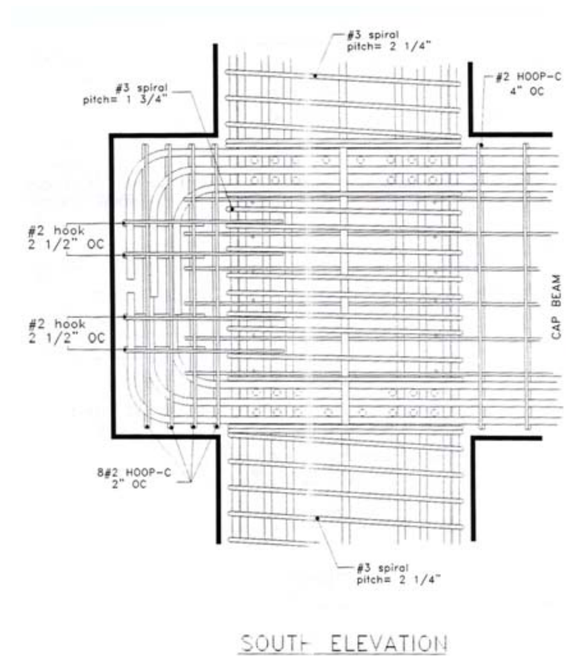


Figure 5 - Transverse Elevation of DD-Maz-1

Figure 6 - Longitudinal Elevation of DD-Maz-2

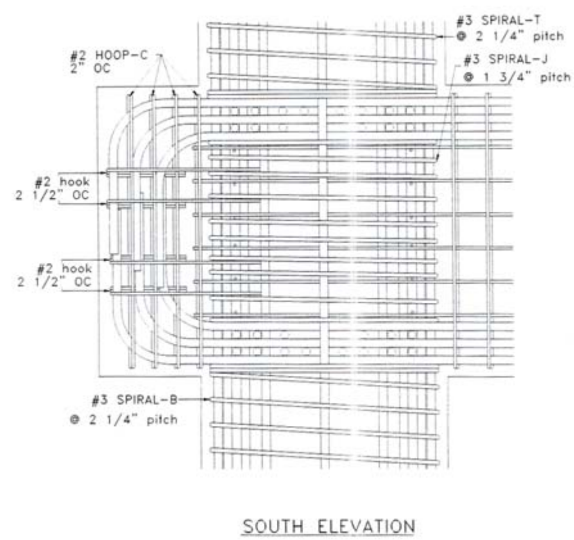


Figure 7 - Transverse Elevation of DD-Maz-2

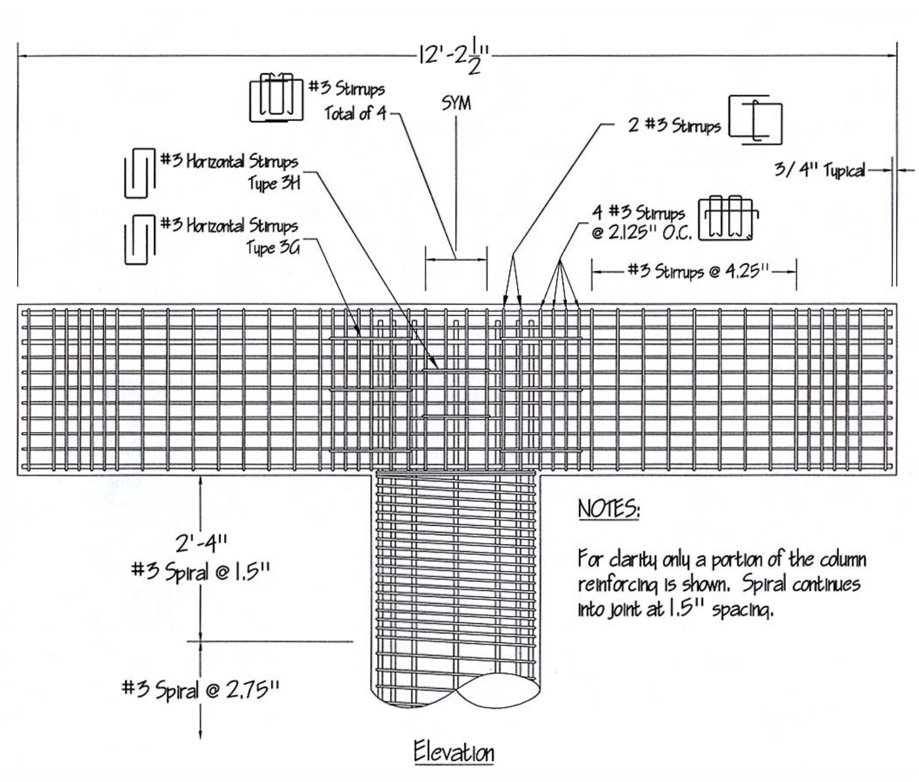


Figure 8 - Tee-Nai-RND-A1

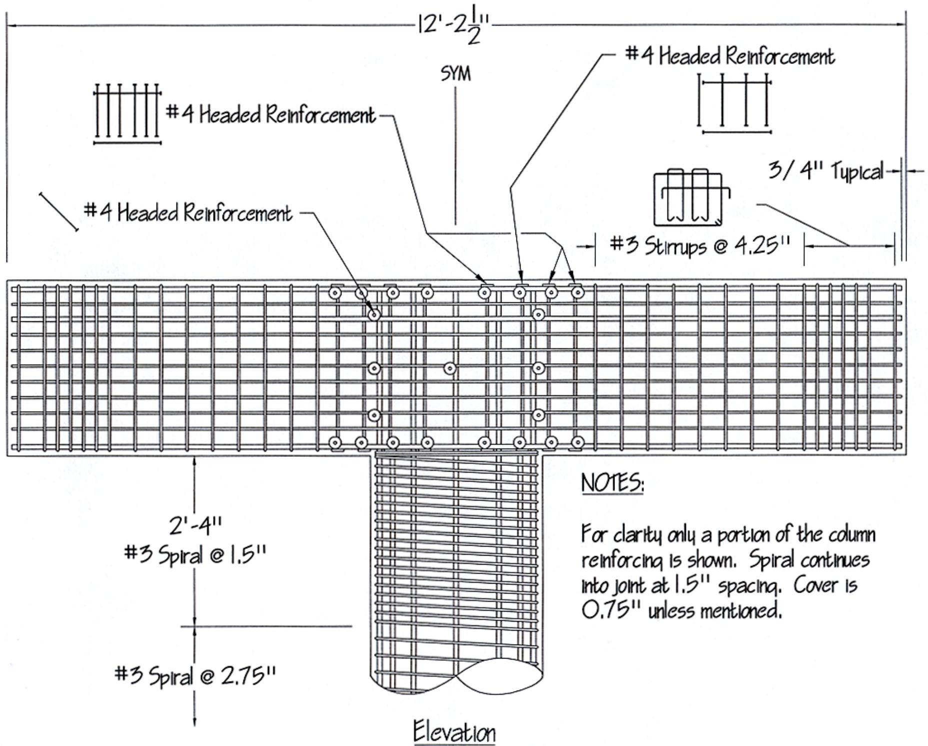


Figure 9 - Tee-Nai-RND-A2

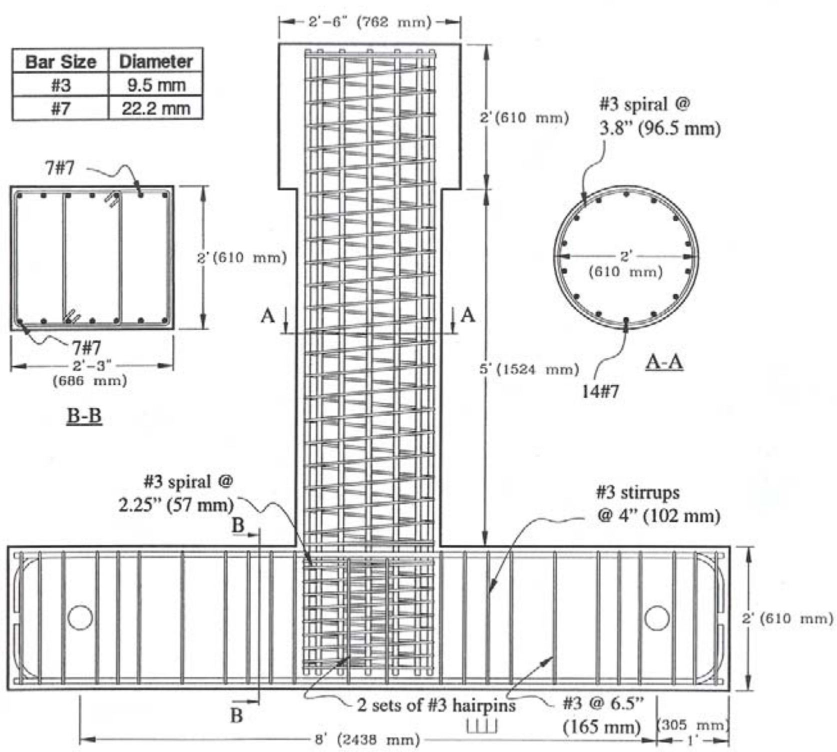
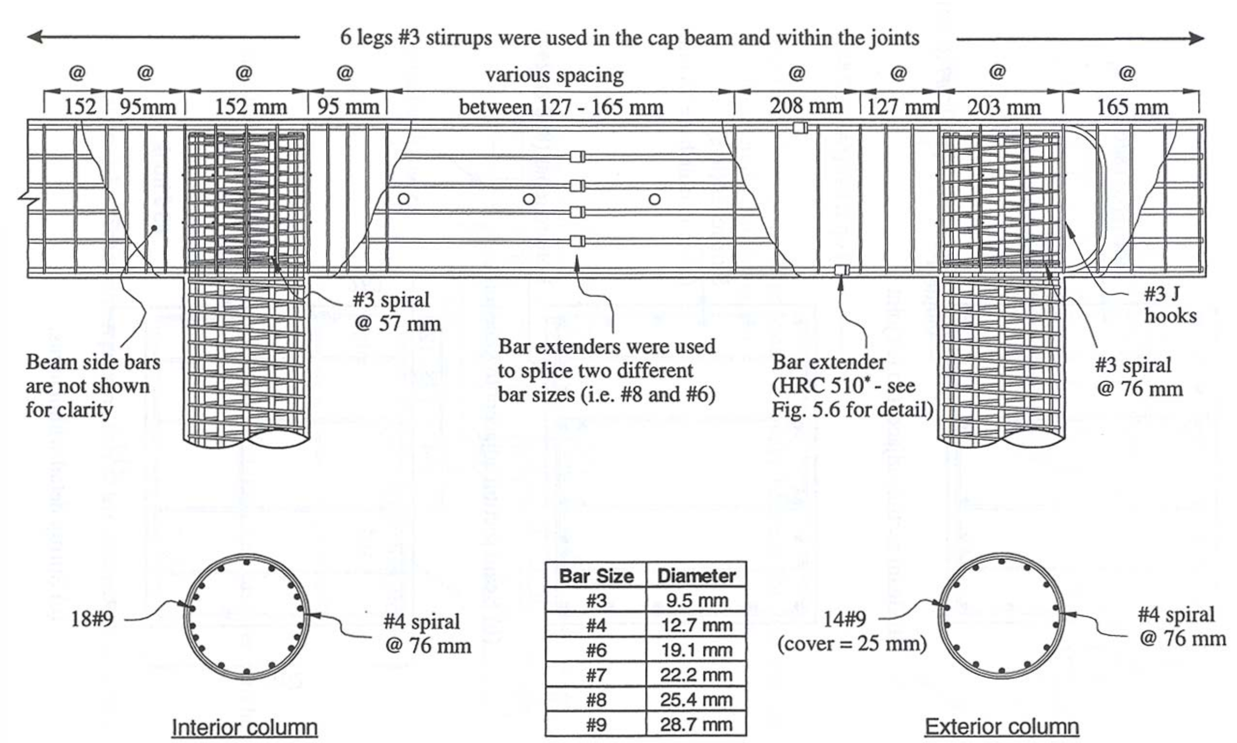


Figure 60 - Tee-Sri-RND-IC1



Bar Size	Diameter
#3	9.5 mm
#4	12.7 mm
#6	19.1 mm
#7	22.2 mm
#8	25.4 mm
#9	28.7 mm

Figure 71 - Knee-Sri-RND-1 & Tee-Sri-RND-2

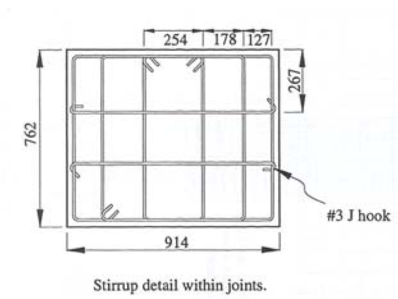


Figure 82 - Joint Section for Knee-Sri-RND-1 & Tee-Sri-RND-2 (units in mm)

SUMMARY OF HYSTERESIS LOOPS
Caltrans Local Deformations Project

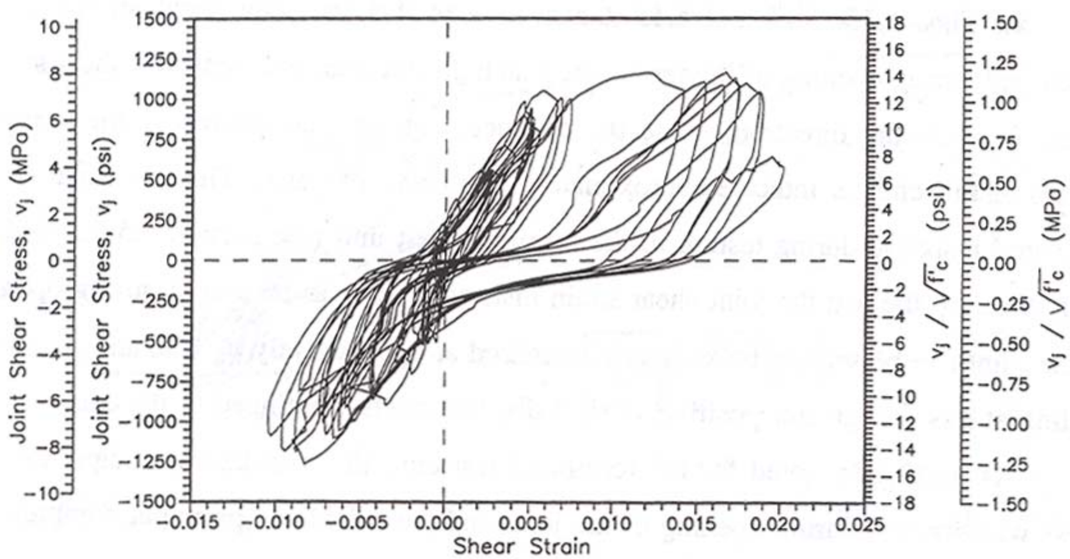


Figure 1 - Knee-Ing-RCT-7

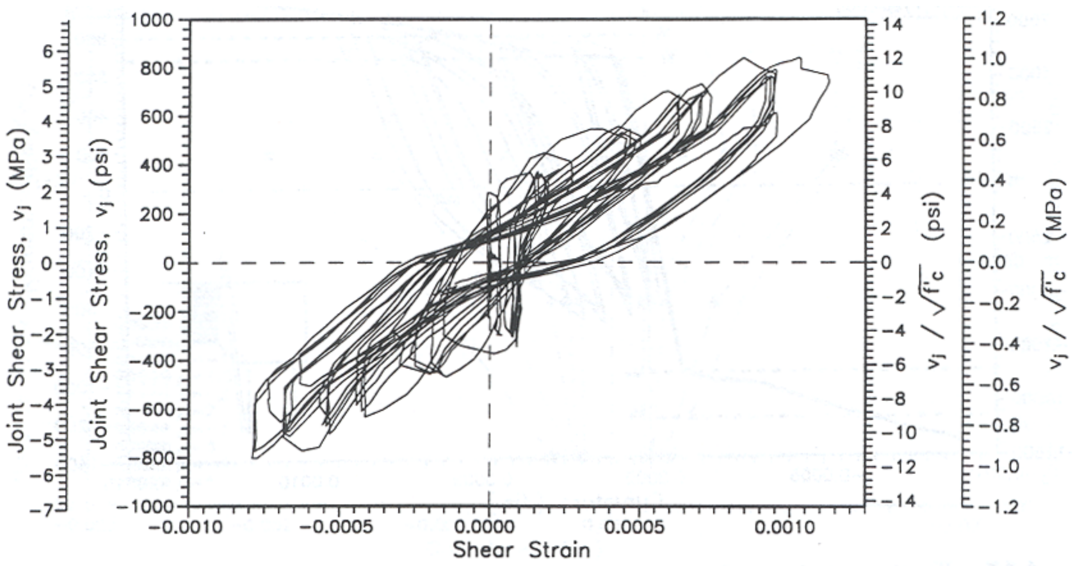


Figure 2 - Knee-Ing-RND-4

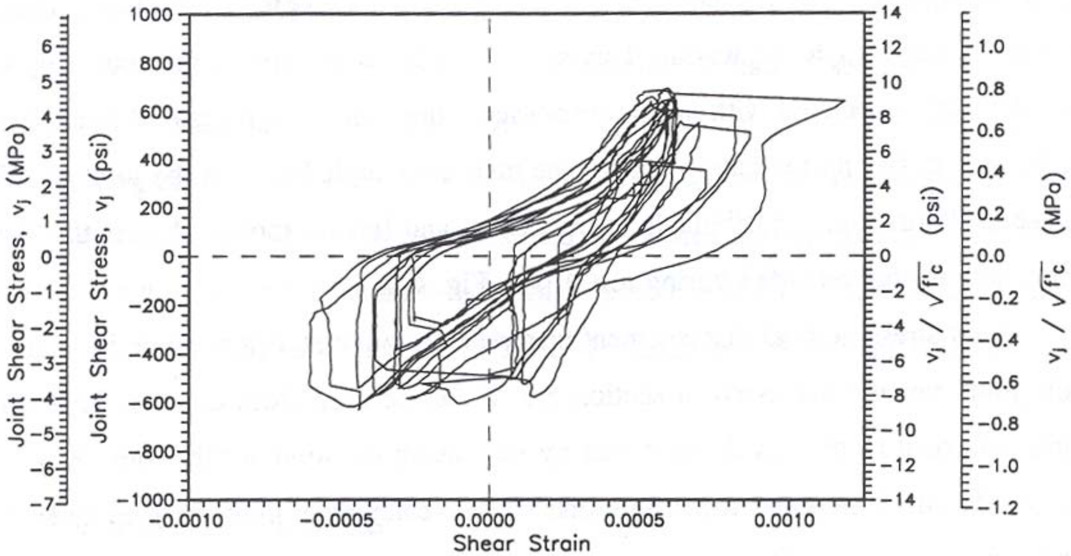


Figure 3 - Knee-Ing-RND-6

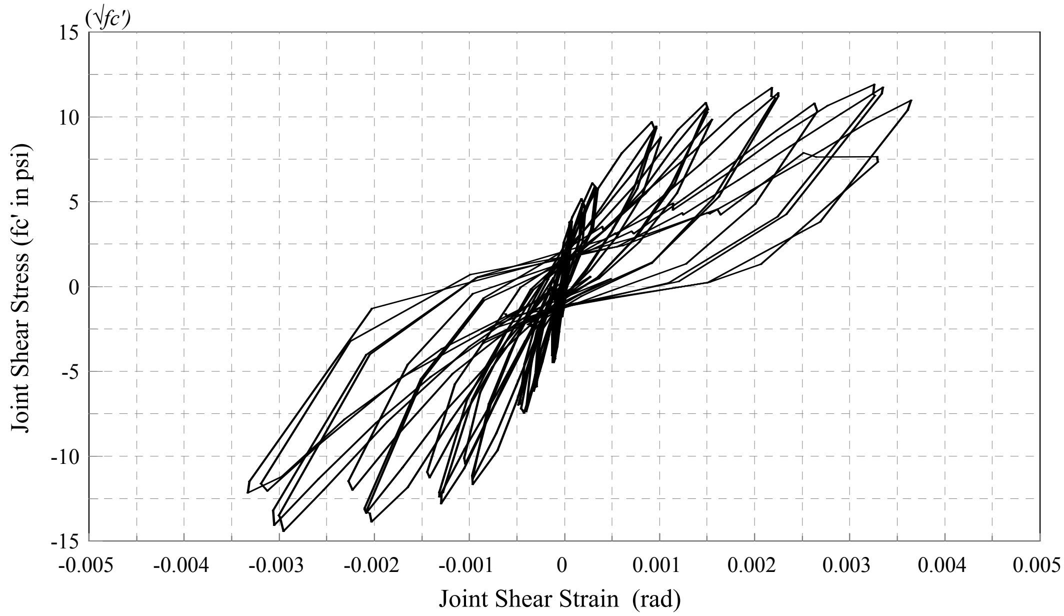


Figure 4 - DD-Maz-T-1

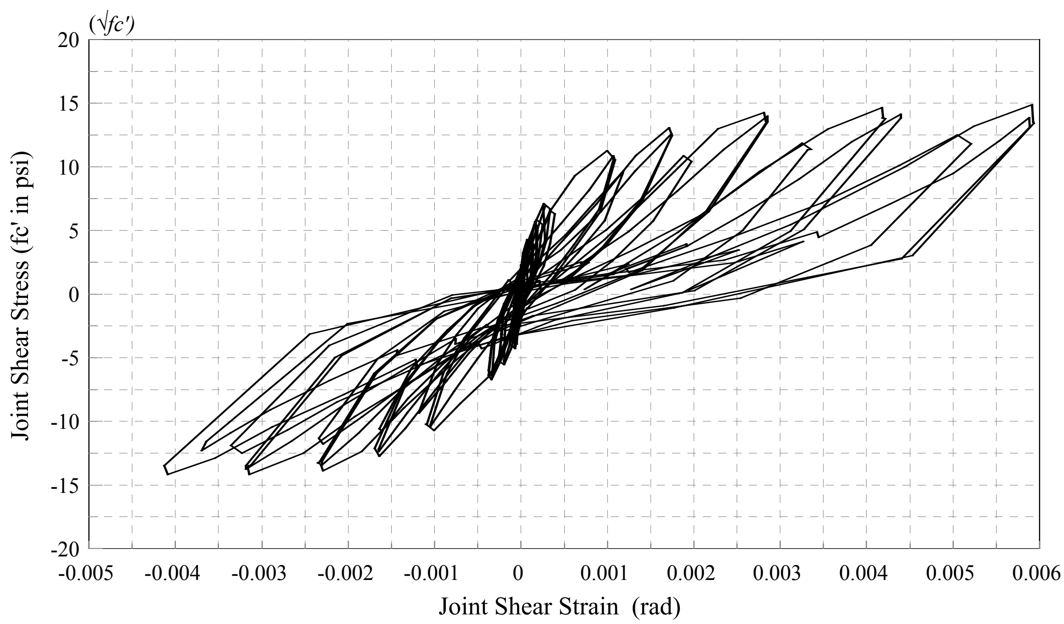


Figure 5 - DD-Maz-L-1

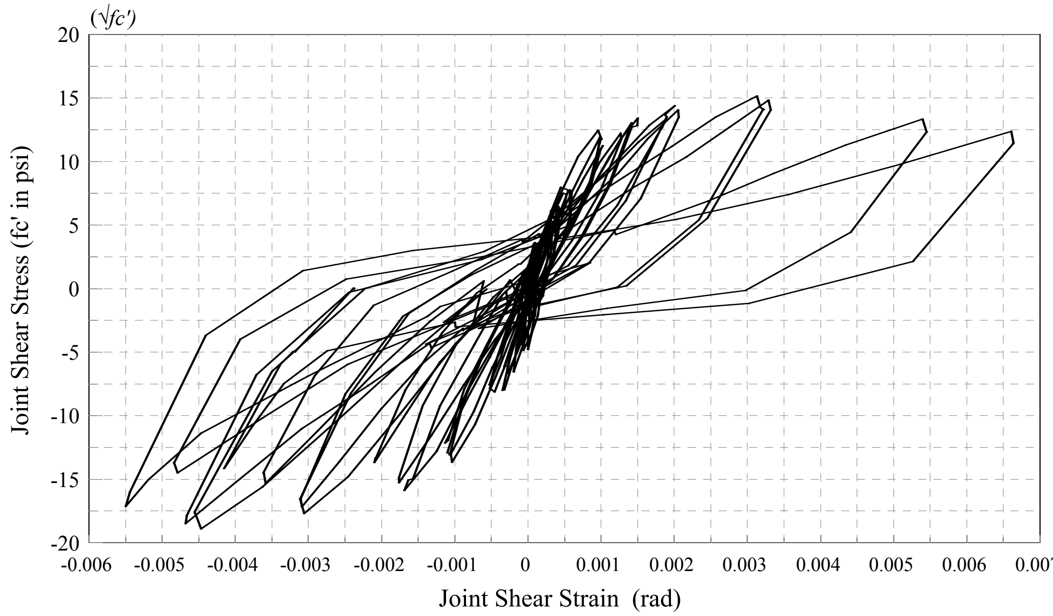


Figure 6 - DD-Maz-T-2

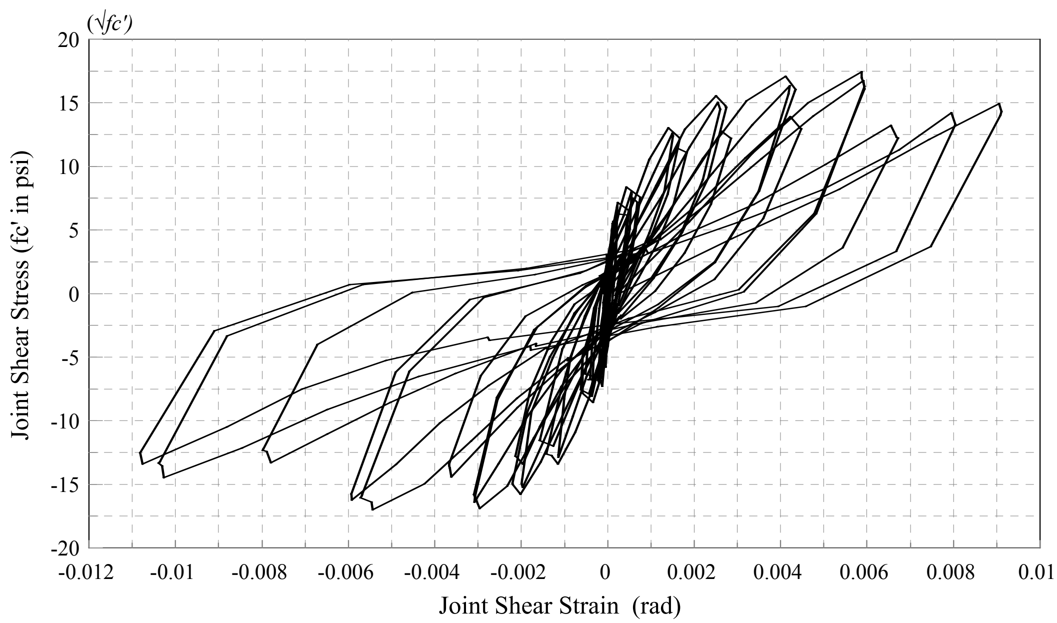


Figure 7 - DD-Maz-L-2

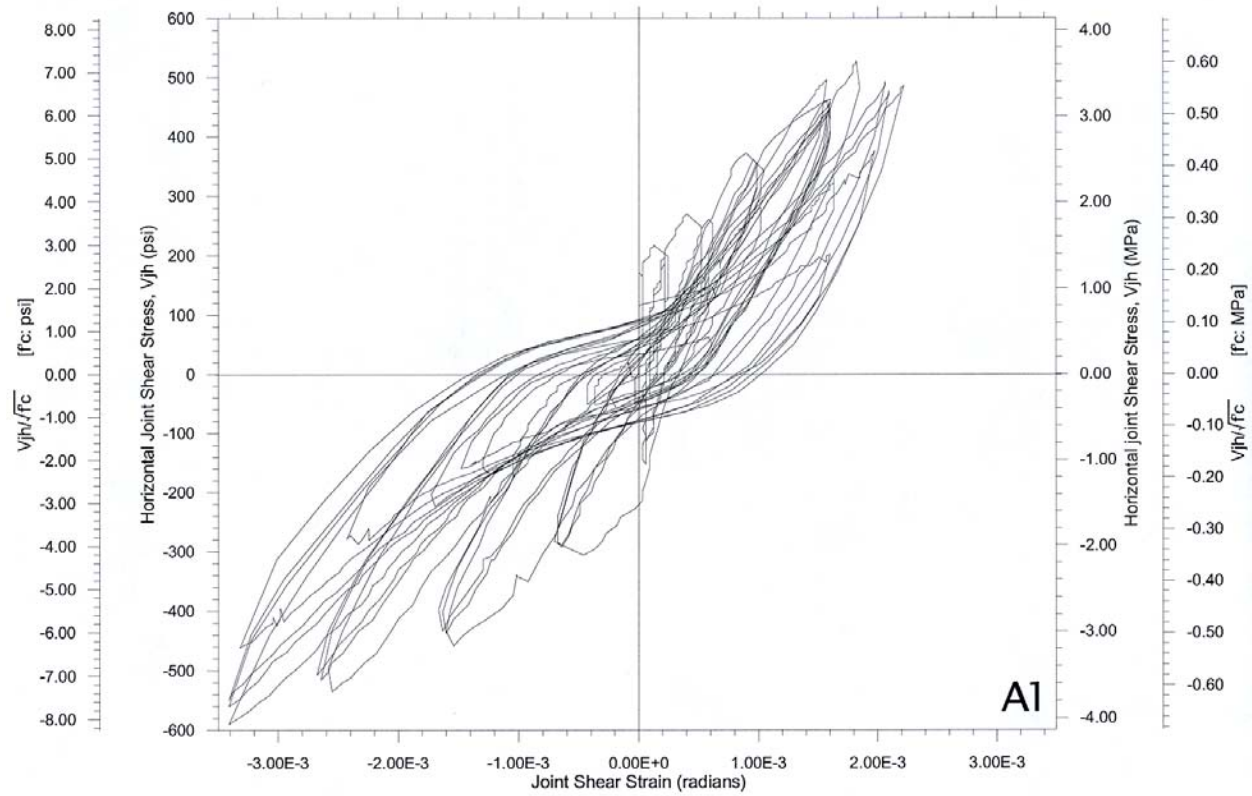


Figure 8 - Tee-Nai-RND-A1

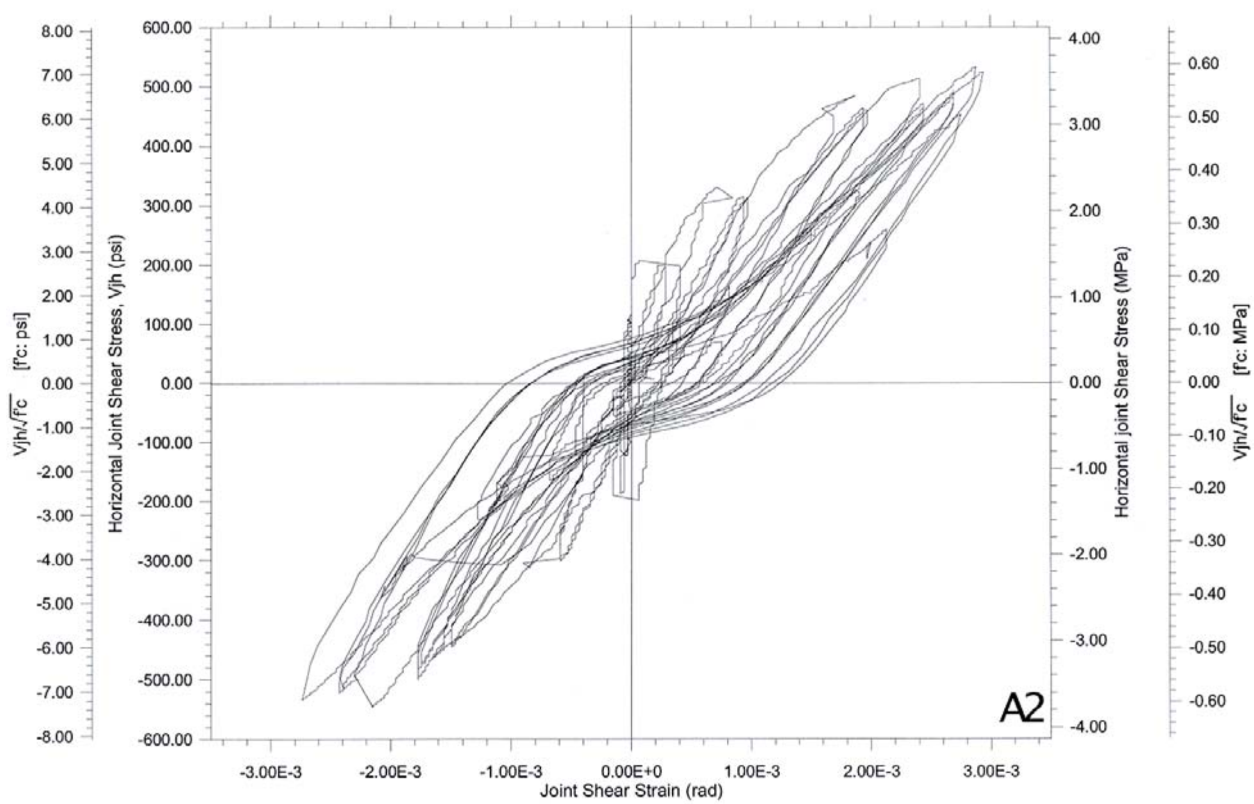


Figure 9 - Tee-Nai-RND-A2

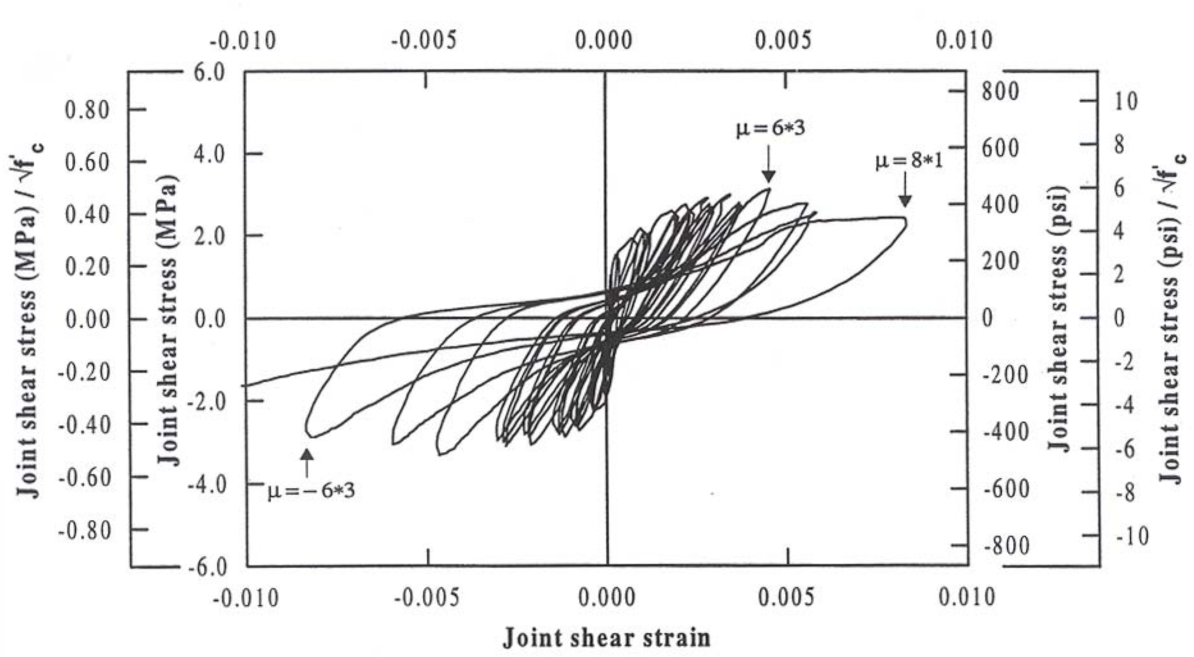


Figure 10 - Tee-Sri-RND-IC1

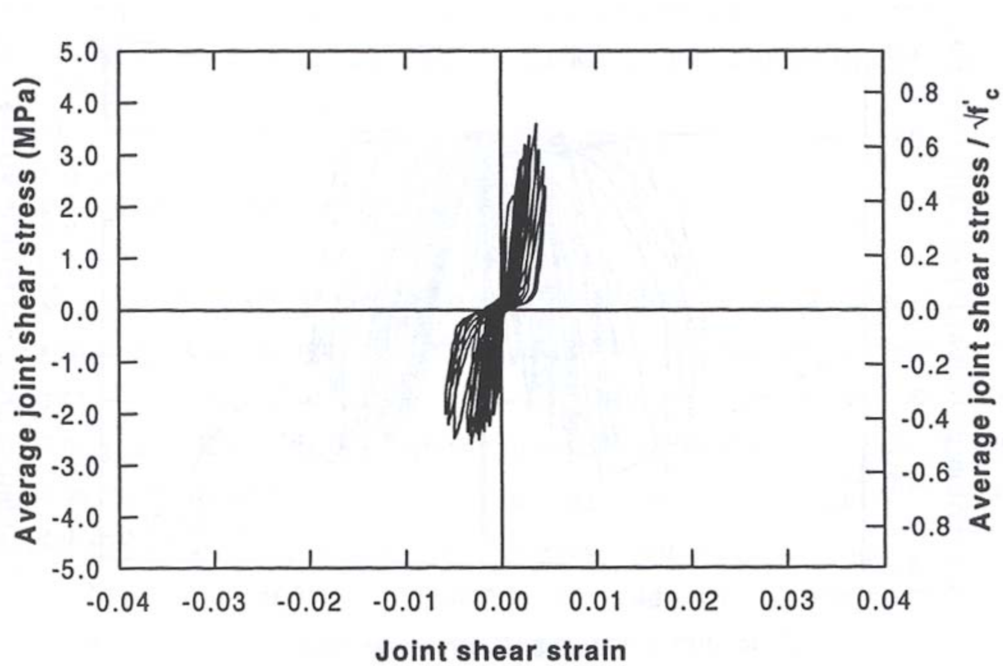


Figure 11 - Knee-Sri-RND-1

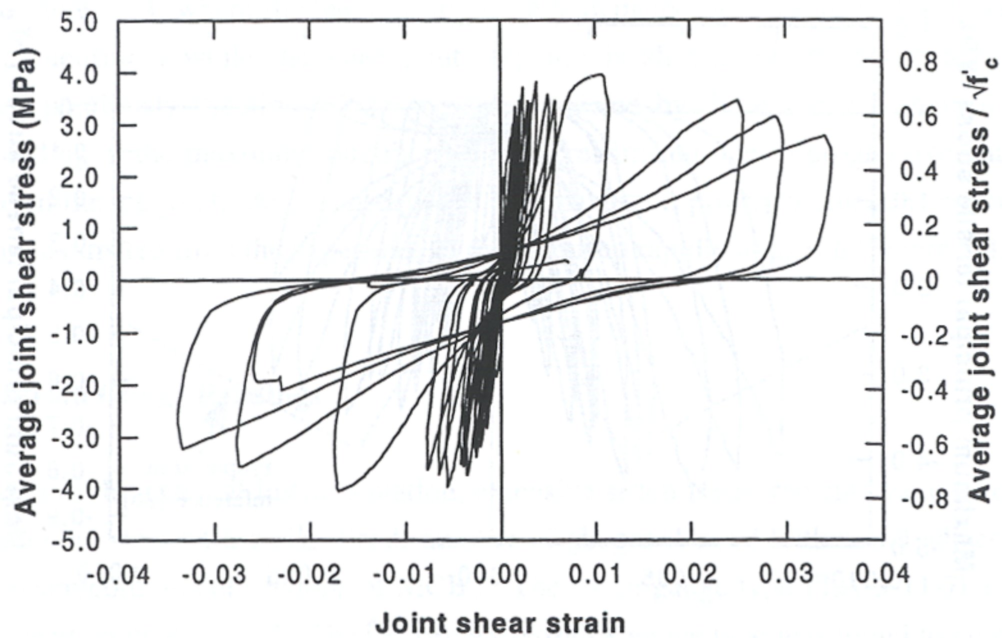


Figure 12 - Tee-Sri-RND-2

Knee-Ing-RCT-7

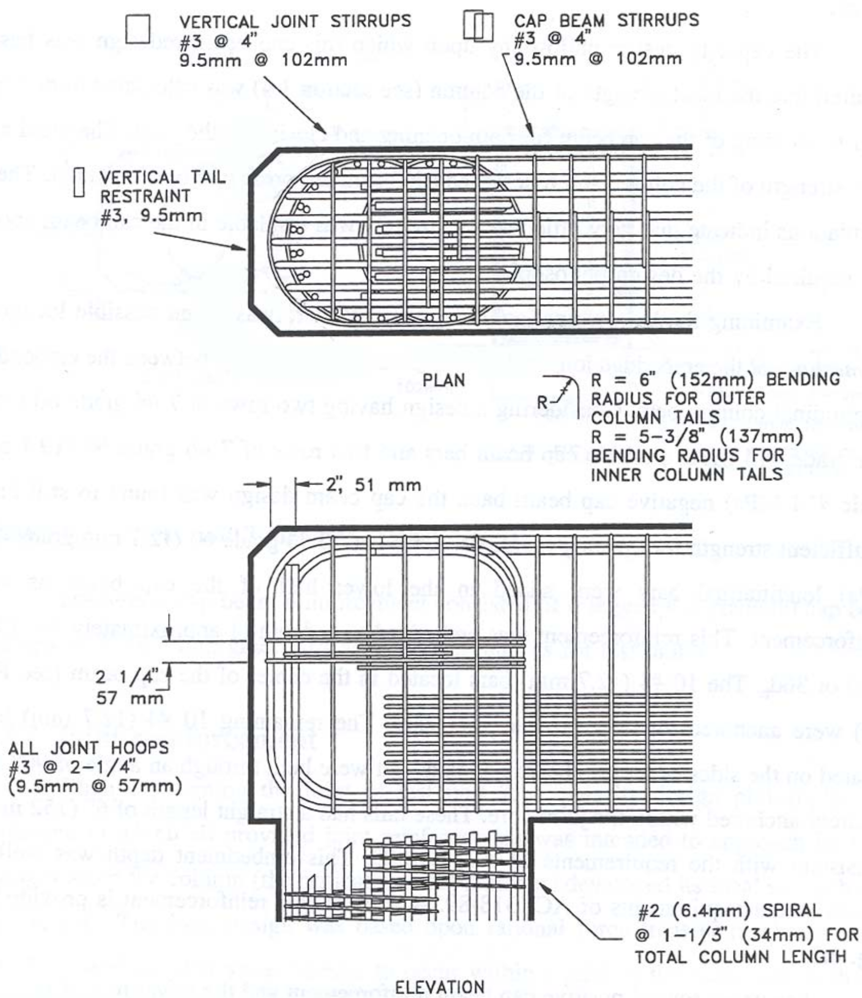


Figure 1 - Plan and Elevation of Specimen Knee-Ing-RCT-7

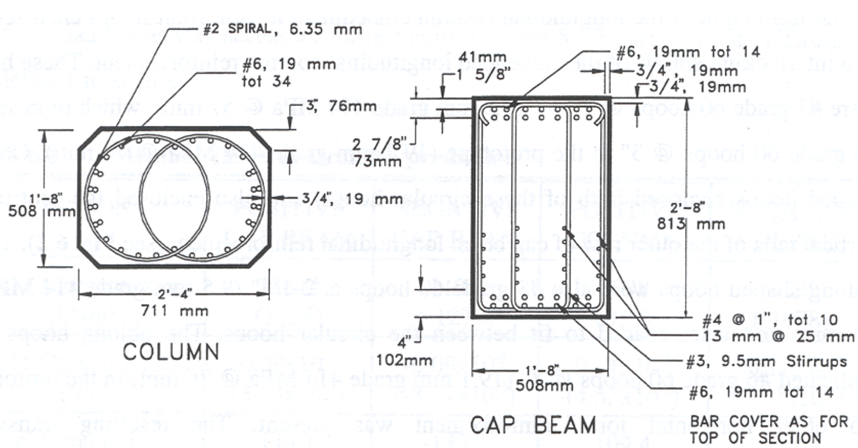


Figure 2 - Cross Sections

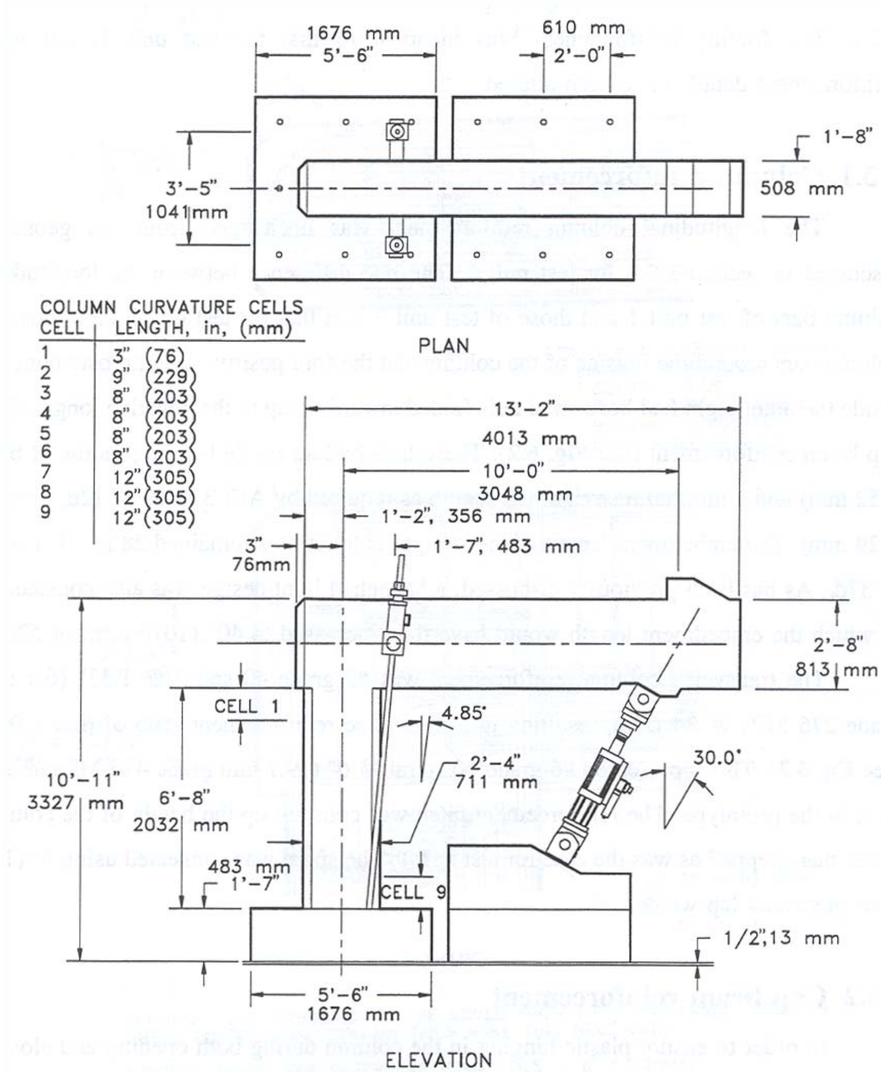


Figure 3 – Specimen Setup

Reference: Knee-Ing-RCT-7
Ingham, et al, UCSD (SSRP 94/12)

Type:	Knee
Column:	Rectangular w/ 3" (7.6 cm) chamfered edges - $\rho_l = 2.8$ %
Joint:	3" (7.6 cm) stub at back of joint (no haunch)
Scale:	1/3
Testing Protocol:	Pseudo-static, early cycles under load control, later cycles by displacement control

TEST OBJECTIVE:

To evaluate the performance of a new knee joint design proposed by the researchers to correct deficiencies found in previous tests.

TEST SCOPE:

The specimen was loaded with dead load before cyclic loading began. Cracking occurred at the column-joint construction seam, near the dead load application point, and down the outside face of the column (column cracking expected). Initial cyclic loading was applied under force control with cycles up to approximate flexural yielding. Beyond this level, load was applied under displacement control with increasing levels of displacement ductility (up to a maximum of μ_Δ=+3.6, -4.0).

DESIGN PHILOSOPHY:

Knee-Ing-RCT-7 was a redesign of the first specimen in the series, a non-ductile design based on actual Caltrans bridges. It was detailed to avoid the deficiencies found in the original test and in retrofits, specifically a lack of proper confinement to the joint region and insufficient embedment of column longitudinal bars. It was intended as a possible replacement for Caltrans standard practice. The design sought well-behaved, ductile joint behavior that forced inelastic deformation to concentrate in the column and not in the joint.

SPECIMEN DETAILS:

f_c': Lower Column = 5340 psi (36.8 MPa) *(column poured up to anticipated haunch, but no haunch used)*
Upper Column = 7370 psi (50.8 MPa)
Cap Beam = 7370 psi (50.8 MPa)
Joint = 7370 psi (50.8 MPa)

COLUMN:
Rectangular column, 1'-8" x 2'-4" (50.8cm x 71.1cm); Interlocking double spiral
L_c = 8' (2.43 m) top of footing to cap beam centroid (modeled 43% of prototype height)

Longitudinal Reinforcement:
34 #6 - ρ_l = 2.8 %
12 "positive" bars for opening moments bent in to joint to lap with beam negative reinforcement (min. tail length of $12d_b$); rest were straight, cut off 4" (10.2 cm) from top surface of cap beam (embedment of $37d_b$)
Transverse Reinforcement:
2x #2 Gr.40 (276 MPa) spiral @$1^1/_3$" (3.4 cm) pitch, ρ_t = 1.09%
Axial Load: 2.78% Ag*f_c' constant gravity load (total axial load varied with lateral load)

BEAM:
Rectangular - 1'-8"b x 2'-8"h (50.8cm x 81.3cm)
L_b=12' to column centerline (3.66 m)
Longitudinal Reinforcement:
Top: 2 layers, 7 #6 each layer (14 total)
Bottom: 2 layers, 7 #6 each layer (14 total)
Additional 20 #4 @ 1" (2.5 cm) in 4 vertical rows (tied to stirrups)

Outer layers of beam reinforcement formed into 'U' shape through stub on back of joint; Inner layer of beam top reinforcement lapped to column negative reinforcement.
Beam strength modification intended as capacity design to allow column to develop ideal strength before beam yielding.(compared to as-built in earlier tests)

Transverse Reinforcement:
4-leg #3 stirrups @ 4" (10.2 cm)
Skin Reinforcement:
no additional beyond the #4 bars used as positive moment reinforcement

JOINT:
same dimensions as beam and column with addition of 3" (7.6 cm) stub on back of joint
#3 horizontal joint hoops @ $2^{1}/_{4}$" (5.7 cm) around column longitudinal reinforcement
#3 horizontal joint stirrups around edge of joint @ $4^{1}/_{2}$" (11.4 cm)
overall horizontal transverse reinforcement ratio: $\rho_t = 4.37\%$
#3 vertical joint stirrups around edge of joint @ 4" (11.4 cm)
#3 vertical tail restraint
total joint reinforcement: 1.76 in^2(11.35 cm^2) (11.8% of column longitudinal reinforcement area)

QUANTIFIED RESPONSE:

stress quantities in ***psi (MPa)***	Open	Close
Max Joint Shear Stress	$14.3\sqrt{f_c'}$ $(1.19\sqrt{f_c'})$	$15.1\sqrt{fc'}$ $(1.25\sqrt{f_c'})$
Joint Strain at Max Shear Stress	0.017 rad	0.008 rad
Max Joint Shear Strain	0.019 rad	0.01 rad
Joint Stress at Max Shear Strain	$7\sqrt{f_c'}$ $(0.58\sqrt{f_c'})$	$13\sqrt{fc'}$ $(1.08\sqrt{f_c'})$
Max Principal Tension	$14\sqrt{f_c'}$ $(1.16\sqrt{f_c'})$	$13\sqrt{fc'}$ $(1.08\sqrt{f_c'})$
Max Principal Compression	$14\sqrt{f_c'}$ $(1.16\sqrt{f_c'})$	$13\sqrt{f_c'}$ $(1.08\sqrt{f_c'})$
% Deformation due to joint	8-10%	8-10%

FAILURE MODE:

Rupture of column spiral and buckling of column longitudinal reinforcement in plastic hinge region below joint. (Occurred during cycles at displacement limits of actuator)

HYSTERESIS DESCRIPTION:

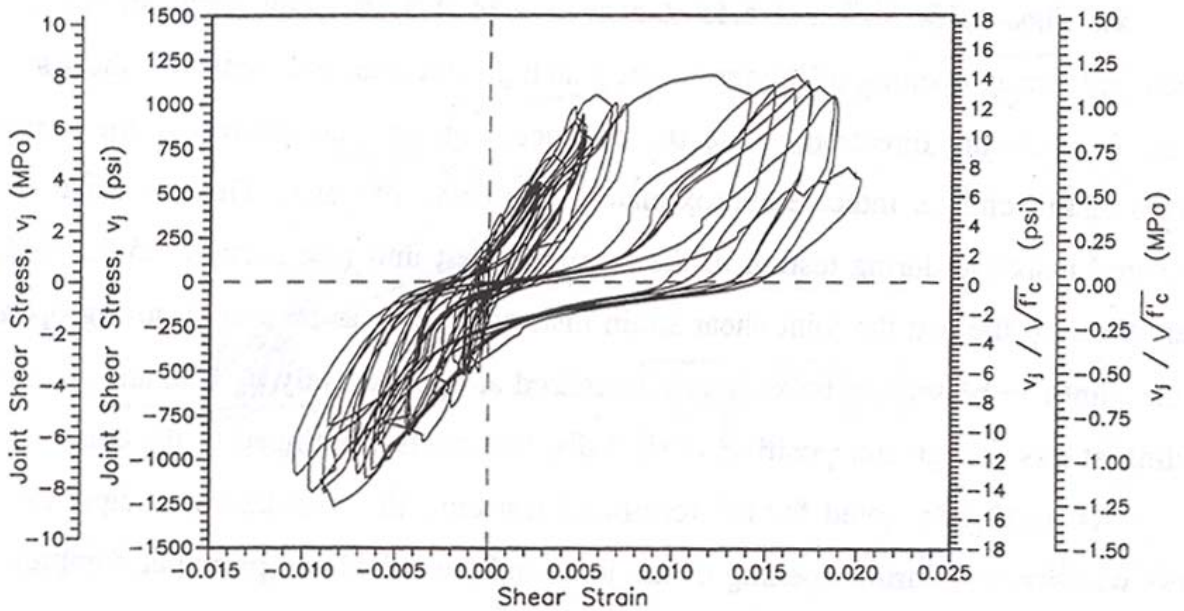


Figure 4 - Joint Shear Stress vs. Shear Strain for Knee-Ing-RCT-7

Larger ultimate strains in closing, generally good response until larger opening cycles when strains suddenly increased. (strains above +0.006) Further cycles resulted in more stiffness degradation and finally some strength degradation in the opening direction with slight strength reductions in closing. Severe pinching of the hysteresis occurred at these later cycles and on to failure.

Shear stress was calculated based on full joint width (20" (50.8 cm)) and full distance between column reinforcement (~26.5" (67.3 cm)), based on moments at column-joint interface. (instead of beam/joint interface as in previous tests, this due to detailing intended to force the column/joint interface to be the critical section) Principal tension and compression were calculated from horizontal and vertical joint stresses based on a 45-degree dispersion of cap beam and column axial forces into the joint.

The joint reached similar maximum shear levels in both loading directions though with very different stiffness characteristics; it was stiffer in the joint closing direction than in joint opening, especially for large strains (Roughly twice the strain in opening for the largest cycles). Beam and column flexural strengths were different in the two directions, but not so different as to affect the joint behavior to the extent observed.

Joint deformations accounted for roughly 8-10% of total specimen displacement throughout the test in both loading directions. (graph in report is difficult to read) Column deformations consistently caused 70-90% of total displacements throughout the test.

Yielding of the column spiral reinforcement and subsequent buckling of the column longitudinal reinforcement in a joint closing cycle initiated failure. Further cycles caused rupture of “positive”(inside face) column bars as subjected to alternate cycles of buckling and straightening. The column was, however, stable within the spiral nearest the negative (outer) side of the column.

RESULTS/CONCLUSIONS:

The specimen exhibited good behavior of due to the ductile redesign. The provision of appropriate embedment (through use of hooks) for column and beam longitudinal bars effectively prevented failure by pullout of column bars and enhanced the formation of the joint compression strut. Joint deformations, while not insignificant, could possibly be ignored without too much effect on overall system displacements.

Bibliography:
Ingham, Jason M., M.J. Nigel Priestley, Frieder Seible. "Seismic Performance of Bridge Knee Joints - Volume 1: Rectangular Column/Cap Beam Experimental Results (Report No. SSRP - 94/12)." La Jolla, CA: Structural Systems Research, University of California, San Diego, 1994.

R/C Bridge Beam-Column Joints
Knee-Ing-RND-4

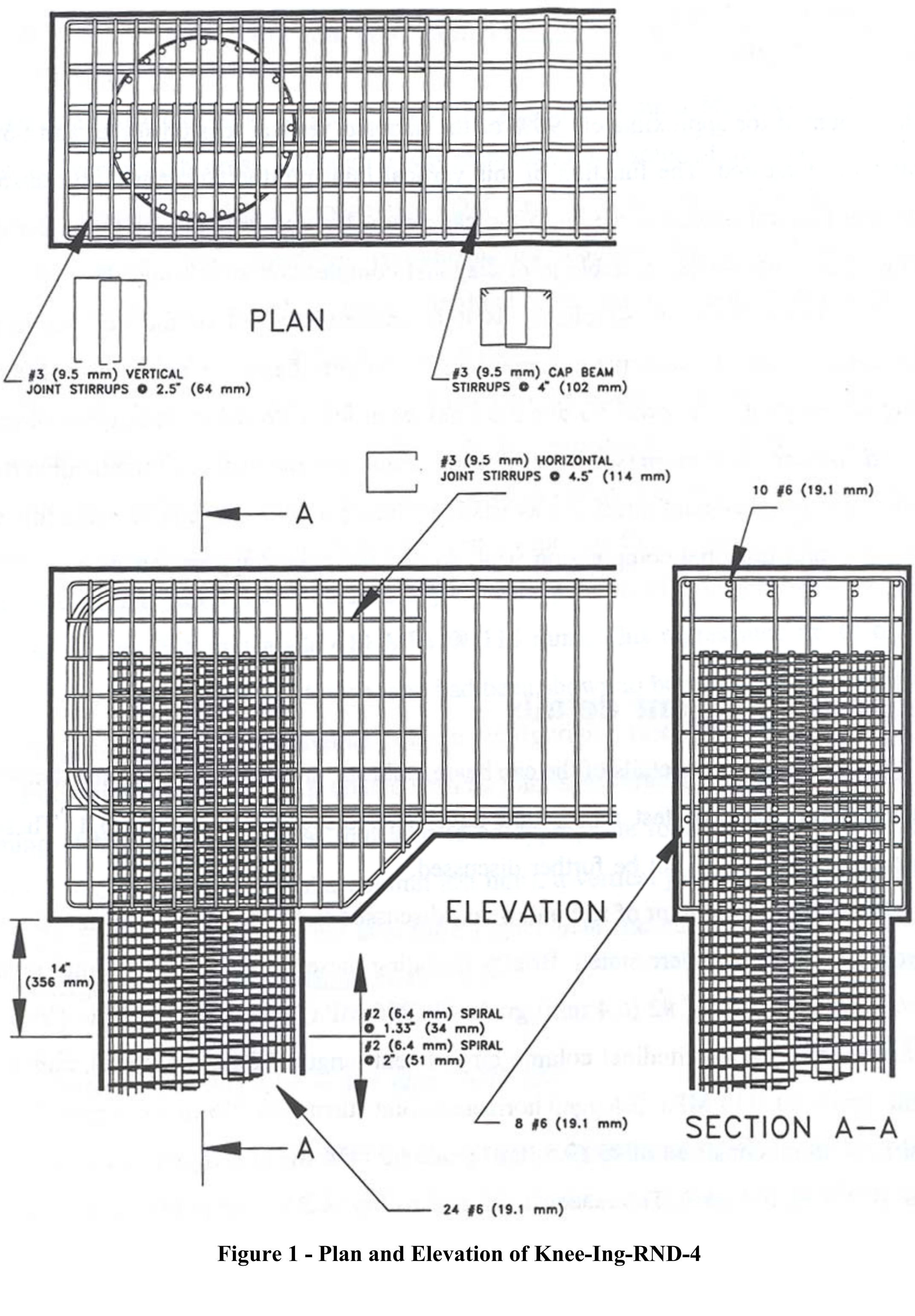


Figure 1 - Plan and Elevation of Knee-Ing-RND-4

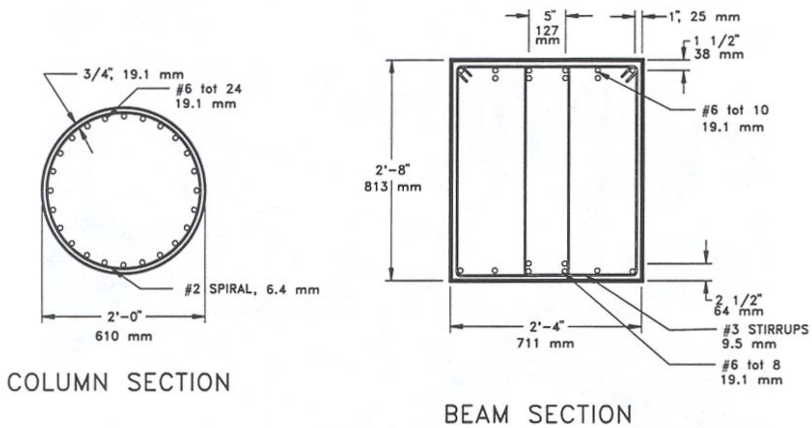


Figure 2 – Cross Sections

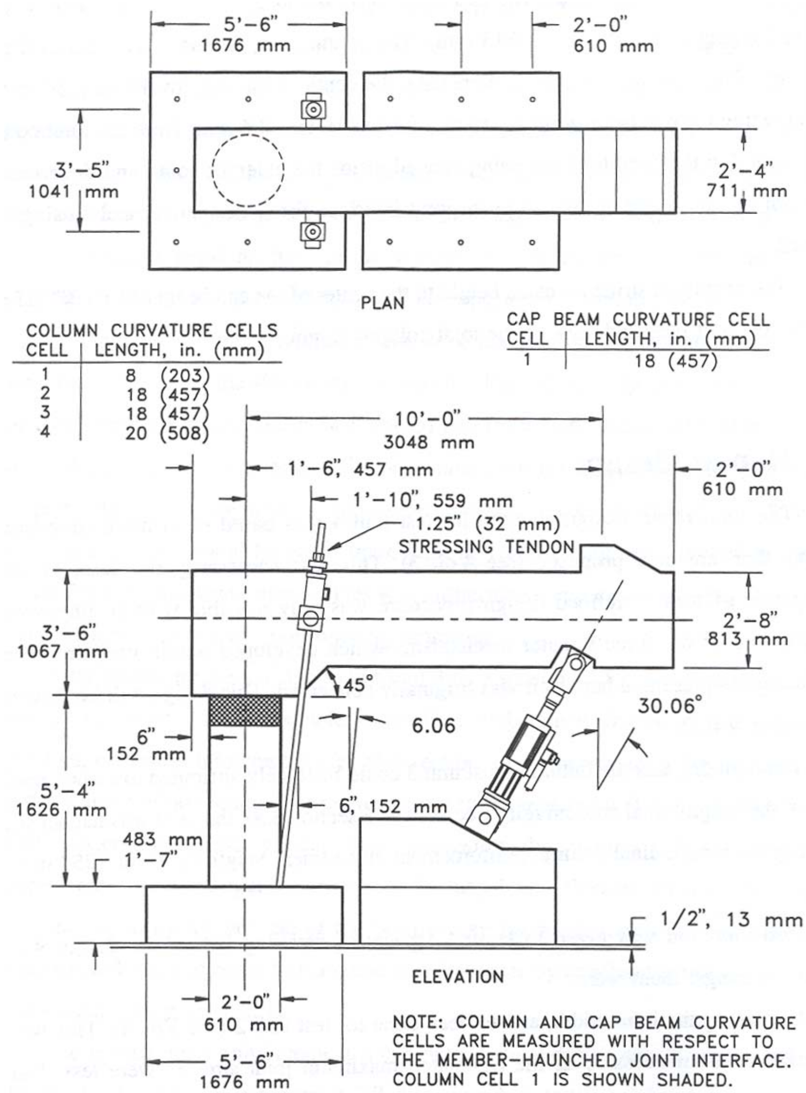


Figure 3 - Specimen Setup

Reference: Knee-Ing-RND-4
Ingham, et al, UCSD (SSRP 94/17)

Type: Knee, repair of 1987 design tested in same report (their Specimen #3)
Retrofit type: Repair – complete replacement of an originally non-ductile joint
Column: Circular - ρ_l = 2.33%
Joint: 10” (25.4 cm) haunch, same width as beam
Scale: 1/3
Testing Protocol: Pseudo-static, early cycles under load control, later by displacement control

TEST OBJECTIVE:

To evaluate the performance of a haunched joint repair strategy in a previously tested specimen (see Specimen #3 in cited report). All concrete in the joint of the original specimen was removed, the joint reinforcement replaced, and a haunched joint poured for testing.

TEST SCOPE:

The specimen was loaded with dead load before cyclic loading began. Initial cyclic loading was applied under force control with cycles up to approximate flexural yielding. Beyond this level, load was applied under displacement control with increasing levels of displacement ductility (up to a maximum of μ_Δ=±5.0).

DESIGN PHILOSOPHY:

The repair strategy employed for Knee-Ing-RCT-7 was designed for ductile response and the haunched detail that provided more embedment length was chosen to remedy the embedment failure of column longitudinal reinforcement observed in the original specimen. Also, a greater volume of vertical joint reinforcement was provided to help develop the full column flexural tension force so that the joint compression strut could develop.

SPECIMEN DETAILS:

f_c’: Column = 5200 psi (35.9 MPa)
Cap Beam = 6290 psi (43.4 MPa)
Joint = 4850 psi (33.4 MPa)

COLUMN:
24” (61 cm) diameter circular column
L_c = 7’-6” (2.29 m) top of footing to cap beam centroid (modeled 43% of prototype height)
Longitudinal Reinforcement:
24 #6, straight cut off 10” (25.4 cm) from top surface of cap beam – ρ_l = 2.33%
Transverse Reinforcement:
#2 spiral @ 2” (5.1 cm) pitch in lower portion; (ρ_t = 0.44%)
#2 spiral @ $1^1/_3$” (3.4 cm) in 14” (35.6 cm) below joint and in 2’-8” (81.3 cm) of joint (ρ_t = 0.66%)

Axial Load: 4.23% Ag*f_c' constant gravity load (total axial load varied with lateral load)

BEAM:
Rectangular - 2'-4"b x 2'-8"h (71.1cm x 81.3cm)
L_b=12' (3.66 m) to column centerline
Longitudinal Reinforcement:
  Top: outer layer 6 #6, inner layer 4 #6 (total 10)
  Bottom: outer layer 6 #6, inner layer 2 #6 (total 8)
  6 bars on top of section and bottom of section detailed with vertical tails welded together at cap-beam mid-height with full penetration butt welds
Transverse Reinforcement:
  closed 4-leg #3 stirrups @ 4" (10.2 cm) in remainder of beam (1^{st} at 4" (10.2 cm) from col. face)
Skin Reinforcement:
  none

JOINT:
10" (25.4 cm) haunch with 45º chamfer to cap-beam, 6" (15.2 cm) stub on back of joint
Vertical Reinforcement:
  Open-ended 4-legged #3 stirrups @ $2^1/_2$" (6.4 cm) (intended to transfer column flexural tension force to cap beam flexural compression force to develop compression strut)
Horizontal Reinforcement:
  #2 spiral @ $1^1/_3$" (3.4 cm) (continuation of column spiral) to 10" (25.4 cm) from cap beam top surface - ρ_t = 0.66 %
  2-legged #3 stirrups @ $4^1/_2$" (11.4 cm)

QUANTIFIED RESPONSE:

stress quantities in ***psi (MPa)***	Open	Close
Max Joint Shear Stress	$12\sqrt{f_c'}$ $(1.0\sqrt{f_c'})$	$11.6\sqrt{f_c'}$ $(0.96\sqrt{f_c'})$
Joint Strain at Max Shear Stress	0.00103 rad	0.0008 rad
Max Joint Shear Strain	0.0011 rad.	0.0009 rad.
Joint Stress at Max Shear Strain	$10.8\sqrt{f_c'}$ $(0.90\sqrt{f_c'})$	$11.4\sqrt{f_c'}$ $(0.95\sqrt{f_c'})$
Max Principal Tension	$12.5\sqrt{f_c'}$ $(1.04\sqrt{f_c'})$	$11\sqrt{f_c'}$ $(0.91\sqrt{f_c'})$
Max Principal Compression	$12\sqrt{f_c'}$ $(1.0\sqrt{f_c'})$	$10.5\sqrt{f_c'}$ $(0.87\sqrt{f_c'})$
% Deformation due to joint	2-3%	2-3%

FAILURE MODE:

Initially, buckling of column longitudinal reinforcement and later rupture of column spiral in plastic hinge region below point where spiral pitch changed from 1.33” (3.4 cm) to 2” (5.1 cm). (failure occurred in region with 2” (5.1 cm) pitch) Final failure occurred at maximum actuator displacement with the pull-out of the column spiral and buckling of reinforcement on both sides of the column.

HYSTERESIS DESCRIPTION:

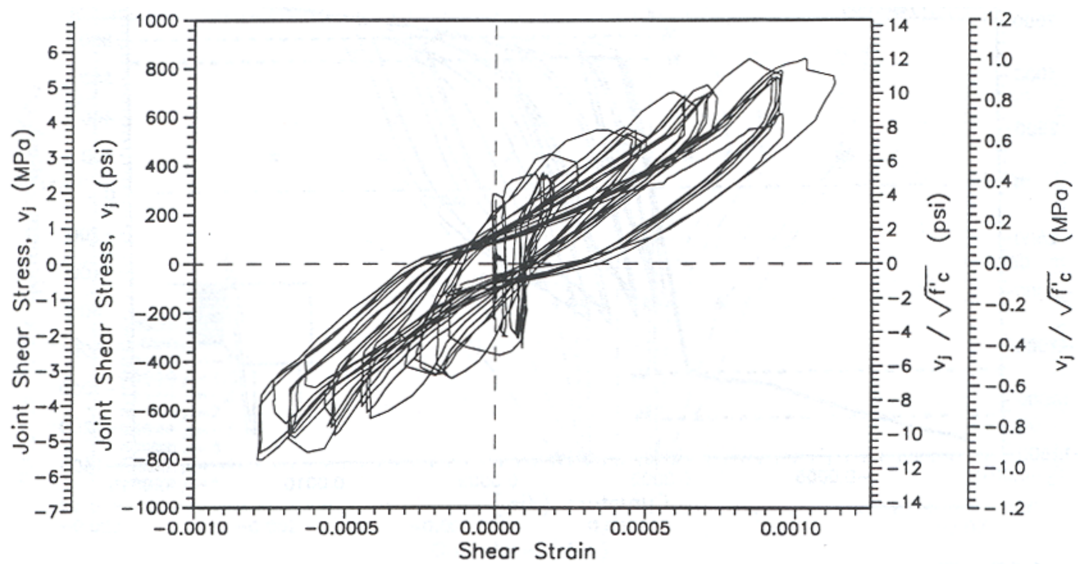


Figure 4 – Joint Shear Stress vs. Shear Strain for Knee-Ing-RND-4

This specimen was a repair of a specimen (#3) with non-ductile detailing discussed earlier in the cited report. The concrete in the old joint was completely removed and recast with ductile detailing in the joint and original steel in the column and beam. The test was intended to test a suggested retrofit strategy.

Shear stress was calculated based on full joint width (28” (71.1 cm)) and full distance between column longitudinal reinforcement (~21.5” (54.6 cm)) based on moments at column-joint interface. Principal tension and compression were calculated from horizontal and vertical joint stresses based on a 45-degree dispersion of cap beam and column axial forces into the joint.

Failure was initiated by buckling of column longitudinal reinforcement at a displacement ductility of about 4.0. The column spiral began to rupture below the point where spiral spacing changed to 2” (5.1 cm) and there was little additional joint damage. Further cycles at greater ductility resulted in the spiral being pulled through the column and longitudinal reinforcement buckling on both sides of the column.

Joint deformation contributed very little to overall specimen displacement, on the order of no more than 2-3% in both directions.

RESULTS/CONCLUSIONS:

The joint behaved well due to correction of the deficient column longitudinal bar embedment conditions. The haunch provided the necessary embedment length and the additional horizontal and vertical reinforcement in the joint allowed for better force transfer and confinement in the joint. Joint deformations contributed very little to overall displacements and therefore could possibly have been ignored.

Bibliography:

Ingham, Jason M., M.J. Nigel Priestley, Frieder Seible. "Seismic Performance of Bridge Knee Joints - Volume II: Circular Column/Cap Beam Experimental Results (Report No. SSRP - 94/17)." La Jolla, CA: Structural Systems Research, University of California, San Diego, 1994.

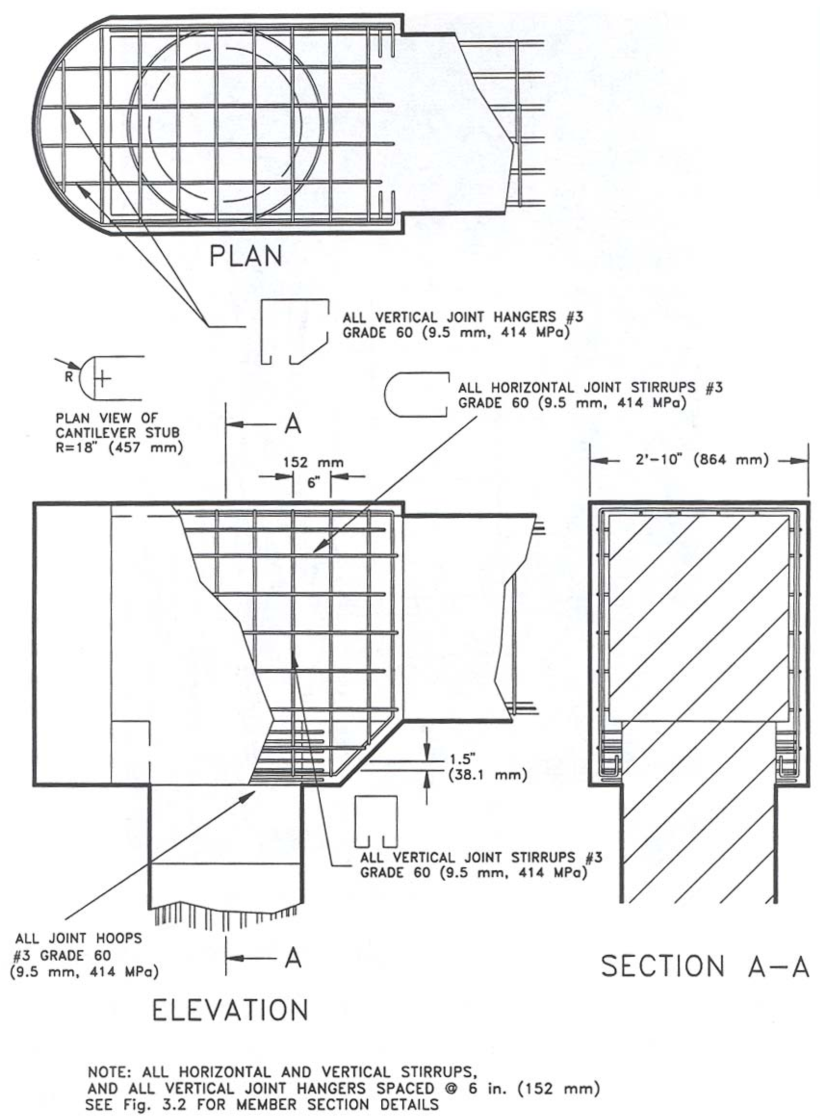


Figure 1 - Elevation and Plan of Knee-Ing-RND-6

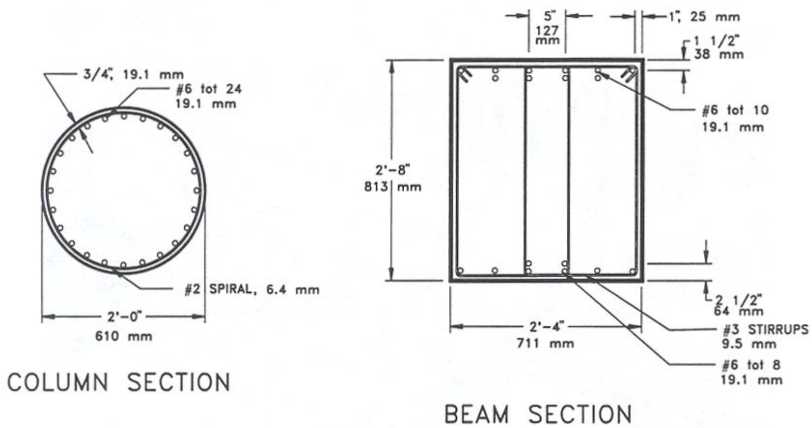


Figure 2 – Cross Sections

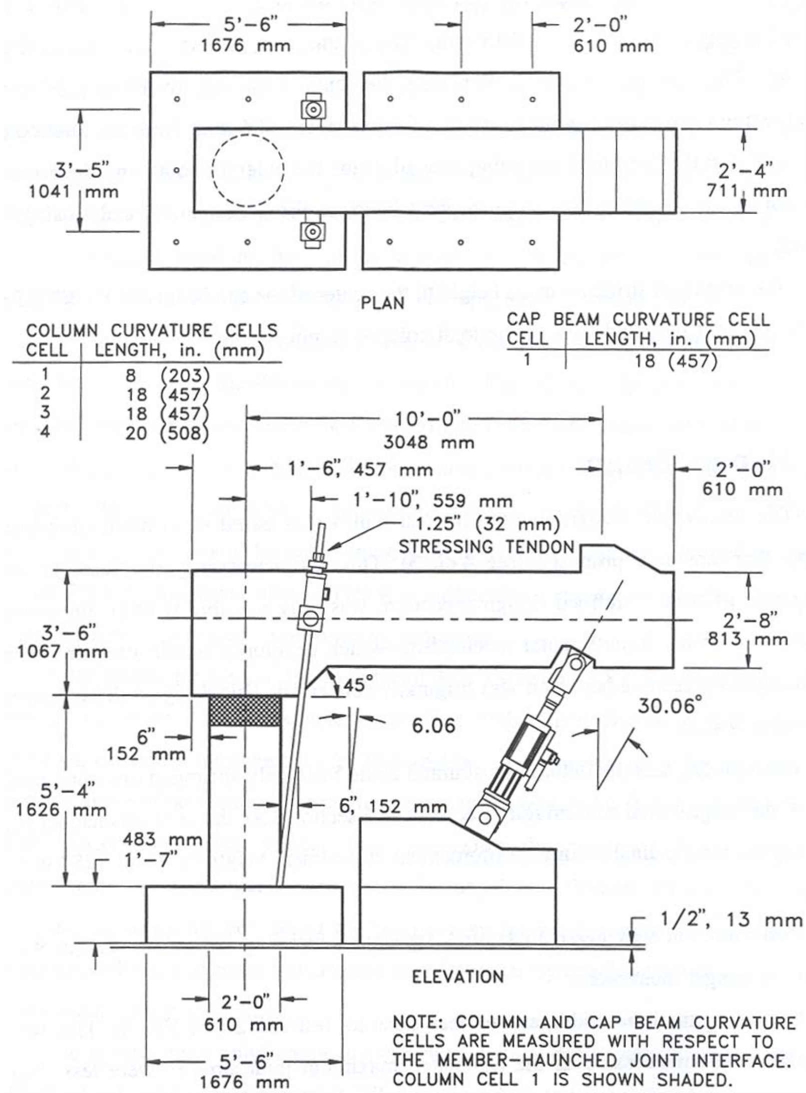


Figure 3 - Specimen Setup

Reference: Knee-Ing-RND-6
Ingham, et al, UCSD (SSRP 94/17)

Type: Knee, as-built of 1987 design (I-105 Bent #31)
Retrofit?: Y Retrofit type: add R/C jacket and haunch to as-built joint.
Column: Circular - ρ_l = 2.33%
Joint: as-built with 6” additional width, 3” (7.6 cm) height, 10” (25.4 cm) haunch, and semi-circular stub
Scale: 1/3
Testing Protocol: Pseudo-static, early cycles under load control, later by displacement control

TEST OBJECTIVE:

This specimen was a suggested retrofit design for Specimen #3 described in the cited report. The retrofit was designed to address the same deficiencies as in Knee-Ing-RND-4 and testing was intended to test the performance of the retrofit under cyclic loading.

TEST SCOPE:

The specimen was loaded with dead load before cyclic loading began. Initial cyclic loading was applied under force control with cycles up to approximate flexural yielding. Beyond this level, load was applied under displacement control with increasing levels of displacement ductility (up to a maximum of μ_Δ=-6.0 closing, +4.6 opening).

DESIGN PHILOSOPHY:

The retrofit of Knee-Ing-RND-6 was designed to increase confinement on the joint core, increase joint area, and improve the embedment conditions for column longitudinal bars. The concrete jacket was designed to help restrain beam bars from lifting out of the joint and to improve confinement of beam bar tails in the back of the joint. The additional vertical reinforcement in the jacket was also intended to help transfer the column tension force in the joint-opening direction. Horizontal reinforcement in the jacket was added for redundancy. The retrofit design assumed all reinforcement would yield in the final testing stages (confirmed in the test).

SPECIMEN DETAILS:

f_c’: Column = 5890 psi (40.6 MPa)
Cap Beam = 5100 psi (35.2 MPa)
Joint = 5100 psi (35.2 MPa)
Retrofit Jacket = 4900 psi (33.8 MPa)

COLUMN:
24” (61 cm) diameter circular column - $\rho_l = 2.33\%$
L_c = 7’-6” (2.29 m) top of footing to cap beam centroid (modeled 43% of prototype height)
Longitudinal Reinforcement:
  24 #6, straight cut off 10” (25.4 cm) from top surface of cap beam
Transverse Reinforcement:
  #2 spiral @2” (5.1 cm) pitch in lower portion - $\rho_t = 0.44\%$
  #2 spiral @ $1^1/_3$” (3.4 cm) in 2 feet below joint and in 22” (55.9 cm) of joint - $\rho_t = 0.66\%$
Axial Load: 4.02% Ag**fc’* constant gravity load (total axial load varied with lateral load)

BEAM:
Rectangular - 2’-4”b x 2’-8”h (71.1cm x 81.3cm)
L_b=12’ (3.66 m) to column centerline
Longitudinal Reinforcement:
  Top: outer layer 6 #6, inner layer 4 #6 (total 10)
  Bottom: outer layer 6 #6, inner layer 2 #6 (total 8)
  6 bars on top of section and bottom of section detailed with vertical tails welded together at cap-beam mid-height with full penetration butt welds
Transverse Reinforcement:
  closed 4-leg #3 stirrups @ 4” in remainder of beam (1^{st} at 4” (10.2 cm) from col. face)
Skin Reinforcement:
  None

JOINT:
Vertical Reinforcement:
  Open-ended 2-legged #3 stirrups @ 4” (10.2 cm)
Horizontal Reinforcement:
  #2 spiral @ $1^1/_3$” (3.4 cm) (continuation of column spiral) to 10” (25.4 cm) from cap beam top surface (22” (55.9 cm) embedment) - $\rho_t = 0.66$ %
  Also, 2-legged #3 stirrups @ $4^1/_2$” (11.4 cm)

CONCRETE JACKET:
Added 3” width to each side of joint, resulting in a total joint width of 34” (86.4 cm)
Curved stub added to back of joint w/ 18” (45.7 cm) radius
  Horizontal #3 joint stirrups @ 6” (15.2 cm) (drilled into joint on cap-beam side, secured with epoxy)
  Horizontal #3 joint hoops @ $1^1/_2$” (3.8 cm) around column inside haunch (below cap beam)
  Vertical #3 2-legged joint stirrups @ 6” (15.2 cm)
  Vertical #3 joint hangers @ 6” (15.2 cm) (parallel to cap beam, to help form reinforcement cage)

QUANTIFIED RESPONSE:

stress quantities in ***psi*** **(*MPa*)**	Open	Close
Max Joint Shear Stress	$9.4\sqrt{f_c'}$ $(0.78\sqrt{f_c'})$	$8.6\sqrt{f_c'}$ $(0.71\sqrt{f_c'})$
Joint Strain at Max Shear Stress	0.00053 rad.	0.00048 rad.
Max Joint Shear Strain	0.0012 rad.	0.0006 rad.
Joint Stress at Max Shear Strain	$8.8\sqrt{f_c'}$ $(0.73\sqrt{f_c'})$	$7\sqrt{f_c'}$ $(0.58\sqrt{f_c'})$
Max Principal Tension	$10\sqrt{f_c'}$ $(0.83\sqrt{f_c'})$	$8\sqrt{f_c'}$ $(0.66\sqrt{f_c'})$
Max Principal Compression	$10\sqrt{f_c'}$ $(0.83\sqrt{f_c'})$	$8.5\sqrt{f_c'}$ $(0.71\sqrt{f_c'})$
% Deformation due to joint	2%	<2%

FAILURE MODE:

Initially, buckling of column longitudinal reinforcement about 12" (30.5 cm) below column-joint interface. Final failure due to rupture of column spirals and further buckling of column longitudinal reinforcement. Buckled longitudinal bars ruptured under reversed loading.

HYSTERESIS DESCRIPTION:

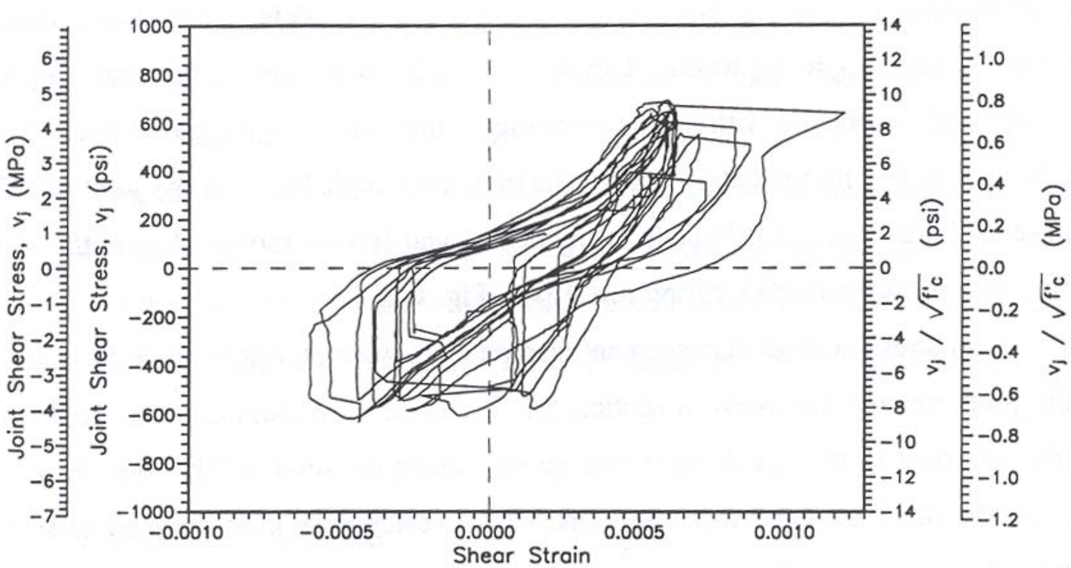


Figure 4 - Joint Shear Stress vs. Shear Strain for Knee-Ing-RND-6

Joint shear strain data was not particularly clean in this test since the joint remained mostly elastic for the duration of the test and thus deformations were very small. Some pinching of the shear stress-strain relationship is present but the report's authors describe an approximately linear relationship due to the small strains.

Shear stress calculated based on full joint width (34" (86.4 cm)) and full distance between column reinforcement (~22$^1/_2$" (57.2 cm)), based on moments at cap beam-joint interface. Principal tension and compression were calculated from horizontal and vertical joint stresses based on a 45 degree dispersion of cap beam and column axial forces into the joint.

Test ended with the rupture of two transverse column spirals and the buckling of several column longitudinal bars.

Joint deformations contributed, on average, 2% of total beam end displacement in the opening direction and less than 2% in the joint closing direction at all deformation levels. Column deformations were by far the largest component of the measured total beam displacement. Joint deformations could have been ignored in this test.

RESULTS/CONCLUSIONS:

This specimen had excellent behavior as compared to the original design due to the correction of embedment and confinement problems. The haunch, as in Knee-Ing-RND-4 improved the embedment conditions of the column longitudinal bars and prevented their pullout under cyclic loading. The concrete jacket improved joint confinement and increased joint area, both of which contributed to the small deformations measured in the test. The shear stress-strain relationship was nominally elastic and joint shear deformations contributed very little to overall system behavior.

Bibliography:

Ingham, Jason M., M.J. Nigel Priestley, Frieder Seible. "Seismic Performance of Bridge Knee Joints - Volume II: Circular Column/Cap Beam Experimental Results (Report No. SSRP - 94/17)." La Jolla, CA: Structural Systems Research, University of California, San Diego, 1994.

R/C Bridge Beam-Column Joints
DD-Maz-T-1, DD-Maz-L-1

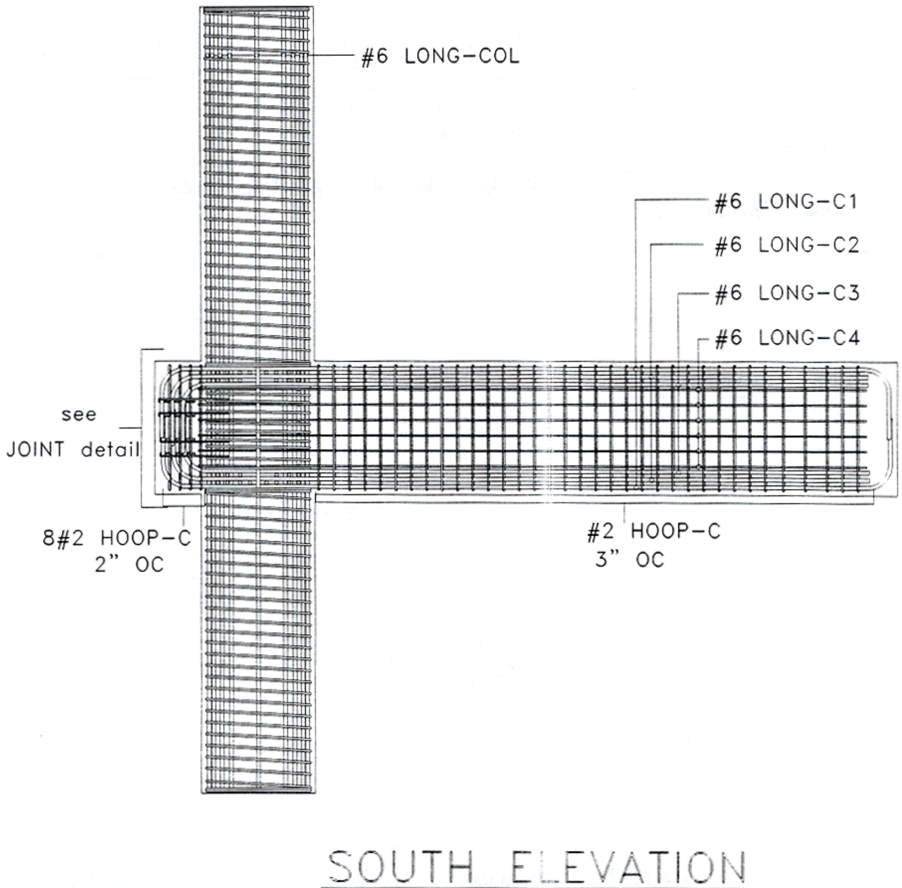


Figure 1 - Elevation of Transverse Bent

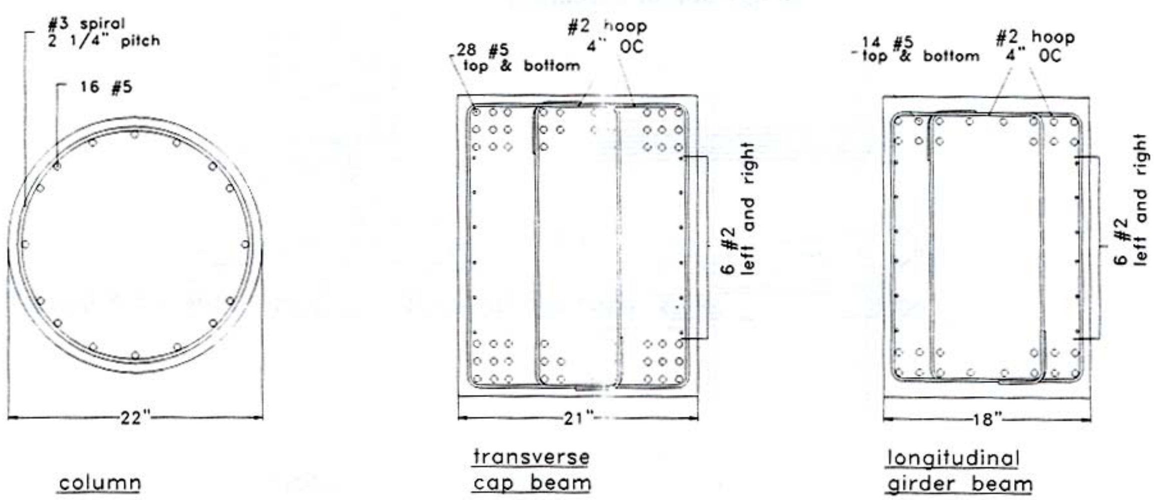


Figure 2 - Cross Sections of Column and Beams

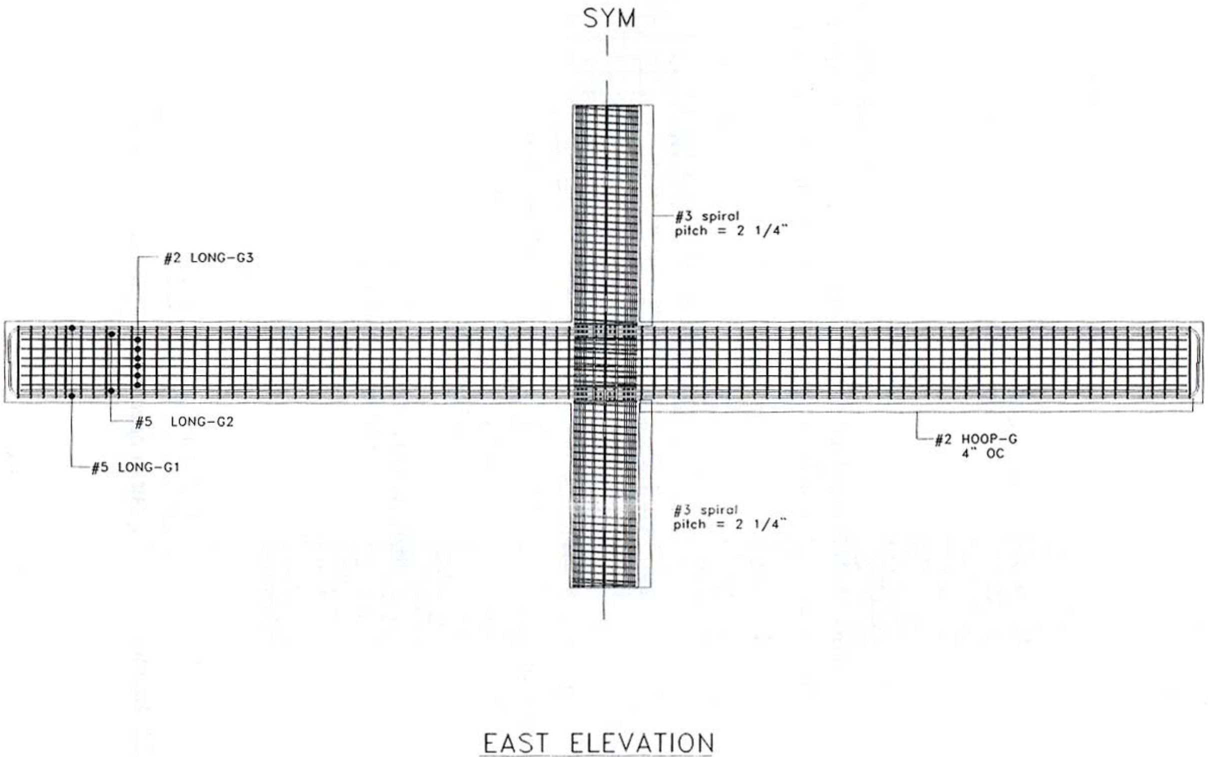


Figure 3 - Elevation of Longitudinal Bent

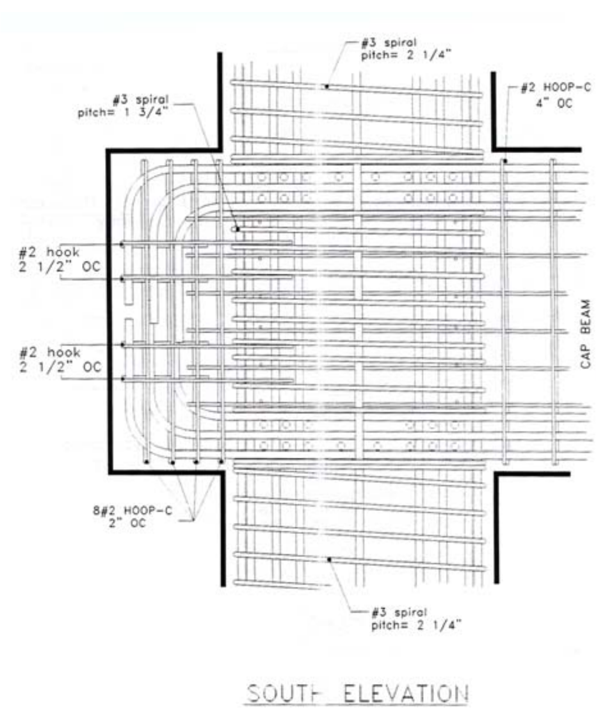


Figure 4 – Joint Elevation (Transverse)

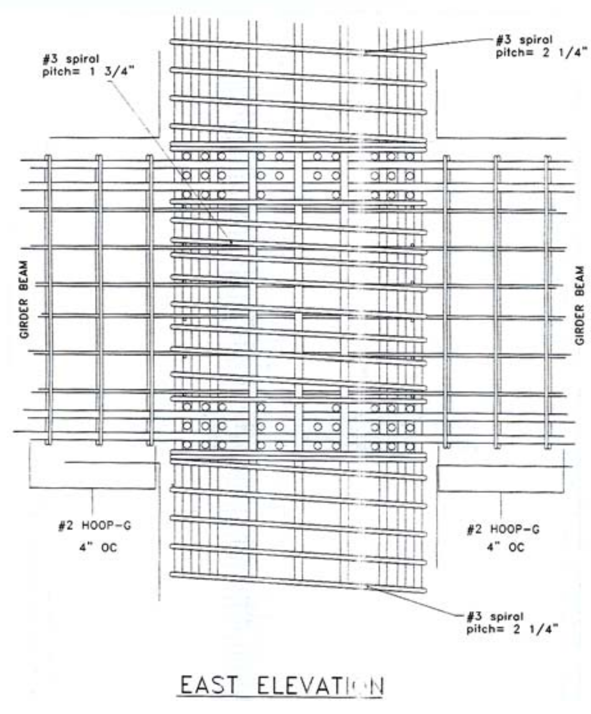


Figure 5 – Joint Elevation (Longitudinal)

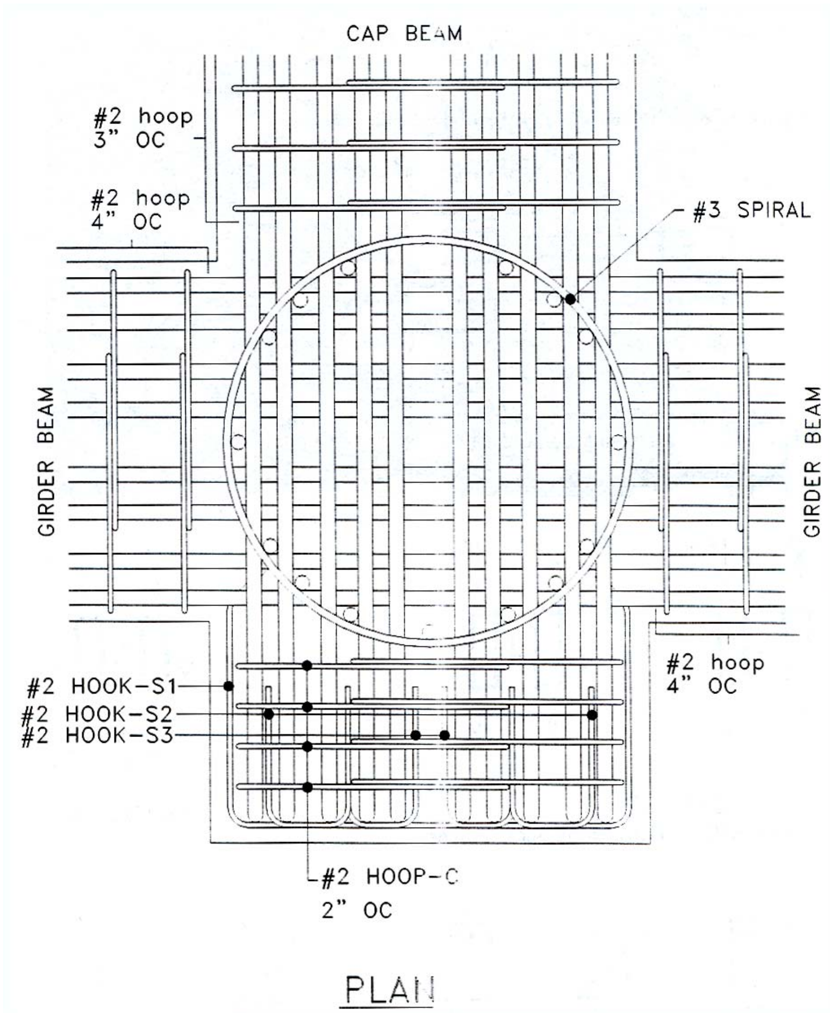


Figure 6 – Plan View of Joint

Reference: DD-Maz-T-1
Mazzoni (Ph.D. Dissertation)

Type: Double-Deck
Column: Round
Scale: 1/3
Testing Protocol: Pseudo-static, under displacement control

TEST OBJECTIVE:

To gain a better understanding of the mechanics and behavior of beam column-joints in the lower-level of reinforced concrete double-deck bridges under uni- and bi-directional cyclic loading. The effect of horizontal and vertical transverse reinforcement in the joint, the effect of column axial load on response, and anchorage of column and beam longitudinal reinforcement were the main parameters studied in the test.

TEST SCOPE:

Cyclic loading was applied independently in the transverse and longitudinal directions and then in a circular pattern intended to test the bi-directional performance of the specimen. Loading was applied to stress the beam-column joint to the limits recommended limits by ACI-352 and the specimen's behavior at this loading level was evaluated.

DESIGN PHILOSOPHY:

DD-Maz-1 was designed to test the joint at the shear stress limits recommended by ACI-352 and column steel was chosen to produce this target stress when the column critical section reached its maximum moment strength. To prevent yielding in the cap and transverse beams, beam moment strength was designed to exceed the demands imposed by the column plastic moment capacity. A joint force transfer mechanism was designed to accommodate the demands placed on the joint assuming a combination of both the truss and compression strut models of joint behavior. Minimal damage was another goal of the design.

SPECIMEN DETAILS:

f_c':	Lower Column =	5480 psi (37.8 MPa)
	Upper Column =	5140 psi (35.4 MPa)
	Cap Beam =	5750 psi (39.6 MPa)
	Joint =	5750 psi (39.6 MPa)

COLUMN:
22" (55.9 cm) diameter spirally reinforced
L_c = 4'-11" (1.50 m) below beam, 5'-10" (1.78 m) above beam, ¾" (1.9 cm) cover
Longitudinal Reinforcement:
16 #5, continuous through joint (no splices along height of column) - $\rho_l = 1.3\%$
Transverse Reinforcement:
#3 spiral @ 2 ¼ " (5.7 cm) pitch - $\rho_s = 0.95\%$
Axial Load: ~5.50 % Ag**fc'* constant gravity load (total axial load varied with lateral load)

TRANSVERSE BEAM:
Rectangular - 1'-9"b x 2'-2"h (53 cm x 66 cm); ¾" (1.9 cm) cover
L_b=9'-7½" (2.93 m) to joint face
11" (28 cm) stub on outer face of joint
Longitudinal Reinforcement:
Top: 28 #5 in 3 layers spaced 1½" (3.8 cm)
Bottom: 28 #5 in 3 layers spaced 1½" (3.8 cm)
Transverse Reinforcement:
4-leg #2 hoops @ 4" (10.2 cm)
Skin Reinforcement:

6 #2 each side

LONGITUDINAL BEAM:
Rectangular - 1'-6"b x 2'-2"h (46 cm x 66 cm); ½" (1.3 cm) cover on sides,
1¼" (3.2 cm) cover top & bottom
L_b=16' (4.91 m) to column centerline
Longitudinal Reinforcement:
Top: 14 #5 in 2 layers
Bottom: 14 #5 in 2 layers
Transverse Reinforcement:
4-leg #2 hoops @ 4" (10.2 cm)
Skin Reinforcement:
6 #2 each side

JOINT:
Vertical Reinforcement:
8 4-leg #2 hoops @ 2" (5.1 cm) in stub (none in joint)
Horizontal Reinforcement:
#3 spiral @ 1 ¾" (4.4 cm) - ρ_s = 1.2 %
4 10-leg #2 hooks (hold hooks of beam longitudinal steel in stub)

QUANTIFIED RESPONSE:

TRANSVERSE DIRECTION

stress quantities in ***psi*** **(*MPa)***	Open	Close
Max Joint Shear Stress	$11.9\sqrt{fc'}$ $(0.99\sqrt{f_c'})$	$14.4\sqrt{fc'}$ $(1.20\sqrt{f_c'})$
Joint Strain at Max Shear Stress	0.00326 rad	0.00295 rad
Max Joint Shear Strain	0.0036 rad.	0.0033 rad.
Joint Stress at Max Shear Strain	$11\sqrt{f_c'}$ $(0.91\sqrt{f_c'})$	$12\sqrt{fc'}$ $(1.00\sqrt{f_c'})$
Max Principal Tension	---	---
Max Principal Compression	---	---
% Deformation due to joint	12%	12%

LONGITUDINAL DIRECTION

stress quantities in psi (MPa)	Push	Pull
Max Joint Shear Stress	$15\sqrt{fc'}$ $(1.25\sqrt{fc'})$	$14.6\sqrt{fc'}$ $(1.21\sqrt{fc'})$
Joint Strain at Max Shear Stress	0.0059 rad	0.0041 rad
Max Joint Shear Strain	0.0059 rad.	0.0041 rad.
Joint Stress at Max Shear Strain	$15\sqrt{f_c'}$ $(1.25\sqrt{f_c'})$	$13.5\sqrt{fc'}$ $(1.12\sqrt{f_c'})$
Max Principal Tension	---	---
Max Principal Compression	---	---
% Deformation due to joint	15%	9%

FAILURE MODE:

Initially, buckling of longitudinal reinforcement in the upper column during 8-inch amplitude cycles, followed by fracture under reversed loading at larger displacements. Several more bars buckled and then fractured at the largest displacement level in the transverse direction and the test was then terminated.

HYSTERESIS DESCRIPTION:
(Results of Longitudinal and Transverse loading presented together)

This specimen was subjected to a complex loading pattern including unidirectional cycles in both the longitudinal and transverse directions followed by bi-directional (circular) cycles at increasing levels of displacement.

Shear stress was calculated based on full joint length equal to column dimension (22" (55.9 cm)) and width equal to $h_c*\sqrt{(^{\pi}/_4}$ (provides equivalent rectangular area, =19.5" (49.5 cm)) based on moments at column-joint interface.

Damage during test cycles was concentrated in the plastic hinge regions of the columns but more so in the top than in the bottom. Upper column damage occurred over a length of roughly 1.5 times the column diameter and included spalling and cracking while lower column damage was limited to spalling over roughly one column diameter. Cracking initiated on the beam faces at the joint corners at larger displacements and fanned out with increasing displacement. Cracking then appeared in the lower column plastic hinge region.

Failure occurred with the buckling and subsequent fracture of column longitudinal bars in the top column. No bars fractured in the lower column. Both upper and lower columns significant experienced spalling in the plastic hinge regions adjacent to the joint. Testing was terminated during the first transverse cycle at 12” (30.5 cm) column tip displacement when significant loss in lateral strength occurred.

Transverse

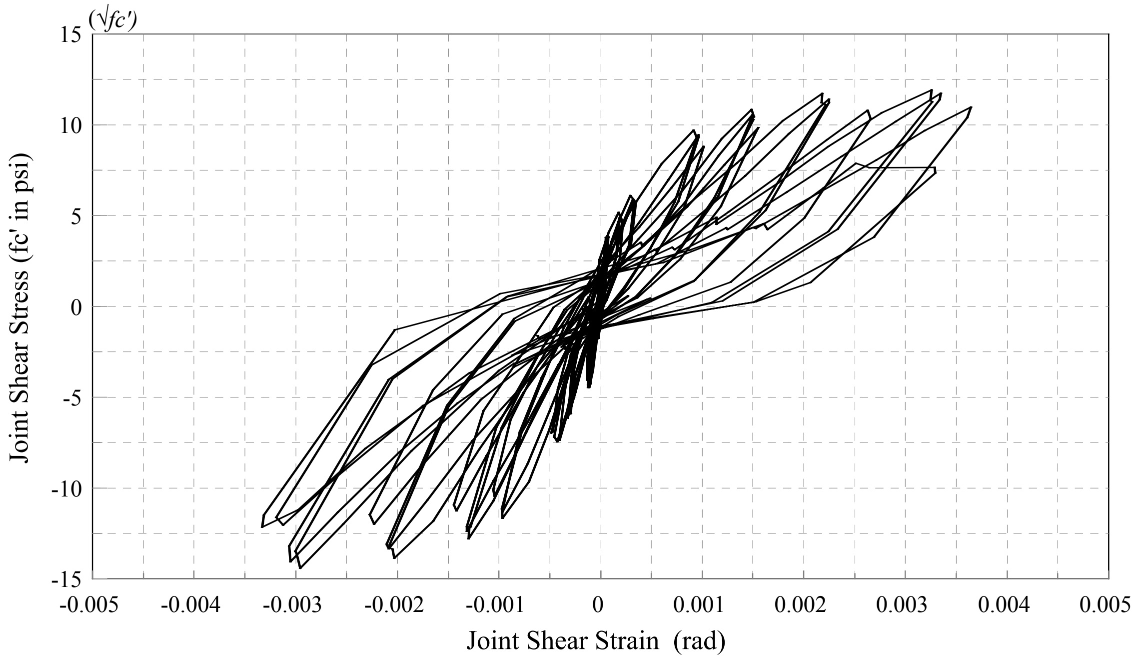


Figure 7 - Joint Shear Stress vs. Shear Strain for DD-Maz-T-1

For transverse loading, joint deformation was calculated to be responsible for an average of 12% of total specimen deformation in both push and pull directions. This level of contribution is significant and should not be ignored in deformation calculations.

Longitudinal:

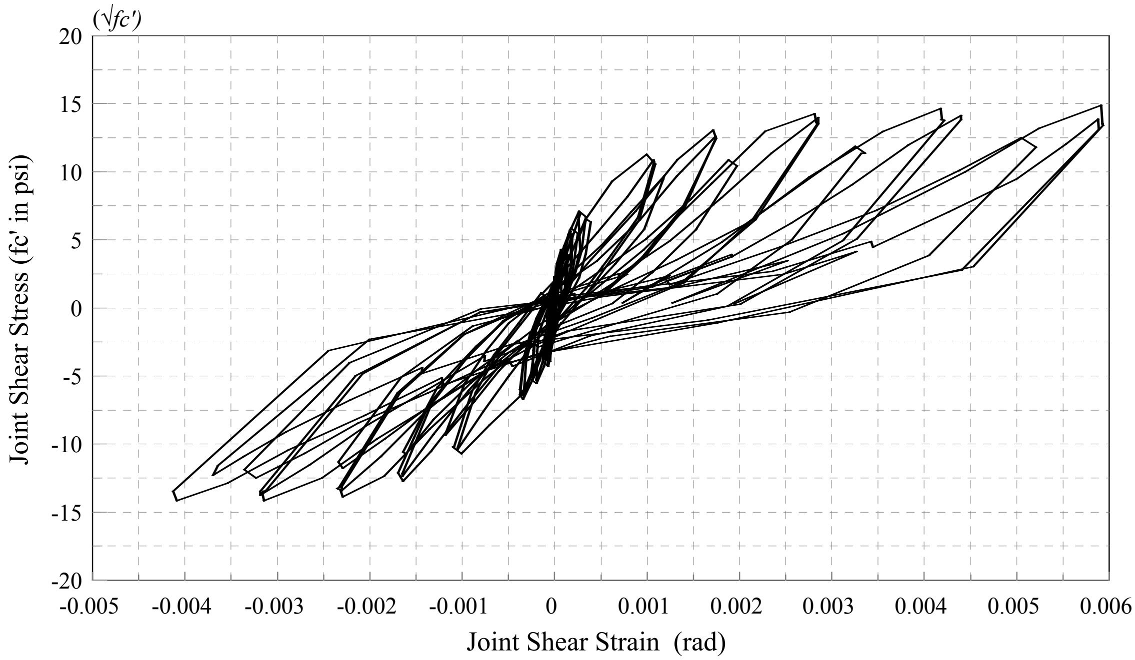


Figure 8 - Joint Shear Stress vs. Shear Strain for DD-Maz-L-1

For longitudinal loading, joint deformation was calculated to be responsible for 3% of total deformation at lower levels to as much as 15% before serious damage began to accumulate in the column plastic hinges. This level of contribution is significant and should not be ignored in calculating system behavior

No significant loss of joint shear strength is seen until very high displacements and stiffness degradation is not excessive. The hysteresis is pinched but does dissipate some energy, especially at larger cycles. For transverse loading, the joint is “stronger” in the closing direction most likely due to higher axial load (and thus strength) in the column that results in larger demands on the joint. Regardless, the joint performed well. The joint behaves symmetrically for longitudinal loading in keeping with the constant axial load and therefore demand on the joint.

RESULTS/CONCLUSIONS:

The joint appears to have behaved well with no serious degradation of strength or stiffness. Damage is localized in the column plastic hinges and the joints hold together well. Joint deformation accounts for a significant portion of system deformation in both loading directions and cannot be ignored for this system despite the otherwise admirable performance. The confining action of beams framing into three sides of the joint and a stub on the fourth side probably contributed to this performance.

BIBLIOGRAPHY:

Mazzoni, Silvia. Design and Response of Lower-Level Beam-Column Joints in Ductile Reinforced-Concrete Double-Deck Bridge Frames Berkeley, CA: University of California, Berkeley, 1997.

R/C Bridge Beam-Column Joints
DD-Maz-T-2, DD-Maz-L-2

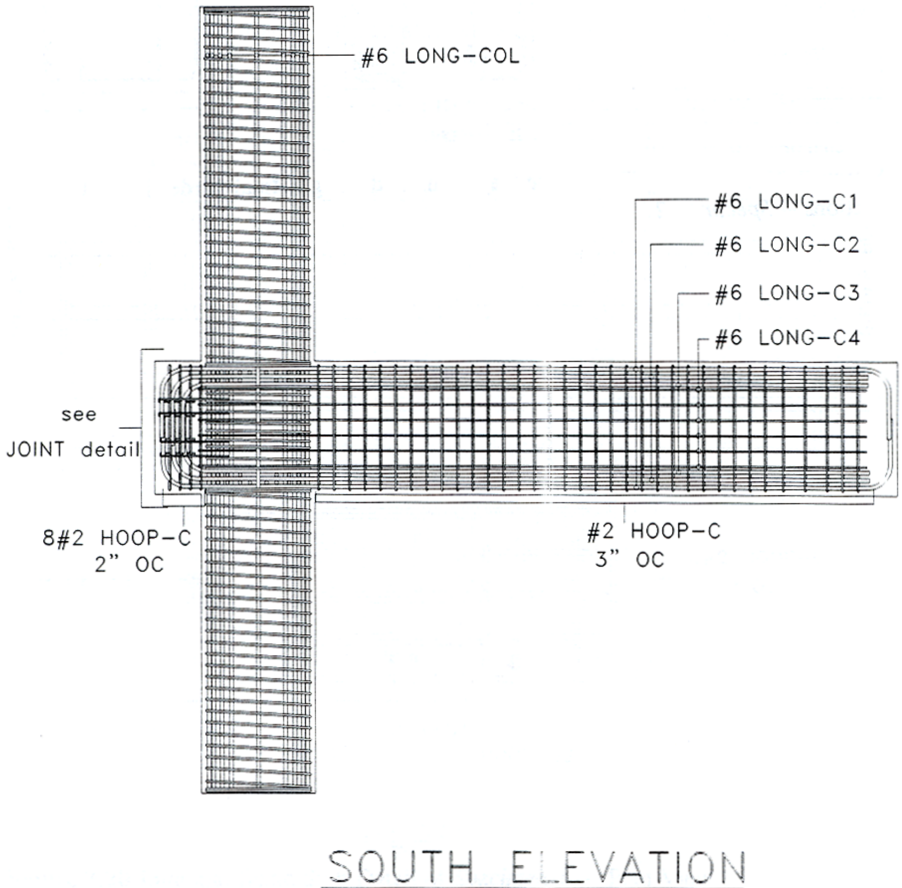


Figure 1 - Elevation of Transverse Bent

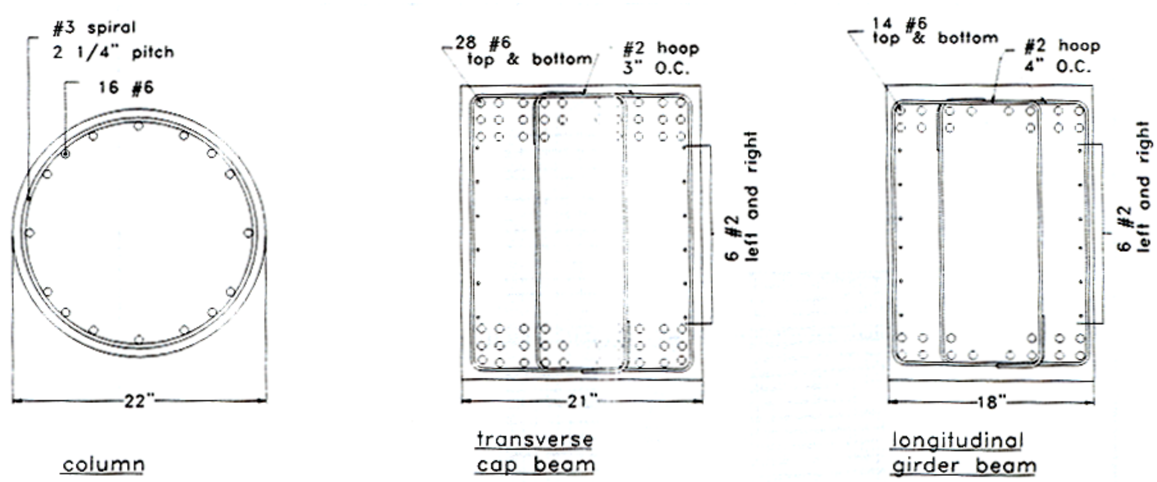


Figure 2 - Cross Sections of Column and Beams

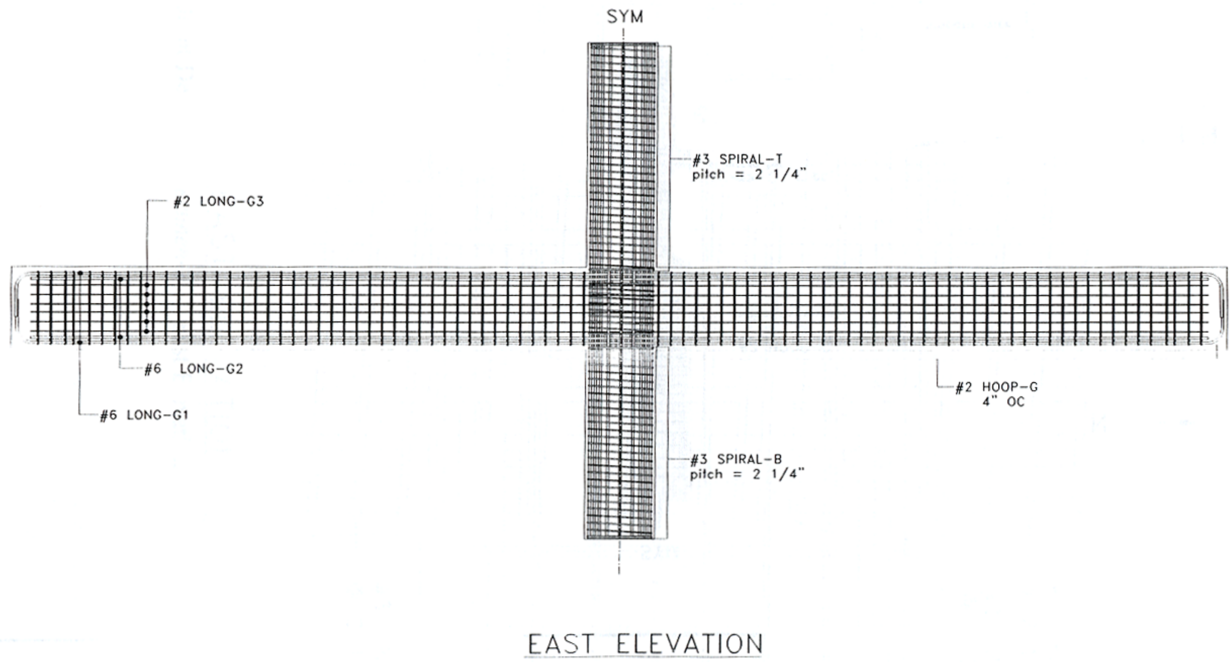


Figure 3 - Elevation of Longitudinal Bent

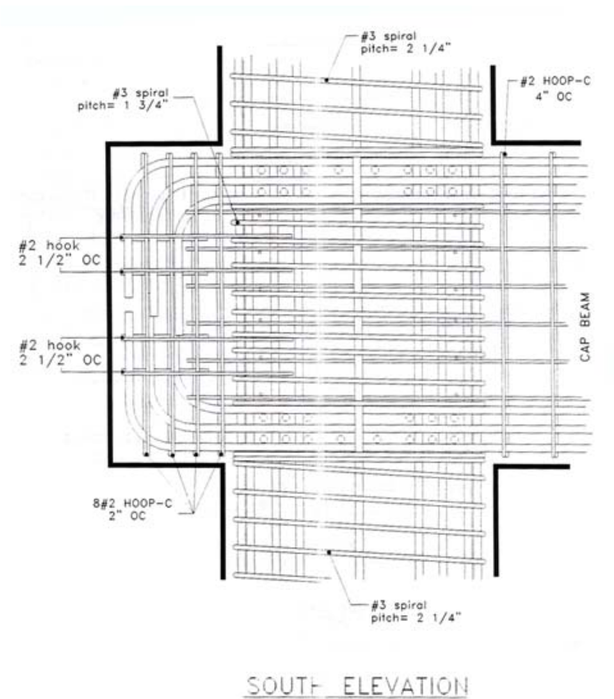


Figure 4 - Joint Elevation (Transverse)

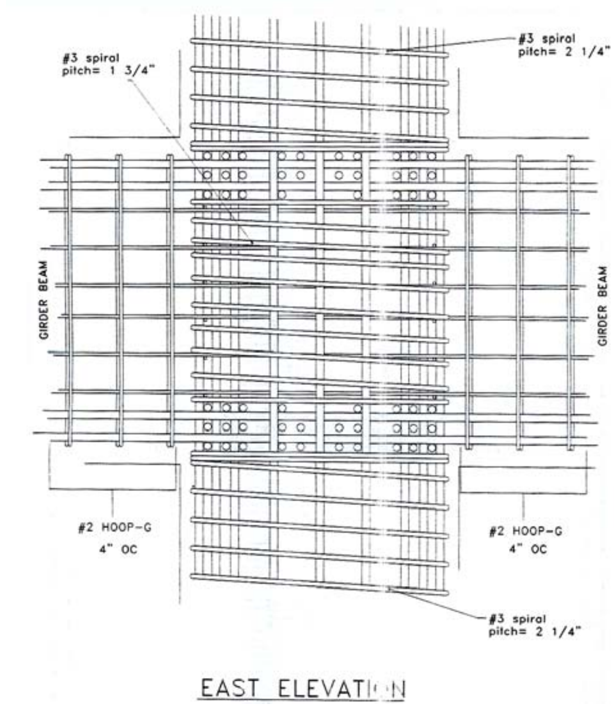


Figure 5 - Joint Elevation (Longitudinal)

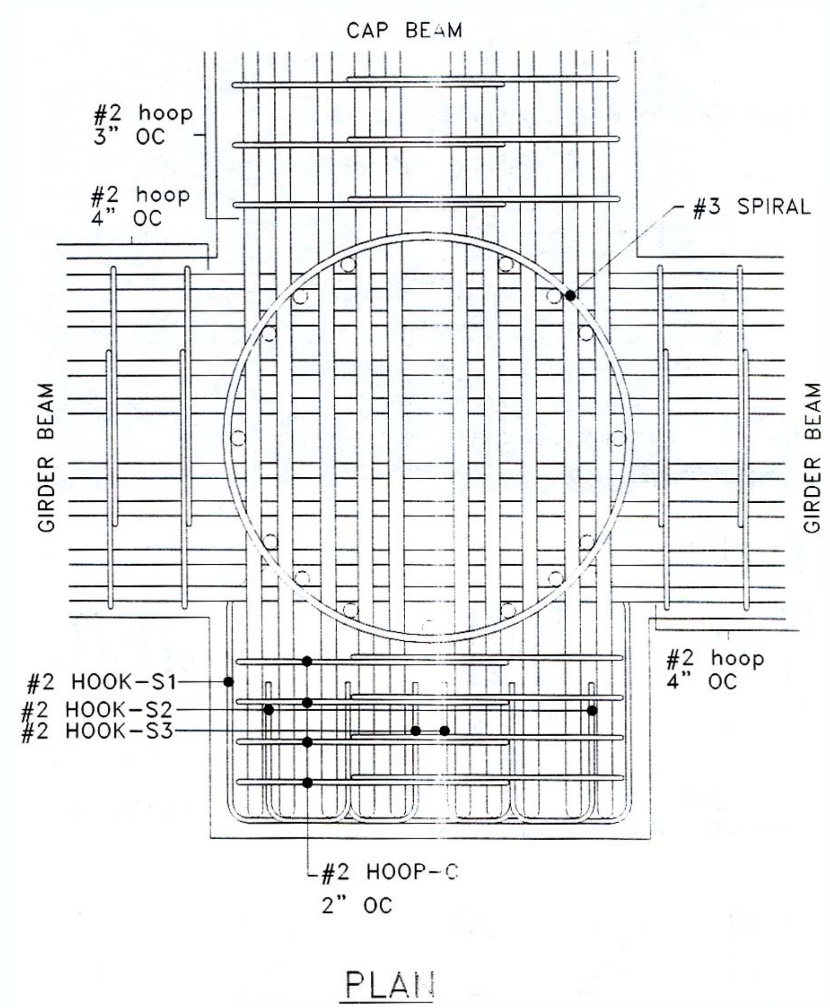


Figure 6 - Plan View of Joint

Reference: DD-Maz-T-2, DD-Maz-L-2
Mazzoni (Ph.D. Dissertation)

Type: Double-Deck
Column: Round
Scale: 1/3
Testing Protocol: Pseudo-static, under displacement control

TEST OBJECTIVE:

To gain a better understanding of the mechanics and behavior of beam column-joints in the lower-level of reinforced concrete double-deck bridges under uni- and bi-directional cyclic loading large enough to "overstress" the joint. The effect of horizontal and vertical transverse reinforcement in the joint, the effect of column axial load on response, and anchorage of column and beam longitudinal reinforcement were the main parameters studied in the test.

TEST SCOPE:

Cyclic loading was applied independently in the transverse and longitudinal directions and then in a circular pattern intended to test the bi-directional performance of the specimen. Loading was applied to stress the beam-column joint beyond the limits recommended limits by ACI-352 and the specimen's behavior at this loading level was evaluated.

DESIGN PHILOSOPHY:

DD-Maz-2 was designed to test the joint at a target shear stress greater than that recommended by ACI-352 and column steel was chosen to produce this target stress when the column critical section reached its maximum moment strength. (#6 instead of #5 bars were used in the column) To prevent yielding in the cap and transverse beams, beam moment strength was designed to exceed the demands imposed by the column plastic moment capacity. (again, #6 instead of #5 bars) A joint force transfer mechanism was designed to accommodate the demands placed on the joint assuming a combination of both the truss and compression strut models of joint behavior.

SPECIMEN DETAILS:

f_c':	Lower Column =	6160 psi (42.5 MPa)
	Upper Column =	5530 psi (38.1 MPa)
	Cap Beam =	5870 psi (40.5 MPa)
	Joint =	5870 psi (40.5 MPa)

COLUMN:
22" (55.9 cm) diameter spirally reinforced
L_c = 4'-11" (1.50 m) below beam, 5'-10" (1.78 m) above beam, ¾" (1.9 cm) cover
Longitudinal Reinforcement:
    16 #6, continuous through joint (no splices along height of column) - ρ_l = 1.9%
Transverse Reinforcement:
    #3 spiral @ 2 ¼ " (3.2 cm) pitch - ρ_s = 0.95%
Axial Load: ~5.60 % Ag**fc'* constant gravity load (total axial load varied with lateral load in Transverse direction only)

TRANSVERSE BEAM:
Rectangular - 1'-9"b x 2'-2"h (53 cm x 66 cm); ¾" (1.9 cm) cover
L_b=9'-7½" (2.93 m) to joint face
11" (28 cm) stub on outside face of joint
Longitudinal Reinforcement:
    Top: 28 #6 in 3 layers spaced 1½" (3.8 cm)
    Bottom: 28 #6 in 3 layers spaced 1½" (3.8 cm)
Transverse Reinforcement:
    4-leg #2 hoops @ 3" (7.6 cm)
Skin Reinforcement:

6 #2 each side

LONGITUDINAL BEAM:
Rectangular - 1’-6”b x 2’-2”h (46 cm x 66 cm); ½” (1.3 cm) cover on sides,
1¼” (3.2 cm) cover top & bottom
L_b=16’ (4.91 m) to column centerline
Longitudinal Reinforcement:
Top: 14 #6 in 2 layers
Bottom: 14 #6 in 2 layers
Transverse Reinforcement:
4-leg #2 hoops @ 4” (10.2 cm)
Skin Reinforcement:
6 #2 each side

JOINT:
Vertical Reinforcement:
8 #2 4-leg #2 hoops @ 2” (5.1 cm) in stub (none in joint)
Horizontal Reinforcement:
#3 spiral @ 1 ¾” (4.4 cm) pitch - ρ_s = 1.2 %
4 10-leg #2 hooks (hold hooks of beam longitudinal steel in stub)

QUANTIFIED RESPONSE:

TRANSVERSE DIRECTION

(stress quantities in psi)	Open	Close
Max Joint Shear Stress	$15.19\sqrt{f_c'}$ ($1.26\sqrt{f_c'}$)	$18.88\sqrt{f_c'}$ ($1.57\sqrt{f_c'}$)
Joint Strain at Max Shear Stress	0.0031 rad	0.0045 rad
Max Joint Shear Strain	0.02 rad.	0.01 rad.
Joint Stress at Max Shear Strain	$11.4\sqrt{f_c'}$ ($0.95\sqrt{f_c'}$)	$17.0\sqrt{fc'}$ ($1.41\sqrt{f_c'}$)
Max Principal Tension	---	---
Max Principal Compression	---	---
% Deformation due to joint	15%	20%

LONGITUDINAL DIRECTION

(stress quantities in psi)	Push	Pull
Max Joint Shear Stress	$17.6\sqrt{f_c'}$ $(1.46\sqrt{f_c'})$	$17.0\sqrt{f_c'}$ $(1.41\sqrt{f_c'})$
Joint Strain at Max Shear Stress	0.0058 rad	0.0054 rad
Max Joint Shear Strain	0.02 rad.	0.01 rad.
Joint Stress at Max Shear Strain	$15\sqrt{f_c'}$ $(1.25\sqrt{f_c'})$	$13.5\sqrt{fc'}$ $(1.12\sqrt{f_c'})$
Max Principal Tension	---	---
Max Principal Compression	---	---
% Deformation due to joint	23%	18%

FAILURE MODE:

Buckling of longitudinal reinforcement in the lower column during 8-inch (20.3 cm) cycles in the transverse direction. No bars were fractured and the test was terminated after 2 cycles.

HYSTERESIS DESCRIPTION:
(Results of Longitudinal and Transverse loading presented together)
See description of loading pattern for specimen 1.

This specimen was designed to place larger demands on the joint region (as much as $19.4\sqrt{f_c'}$ $(1.61\sqrt{f_c'}$ metric) a maximum of $\sim 15\sqrt{f_c'}$ $(1.25\sqrt{f_c'}$ metric) for specimen 1). Shear was stress calculated based on full joint length equal to column dimension (22" (55.9 cm)) and width equal to $h_c*\sqrt{(\pi/4)}$ (provides equivalent rectangular area, =19.5" (49.5 cm)) based on moments at column-joint interface.

Damage during early test cycles (0.25" (0.64 cm)) initiated at the column-joint boundary with additional cracks appearing along the height of the column at slightly larger (0.5" (1.27 cm)) cycles. Later cycles(3" (7.6 cm)) concentrated damage in the bottom column plastic hinge. Also, inclined cracks appeared in both the columns and beams. As with specimen 1, cracks formed at the corners of the beam-joint interface and fanned out to mid-depth with increasing displacement. Similar cracks were observed on the beam stub and all cracks increased in number and width with increasing displacement.

Bottom column longitudinal bars began to buckle at 8" (20.3 cm) displacement cycles. The test was terminated after two cycles in the transverse direction with no fractured bars.

Transverse:

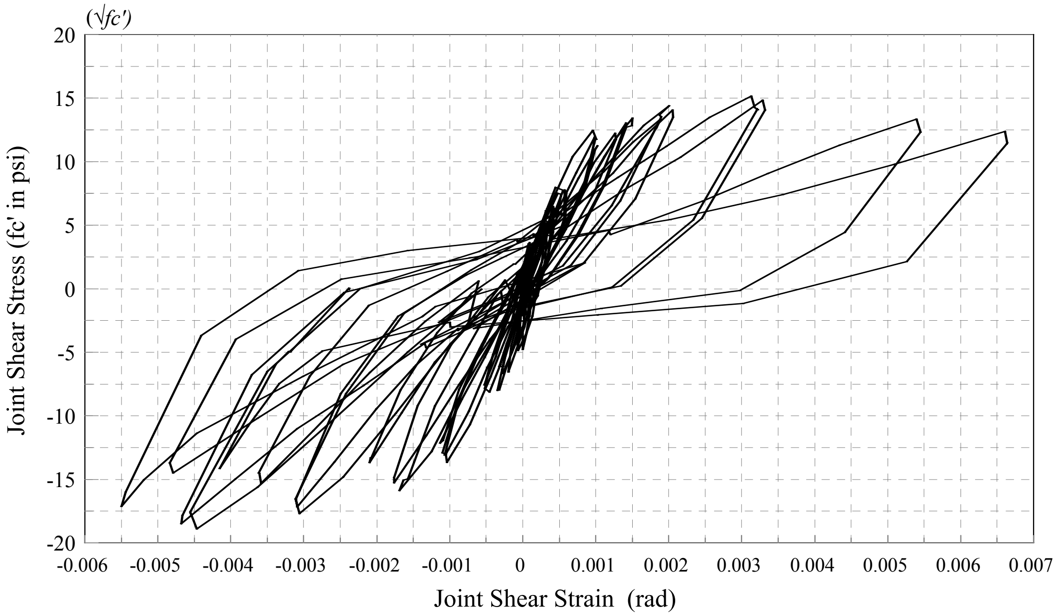


Figure 7 - Joint Shear Stress vs. Shear Strain for DD-Maz-T-2

Joint deformations accounted for as much as 20% of total specimen deformation with an average contribution closer to 10-12% in both the weak and strong transverse directions.

Longitudinal:

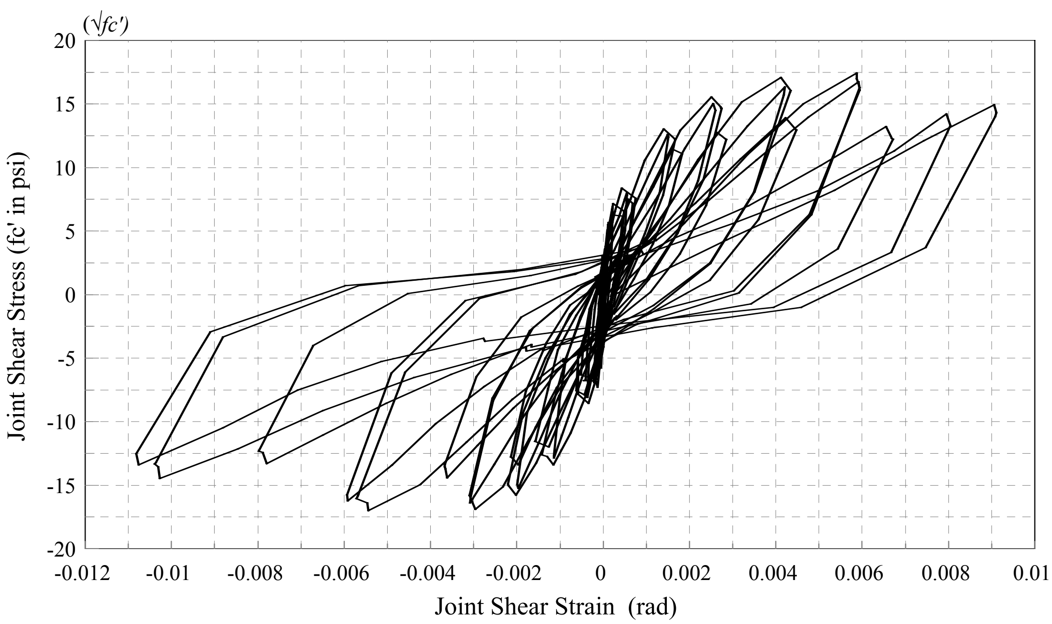


Figure 8 - Joint Shear Stress vs. Shear Strain for DD-Maz-L-2

For longitudinal displacement, joint deformations were responsible for as much as 22-23% of total system displacement an as little as 5% at very low cycles.

The joint displays strength degradation at higher displacement levels not seen in the specimen 1. Therefore, the joint is not able to sustain the stresses associated with the probable column plastic moment. This is due to the higher demands placed on the joint (as opposed to joint 1) but it still sustains demands higher than those prescribed for exterior joints by ACI-ASCE 352 ($15\sqrt{f_c'}$ ($1.25\sqrt{f_c'}$ metric)) for most directions of loading. (weak transverse direction excepted)

RESULTS/CONCLUSIONS:
The joint displays generally good behavior but has poor strength characteristics at larger displacement and stress demands. Hysteretic energy is dissipated especially at larger strains. Joint deformations were responsible for a significant portion of overall displacement and cannot be ignored in this case. The joint held together at much larger strains and stresses than the comparison specimen (DD-Maz-1) but stiffness and strength degradation make this joint's behavior unacceptable.

BIBLIOGRAPHY:
Mazzoni, Silvia. Design and Response of Lower-Level Beam-Column Joints in Ductile Reinforced-Concrete Double-Deck Bridge Frames Berkeley, CA: University of California, Berkeley, 1997.

R/C Bridge Beam-Column Joints
Tee-Nai-RND-A1

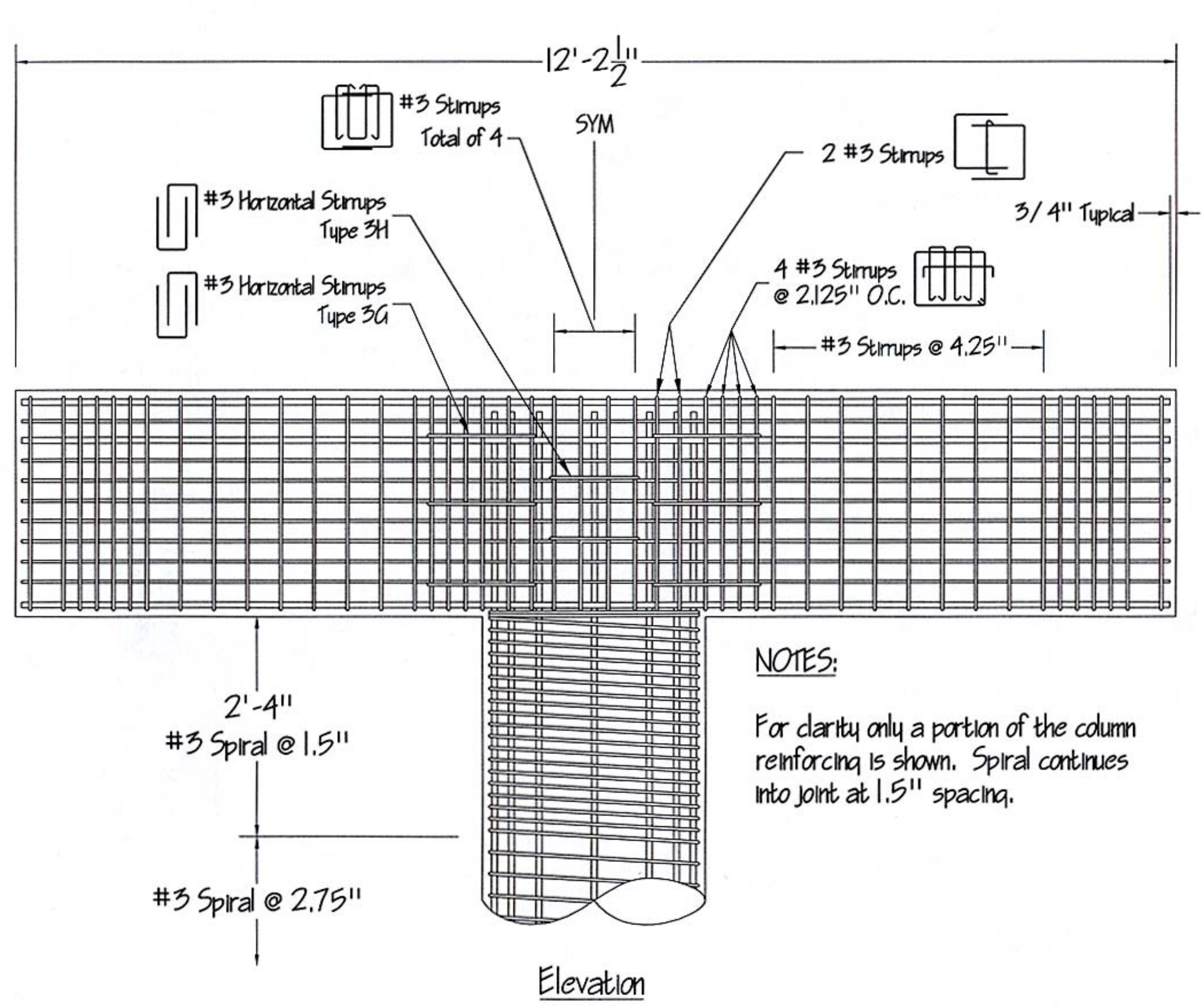


Figure 1 - Elevation of Tee-Nai-RND-A1

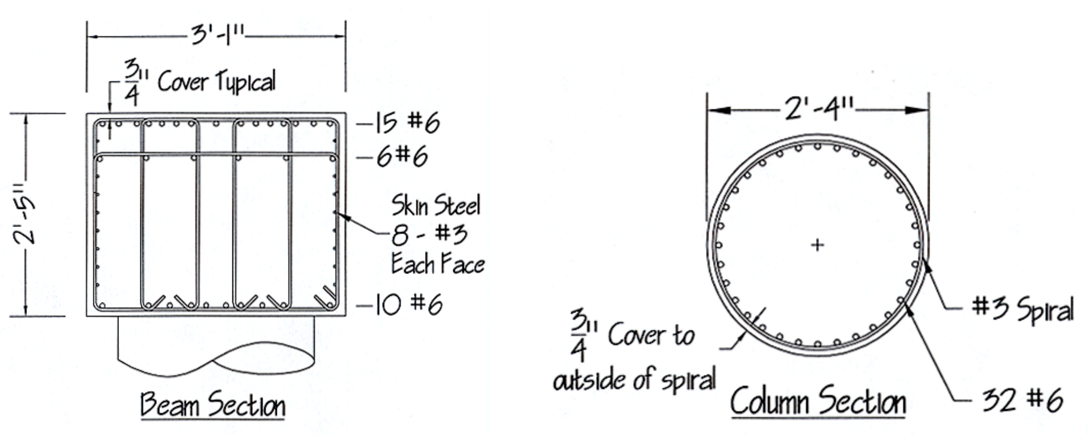


Figure 2 - Beam and Column Cross Sections

Reference: Tee-Nai-RND-A1
Naito & Moehle

Type: Tee
Column: Round - ρ_l = 2.29%
Scale: 3/8
Testing Protocol: Quasi-static by displacement control

TEST OBJECTIVE:

This specimen was tested to evaluate the performance of current Caltrans joint details (as specified in the Bridge Design Specification) and to provide a baseline for comparison with later tests in the experimental program.

TEST SCOPE:

The test was performed upside down with a constant gravity load provided by a hydraulic jack and reacted in such a way as to produce shear and moment at the joint face similar to that in the prototype. Cyclic loading was applied at the 'footing' under displacement and reacted by a jack on one end of the beam to produce an even distribution of axial force in the beam. Low level cycles ranging from 0.1" to 0.5" (2.54mm to 12.7mm) were followed by larger displacements to as much as 10" (25.4 cm) base displacement.

DESIGN PHILOSOPHY:

Tee-Nai-RND-A1 was designed to meet Caltrans' current design requirements with the goal of concentrating inelastic damage into the column and preserving joint integrity by keeping it elastic. The method is based on a strut and tie method with a compression strut across the joint core and some strutting into the cap beam. Joint shear stress was limited to $12\sqrt{f_c'}$ ($1.0\sqrt{f_c'}$ MPa) and beam flexural strength was designed to be larger than the plastic moment strength of the column.

SPECIMEN DETAILS:

f_c': Column = 5560 psi (38.3 MPa)
Cap Beam = 5700 psi (39.3 MPa)
Joint = 5300 psi (36.5 MPa)

COLUMN:
2'-4" (71.1 cm) diameter circular spiral
L_c = 25'-3" (7.7m)
Longitudinal Reinforcement:
32 #6 with straight cut offs - ρ_l = 2.29%
Transverse Reinforcement:
through joint to 2'-4" (71.1 cm) below beam: #3 spiral @ 1.5" (3.8 cm) pitch

Rest: #3 spiral @ 2.75” (7.0 cm) pitch
Axial Load: 5.0% Ag*f_c' constant gravity load

BEAM:
Rectangular - 3’-1”b x 2’-5”h (83.8cm x 73.7cm)
L_b = 12’-2½” (3.72 m)
Longitudinal Reinforcement:
Top: 15 #6 outer layer, 6 #6 inner layer (21 total)
Bottom: 10 #6
Transverse Reinforcement:
6-leg #3 stirrups @ 4.25” (10.8 cm)
Skin Reinforcement:
8 #3 (each face)

JOINT:
Horizontal Reinforcement:
#3 spiral # 1.5” (3.8 cm) pitch – ρ_s = 1.0%
4-leg #3 stirrups (see schematic)
Vertical Reinforcement:
2 6-leg #3 stirrups, 2 3-leg #3 (7.6 cm) stirrups either side of centerline (total 4 each kind) (see schematic)
4 6-leg #3 stirrups @ 2.125” (5.5 cm) in cap beam immediately adjacent to joint

QUANTIFIED RESPONSE:

stress quantities in ***psi (MPa)***	Push	Pull
Max Joint Shear Stress	$7.2\sqrt{f_c'}$ $(0.60\sqrt{f_c'})$	$8.1\sqrt{f_c'}$ $(0.67\sqrt{f_c'})$
Joint Strain at Max Shear Stress	0.0018 rad.	0.0034 rad.
Max Joint Shear Strain	0.0022 rad.	0.0034 rad.
Joint Stress at Max Shear Strain	$6.7\sqrt{f_c'}$ $(0.56\sqrt{f_c'})$	$8.1\sqrt{f_c'}$ $(0.67\sqrt{f_c'})$
Max Principal Tension	---	---
Max Principal Compression	---	---
% Deformation due to joint	~ 0%	

Principal tension and compression values not available in report.

FAILURE MODE:

Buckling of column longitudinal reinforcement and subsequent fracture under reversed loading. (16 of 32 bars fractured by end of test) Yielding of beam bars closest to column (reached twice yield strain)

HYSTERESIS DESCRIPTION:

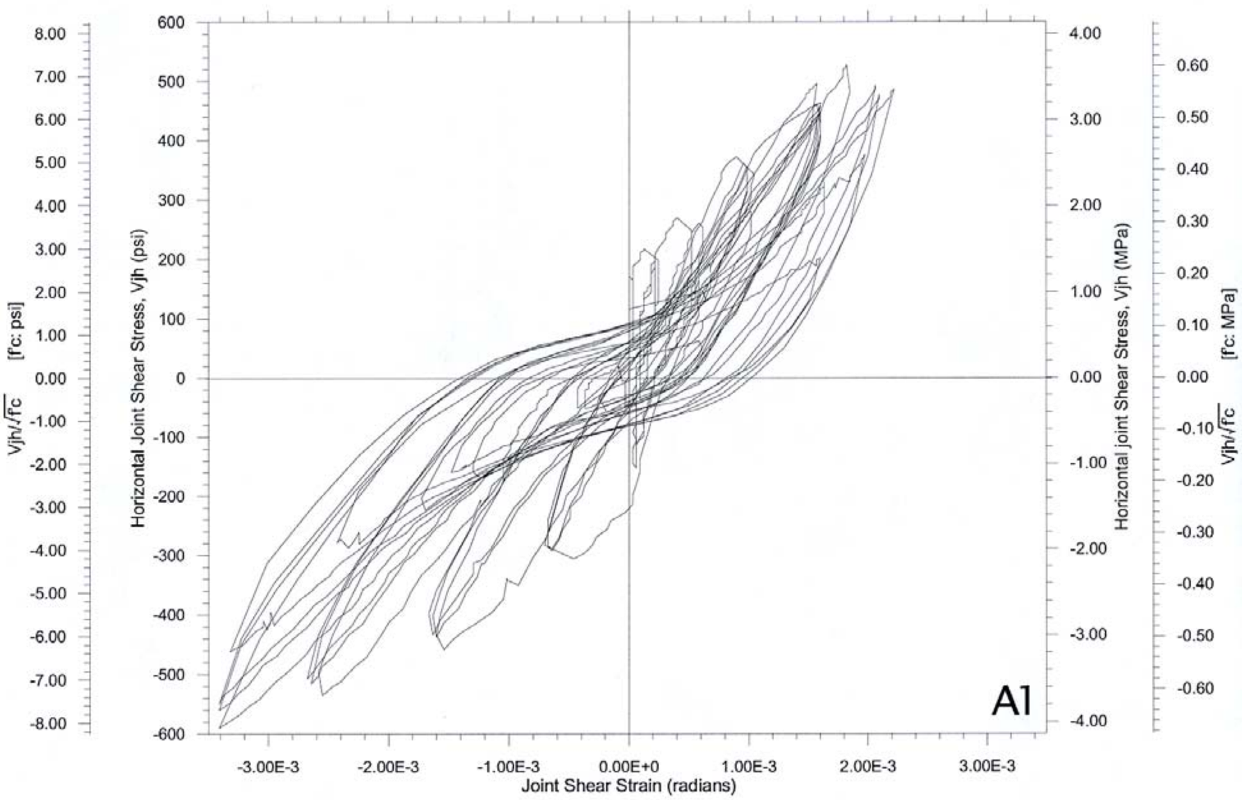


Figure 3 – Joint Shear Stress vs. Shear Strain for Tee-Nai-RND-A1

This specimen was detailed to test current (BDS 1995) Caltrans standards for ductile design of bridge beam-column joints.

Shear stress was calculated based on the full beam width (37” (94.0 cm)) and the spacing of the joint instrumentation (slightly less than the column dimension of 28” (71.1 cm)) and was based on average forces.

Damage was concentrated in the column while the beam remained elastic in all but the portion closest to the column where strains reached twice yield within the joint. The joint saw distributed cracking with cracks on the order of 0.016” (0.41 mm) wide. Cracking was not severe but indicated a possible loss of bond for joint reinforcement.

RESULTS/CONCLUSIONS:

Stable behavior with some pinching. Displacement is not broken into individual components so it isn't clear whether joint deformation had a significant effect on overall system displacement. However, since cracking was distributed and not particularly large in the joint and since damage was concentrated in the column, it can be assumed that joint deformation was not very significant. The author determined that the joint remained essentially elastic throughout the test and was therefore a satisfactory design.

Bibliography:

Naito, Clay, Jack P. Moehle. "Design of Innovative Reinforced Concrete Bridge Joints." Sixth U.S. National Conference on Earthquake Engineering Oakland, CA: Earthquake Engineering Research Inst., 1998. (*computer file)*

R/C Bridge Beam-Column Joints

Tee-Nai-RND-A2

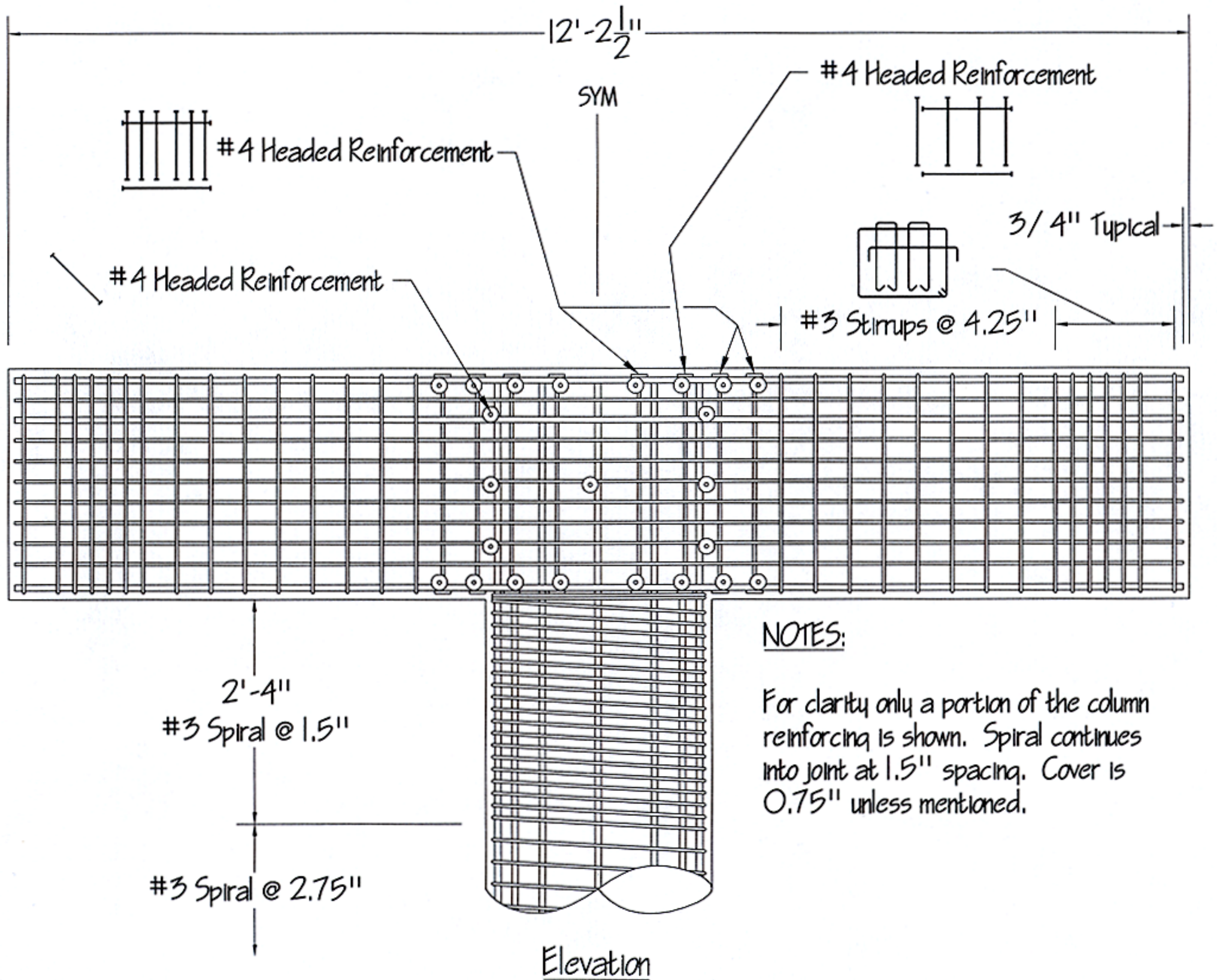


Figure 1 - Elevation and Cross-Sections of Tee-Nai-RND-A2

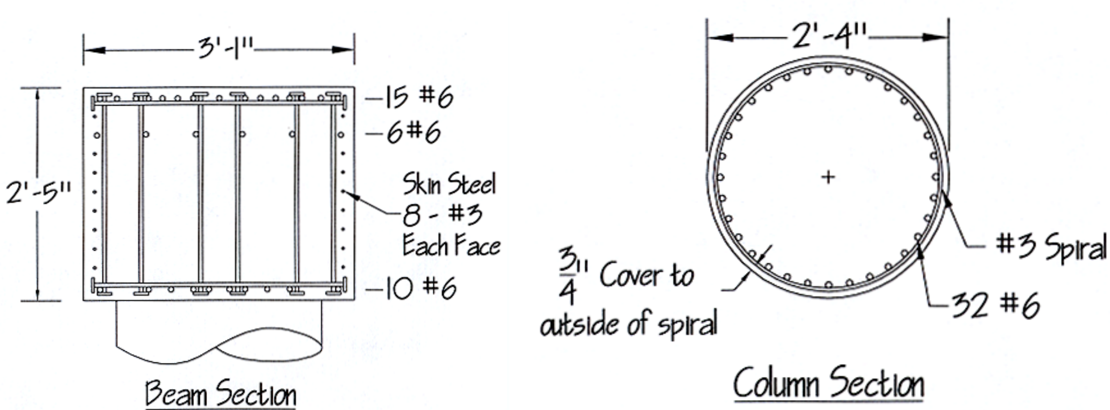


Figure 2 - Beam and Column Cross Sections

Reference: Tee-Nai-RND-A2
Naito & Moehle

Type: Tee
Column: Round - ρ_l = 2.29%
Scale: 3/8
Testing Protocol: Quasi-static by displacement control

TEST OBJECTIVE:

This specimen was tested to evaluate the performance of a joint reinforced with headed reinforcement replacing all standard transverse reinforcement. The test also provided a baseline for future tests of joints with headed reinforcement.

TEST SCOPE:

The test was performed upside down with a constant gravity load provided by a hydraulic jack and reacted in such a way as to produce shear and moment at the joint face similar to that in the prototype. Cyclic loading was applied at the 'footing' under displacement and reacted by a jack on one end of the beam to produce an even distribution of axial force in the beam. Low level cycles ranging from 0.1" to 0.5" (2.54mm to 12.7mm) were followed by larger displacements to as much as 10" (25.4 cm) base displacement.

DESIGN PHILOSOPHY:

Tee-Nai-RND-A2 was designed with the idea of preserving joint integrity and forcing inelastic action into the column while keeping the joint and cap beams elastic. An equivalent area of headed reinforcement was substituted for all transverse joint reinforcement (the column spiral continued into the joint as in A1) and column longitudinal reinforcement was headed at the cap beam top surface.

SPECIMEN DETAILS:

f_c':	Column =	5560 psi (38.3 MPa)
	Cap Beam =	5700 psi (39.3 MPa)
	Joint =	5530 psi (38.1 MPa)

COLUMN:
2'-4" (71.1 cm) diameter circular spiral
L_c = 25'-3" (7.7m)
Longitudinal Reinforcement:
  32 #6 with straight cut offs - ρ_l = 2.29%
Transverse Reinforcement:
  through joint to 2'-4" (71.1 cm) below beam: #3 spiral @ 1.5" (3.8 cm) pitch
  Rest: #3 spiral @ 2.75" (7.0 cm) pitch
Axial Load: 5.0% Ag*f_c' constant gravity load

BEAM:
Rectangular - 3'-1"b x 2'-5"h (83.8cm x 73.7cm)
L_b = 12'-2½" (3.72 m)
Longitudinal Reinforcement:
    Top: 15 #6 outer layer, 6 #6 inner layer (21 total)
    Bottom: 10 #6
Transverse Reinforcement:
    4 6-leg #3 stirrups @ 2.125" (5.4 cm) starting at joint face
    Rest: 6-leg #3 stirrups @ 4.25" (10.8 cm)
Skin Reinforcement:
    8 #3 (each face)

JOINT:
Horizontal Reinforcement:
    #3 spiral # 1.5" (3.8 cm) pitch - ρ_t = 1.0 %
    15 #4 headed reinforcement (see schematic for layout)
    4 #4 headed bars in cap beam immediately adjacent to each side of joint (8 total)
Vertical Reinforcement:
    20 #4 headed reinforcement (see schematic for layout)
    12 #4 headed bars in cap beam immediately adjacent to each side of joint (24 total)
headed reinforcement as transverse reinforcement continues beyond joint (dimensions not noted)

QUANTIFIED RESPONSE:

stress quantities in ***psi (MPa)***	Push	Pull
Max Joint Shear Stress	$7.15\sqrt{f_c'}$ $(0.59\sqrt{f_c'})$	$7.3\sqrt{f_c'}$ $(.61\sqrt{f_c'})$
Joint Strain at Max Shear Stress	0.0029 rad.	0.0024 rad.
Max Joint Shear Strain	0.003 rad.	0.0028 rad.
Joint Stress at Max Shear Strain	$7.0\sqrt{f_c'}$ $(0.58\sqrt{f_c'})$	$7.1\sqrt{f_c'}$ $(0.59\sqrt{f_c'})$
Max Principal Tension	---	---
Max Principal Compression	---	---
% Deformation due to joint	~ 2%	

Principal tension and compression values not available in report.

FAILURE MODE:

Buckling of column longitudinal reinforcement followed by fracture after reversed loading. 13 of 32 longitudinal bars fractured after final cycle (3^{rd}) at peak displacement.

HYSTERESIS DESCRIPTION:

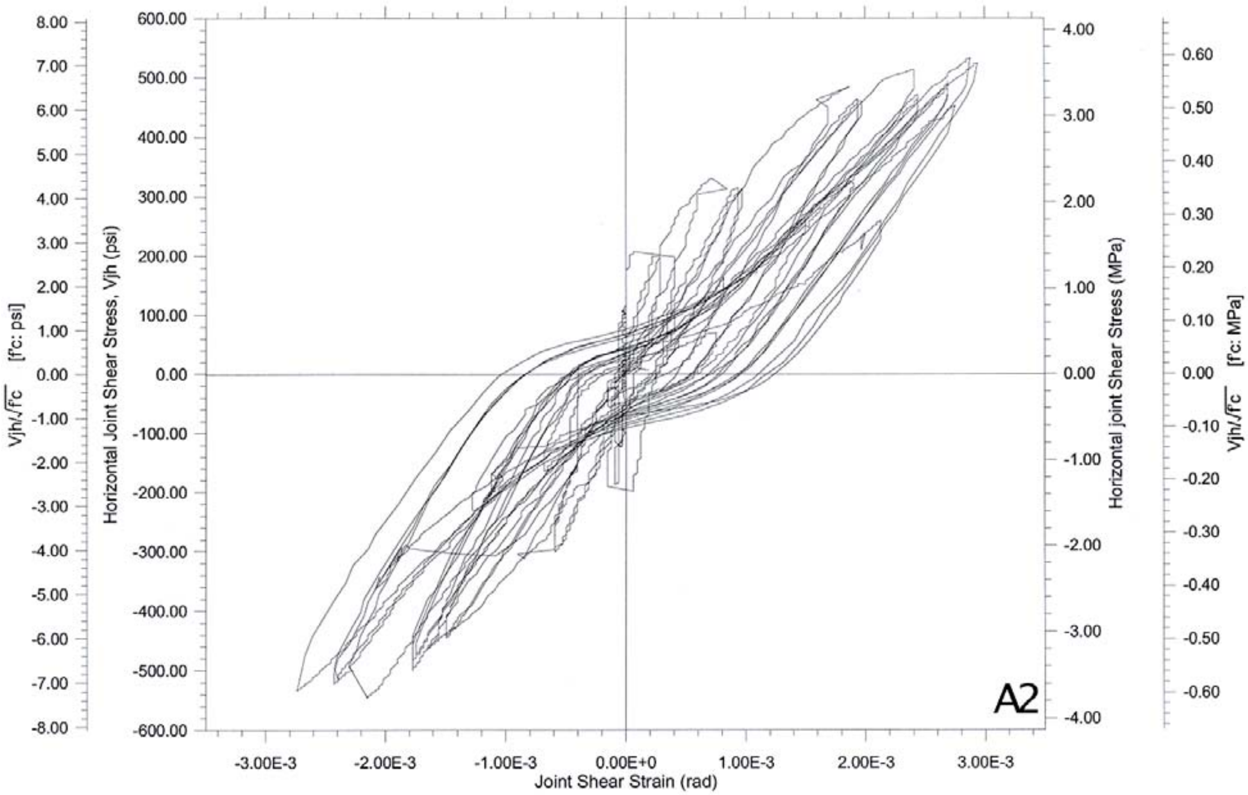


Figure 3 - Joint Shear Stress vs. Shear Strain for Tee-Nai-RND-A2

This specimen was designed to test the applicability and performance of headed reinforcement as a replacement for standard joint reinforcement. All transverse steel in the joint region was replaced with an equal area of headed reinforcement.

Shear stress was calculated based on the full beam width (37" (94.0 cm)) and the spacing of the joint instrumentation (slightly less than the column dimension of 28" (71.1 cm)) and was based on average forces.

Damage was concentrated in the column while the beam remained elastic in all but the portion closest to the column where strains reached three times yield within the joint. The joint saw distributed cracking with cracks on the order of 0.016" (0.41 mm) wide. This cracking suggests loss of bond in the joint reinforcement that, while not serious in this test, could become serious in a repair or retrofit.

Buckling of the column longitudinal reinforcement and later fracture after repitition of the large cycles brought on failure. 13 of the 32 column bars were fractured by the end of the test.

RESULTS/CONCLUSIONS:

Stable behavior with some pinching. Displacement is not broken into individual components so it is not clear whether joint deformation had a significant effect on overall system displacement. Cracking was distributed and not particularly large in the joint and damage was concentrated in the column. Headed reinforcement was determined to be a viable alternative to conventional joint reinforcement since damage was concentrated in the column as desired.

Bibliography:

Naito, Clay, Jack P. Moehle. "Design of Innovative Reinforced Concrete Bridge Joints." Sixth U.S. National Conference on Earthquake Engineering Oakland, CA: Earthquake Engineering Research Inst., 1998. (*computer file)*

R/C Bridge Beam-Column Joints
Tee-Sri-RND-IC1

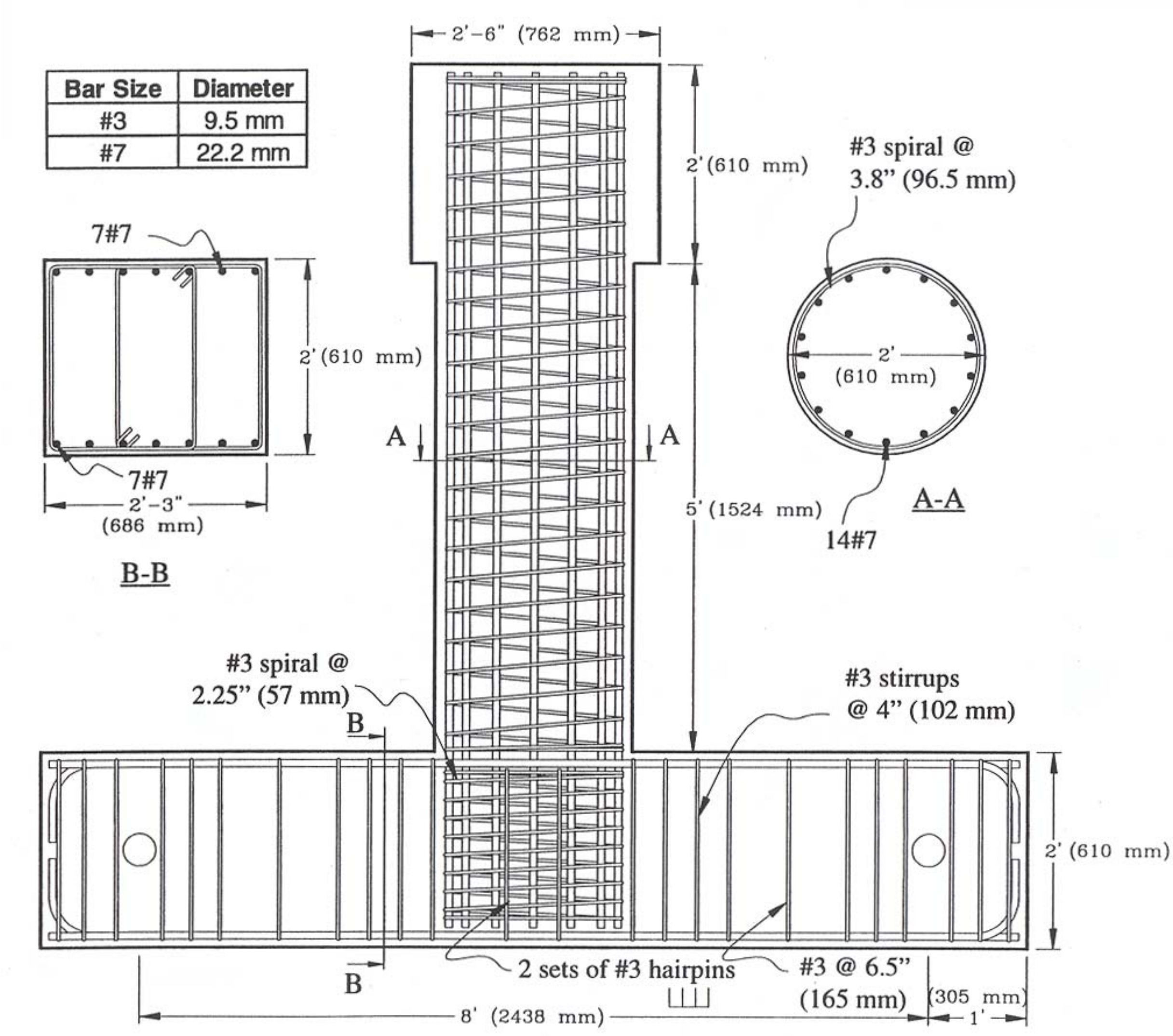


Figure 1 - Elevation of Tee-Sri-RND-IC1

Reference: Tee-Sri-RND-IC1
Sritharan, et al. (SSRP 96/09)

Type: Tee
Column: Rectangular w/ 3" (7.6 cm) chamfered edges - ρ_l = 1.86 %
Scale: 1/3
Testing Protocol: Pseudo-static, early cycles load control, later by displacement control

TEST OBJECTIVE:

To evaluate the performance of a redesign of a typical bridge joint detail. A previous test was performed on an 'as-built' specimen which established a baseline response. Tee-Sri-RND-IC1 was first in a series of three specimens designed to test new joint detailing strategies. It used a reinforced concrete cap beam as opposed to the prestressed solutions used in the two other tests in the series.

TEST SCOPE:

Loading was designed to test the cyclic behavior of the specimen and to test analytical strength predictions with actual test results. Also, the strength of the joint force-transfer mechanism was targeted through excursions to displacements that severely stressed the joint region.

The specimen was initially loaded with dead load before cyclic loading began. Initial cyclic loading was applied under force control with cycles up to approximate flexural yielding. Beyond this level, load was applied under displacement control with increasing levels of displacement ductility (up to a maximum of μ_Δ=±8.0).

DESIGN PHILOSOPHY:

Tee-Sri-RND-IC1 was a redesign of an older Caltrans design intended to test the feasibility of using an external joint force mechanism. Instead of depending solely on the concrete and reinforcement in the joint core to resist the forces introduced by yielding of the column, additional reinforcement was added into the cap beam adjacent to the joint to resolve joint forces. The researchers used the external force transfer mechanism to obviate the need for using hooked longitudinal column bars in favor of straight anchorage into the joint.

The column compression force and 50% of the column tension force (assuming the moment forms a tension-compression couple) is assumed to be anchored directly by bond into the joint compression strut. The other 50% of the tension force is anchored through two compression struts, one directed into the beam directly adjacent to the joint and the other directed across the joint. The external strut is resolved into vertical forces resisted by additional stirrups in the cap beam and horizontal forces resisted by additional tension and compression steel in the beam. Spiral joint reinforcement was provided to control crack width, to support unbalanced

SPECIMEN DETAILS:

f_c':	Column =	4560 psi (31.4 MPa)
	Cap Beam =	5760 psi (39.7 MPa)
	Joint =	5760 psi (39.7 MPa)

COLUMN:
24" (61.0 cm) diameter spirally reinforced round column
L_c = 5' (1.52 m) (with additional 2' (61.0 cm) long footing)
Longitudinal Reinforcement:

14 #7 bars (straight, not hooked) - ρ_l = 1.86 %
Transverse Reinforcement:
#3 spiral @ 3.8” (9.65 cm) pitch
Axial Load: 4.36% Ag*f_c’ constant gravity load

BEAM:
Rectangular - 2’-3”b x 2’-0”h
L_b=10’-0” (3.0 m) end-to-end
Longitudinal Reinforcement:
Top: 7 #7
Bottom: 7 #7
Transverse Reinforcement:
4-leg #3 stirrups @ 4” (10.2 cm)
Skin Reinforcement:
9 #2 @ 2” (5.1 cm), each side

JOINT:
Horizontal Reinforcement:
#3 spiral @ 2 ¼” (5.7 cm) pitch, ρ_s = 0.087
Vertical Reinforcement:
2 sets of 4-leg #3 hairpins

QUANTIFIED RESPONSE:

stress quantities in ***psi*** **(*MPa)***	Push	Pull
Max Joint Shear Stress	$6\sqrt{f_c'}$ $(0.5\sqrt{f_c'})$	$6\sqrt{f_c'}$ $(0.5\sqrt{f_c'})$
Joint Strain at Max Shear Stress	0.004 rad.	0.045 rad.
Max Joint Shear Strain	0.008 rad.	0.009 rad.
Joint Stress at Max Shear Strain	$4.2\sqrt{f_c'}$ $(0.35\sqrt{f_c'})$	$5\sqrt{f_c'}$ $(0.42\sqrt{f_c'})$
Max Principal Tension	$6\sqrt{f_c'}$ $(0.5\sqrt{f_c'})$	$5.5\sqrt{f_c'}$ $(0.46\sqrt{f_c'})$
Max Principal Compression	$6\sqrt{f_c'}$ $(0.5\sqrt{f_c'})$	$7.5\sqrt{f_c'}$ $(0.62\sqrt{f_c'})$
% Deformation due to joint	25-30%	25-30%

FAILURE MODE:

Large joint shear deformation and deterioration (spalling of joint cover concrete), buckling of column longitudinal reinforcement and near-fracture of column transverse spiral.

HYSTERESIS DESCRIPTION:

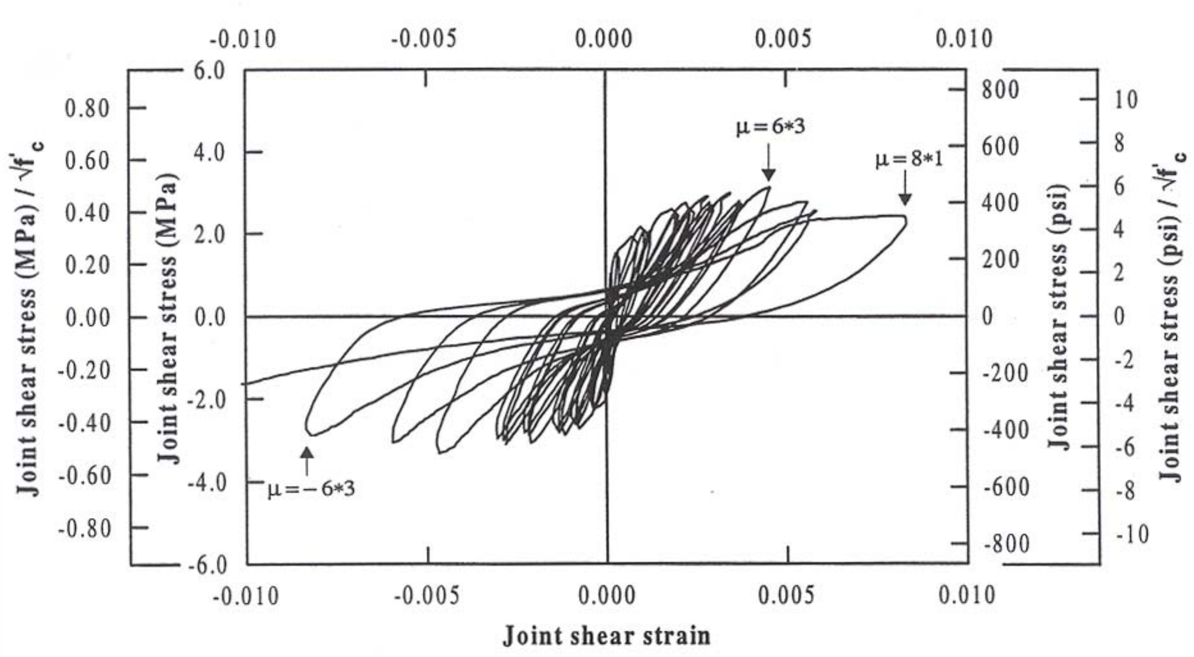


Figure 2 - Joint Shear Stress vs. Shear Strain for Tee-Sri-RND-IC1

Shear stress reported here were calculated based on full effective area of the joint and the column overstrength moment at the column-joint interface. Principal tension and compression were calculated from horizontal and vertical joint stresses with no dispersion considered. (The report also calculated maximum joint shear stresses and principal stresses which were based on the resultant tension and compression forces of members framing into the joint)

Initial joint shear cracks appeared at a load level below expected first yield. At first yield, diagonal joint cracks extended from joint corner to corner. Later cycles increased both the size and number of joint cracks with larger than expected crack widths. A large diagonal crack opened in the joint during cycles at a system displacement ductility of $\mu_\Delta=\pm6.0$. The lack of confinement provided by the vertical hairpins in the joint was deemed partially responsible for the cracking and expansion of the joint. A real structure should have better confinement from hoops.

At a system ductility level of $\mu_\Delta\pm8.0$, significant joint shear deformation was noted and was assumed to be responsible for a significant portion of the column displacement.(as much as 25-30% at late cycles) Further deterioration and damage to the joint continued in later cycles. At the end of testing several column longitudinal bars had buckled and the column spiral had nearly fractured near the joint interface.

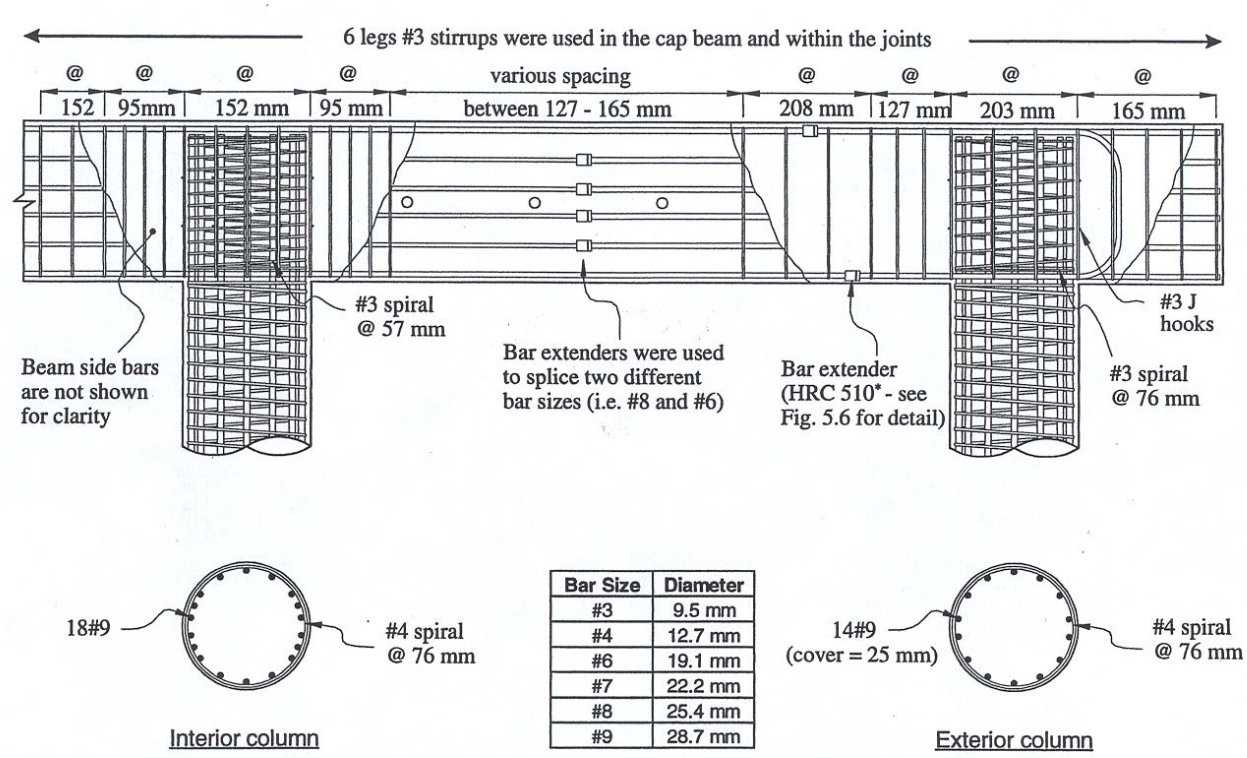
RESULTS/CONCLUSIONS:

Joint damage appeared excessive and an elastic response would certainly be preferred, but according to the report's authors, the joint region performed well in the expected range of ductilities. The joint shear stress-strain hysteresis was pinched and there was significant stiffness degradation along with some strength degradation at higher ductilities. Again, the vertical hairpins created part of the poor performance by failing to effectively confine the core.

Bibliography:

Sritharan, Sri, M.J.Nigel Priestley, Frieder Seible. "Seismic Response of Column/Cap Beam Connections with Cap Beam Prestressing (Report No. SSRP - 96/09)." La Jolla, CA: Structural Systems Research, University of California, San Diego, 1996.

R/C Bridge Beam-Column Joints
Knee-Sri-RND-1



Bar Size	Diameter
#3	9.5 mm
#4	12.7 mm
#6	19.1 mm
#7	22.2 mm
#8	25.4 mm
#9	28.7 mm

Figure 1 - Elevation and Column Cross Sections of Knee-Sri-RND-1

Note: All dimensions in (mm).

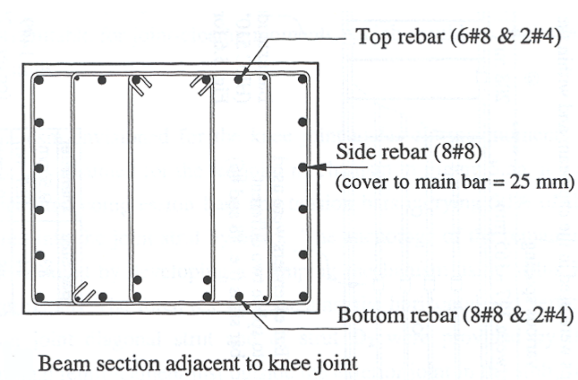


Figure 2 - Beam Cross Section

Note: All dimensions in (mm).

Reference: Knee-Sri-RND-1
Sritharan, et al. (SSRP 97/03)

Type: Knee (part of a multi-column bent specimen)
Column: Round - ρ_l = 3.09%
Scale: 1/2
Testing Protocol: Pseudo-static, early cycles under load control, later by displacement control

TEST OBJECTIVE:

To examine the most efficient beam-column joint detail for use in multi-column bridge bents.

TEST SCOPE:

The test specimen was loaded first with gravity loads in a sequence intended to minimize cracking. Initial cycles were force controlled up to initial yield and further cycles were conducted under displacement control. Displacement ductility was increased from μ_Δ=1.0 up to 8.0. At least three cycles were applied at each displacement step.

DESIGN PHILOSOPHY:

Knee-Sri-RND-1 was designed to meet current (as of 1997) seismic design criteria while also satisfying capacity design concepts. Rational joint force transfer mechanisms were used in the design – the main joint compression strut was expected to anchor half of the tension force and the total compression force in the column bars and the remainder was expected to be resisted by a clamping strut anchored outside the joint region. For opening moments, this clamping strut is developed by additional vertical ties and beam bottom longitudinal steel. For closing moments, the strut was resisted by the continuous beam longitudinal reinforcement in the beam stub just outside of the joint. The cap beam and joint were designed for the maximum flexural capacity of the column (capacity design).

SPECIMEN DETAILS:

f_c':	Column =	5000 psi (34.5 MPa)
	Cap Beam =	4000 psi (27.6 MPa)
	Joint =	4000 psi (27.6 MPa)

COLUMN:
24" (61 cm) diameter spirally reinforced circular column
L_c = 9'-0" (2.75 m) top of footing to bottom of cap beam
Longitudinal Reinforcement:
14 #9 with straight cut offs - ρ_l = 3.09%
Transverse Reinforcement:
#4 spiral @ 3" (7.6 cm) pitch - ρ_s = 1.2%
Axial Load: 4.30% Ag*f_c' constant gravity load (total axial load varied with lateral load)

BEAM:
Rectangular - 3'-0"b x 2'-6"h (91.4cm x 76.2 cm)
Longitudinal Reinforcement:
Top: 6 #8, 2 #4
Bottom: 8 #8, 2 #4 in two layers
Transverse Reinforcement:
6-leg #3 stirrups
Skin Reinforcement:
8 #8 (4 each side)

JOINT:
Horizontal Reinforcement: #3 spiral at 3" (7.6 cm) pitch, $\rho_s = 0.67\%$
Vertical Reinforcement: 4 sets of 6-leg #3 stirrups

QUANTIFIED RESPONSE:

stress quantities in ***psi (MPa)***	Open	Close
Max Joint Shear Stress	$8\sqrt{f_c'}$ $(0.66\sqrt{f_c'})$	$6\sqrt{f_c'}$ $(0.50\sqrt{f_c'})$
Joint Strain at Max Shear Stress	0.003 rad.	0.003 rad.
Max Joint Shear Strain	0.004 rad.	0.006 rad.
Joint Stress at Max Shear Strain	$5.4\sqrt{f_c'}$ $(0.45\sqrt{f_c'})$	$4.2\sqrt{f_c'}$ $(0.35\sqrt{f_c'})$
Max Principal Tension	$6.9\sqrt{f_c'}$ $(0.57\sqrt{f_c'})$	$6.4\sqrt{f_c'}$ $(0.53\sqrt{f_c'})$
Max Principal Compression	$9.5\sqrt{f_c'}$ $(0.79\sqrt{f_c'})$	$5.3\sqrt{f_c'}$ $(0.44\sqrt{f_c'})$
% Deformation due to joint	6%	

FAILURE MODE:

Buckling of column longitudinal reinforcement in knee joint with deterioration of the joint and buckling of beam longitudinal bars in tee joint.

HYSTERESIS DESCRIPTION:

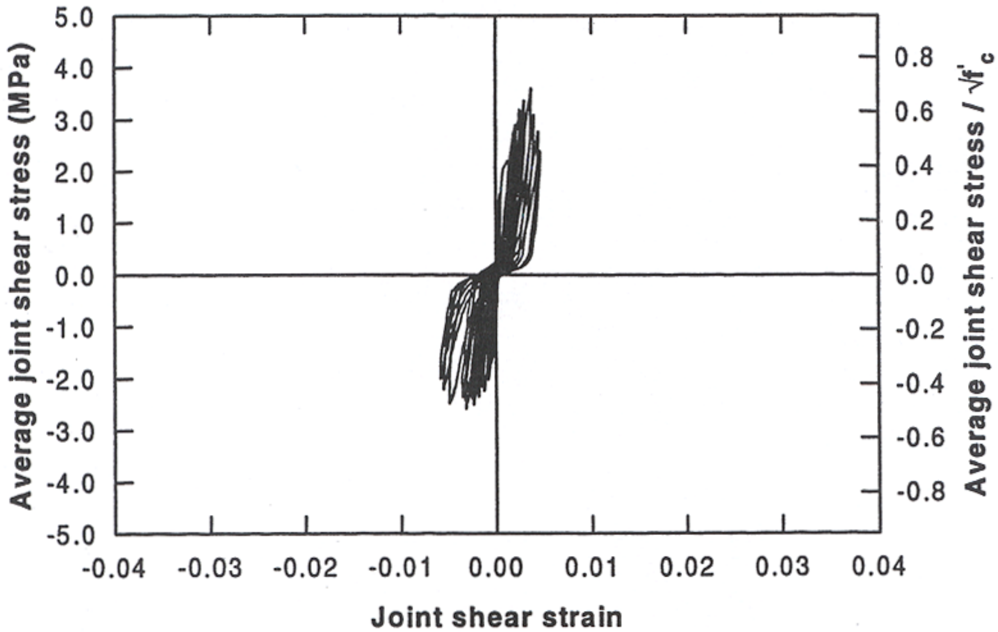


Figure 3 - Joint Shear Stress vs. Shear Strain for Knee-Sri-RND-1 (SI Units)

Larger stresses were developed in the closing (push) direction, pinched, with both stiffness and strength degradation. Stiffness degradation appears to be greater in the opening direction and strength degradation appears greater in the closing direction. Greater strains were developed in the opening direction.

Shear stress calculated based on full joint length equal to column dimension (24" (61 cm)) and width equal to $h_c*\sqrt{2}$, as suggested by Priestley (33.9" (86.1 cm)), based on moments at column-joint interface.

Cracking in the joint began in early elastic cycles with cracks of up to 1mm width in cycles at a system ductility of 1.0. Lengthening of cracks occurred in subsequent cycles at greater displacement ductility and splitting cracks developed at the top of the joint as well. A shear crack formed from the bottom of the joint to the top beam reinforcement at ductility 3.0. Continued cracking and widening of existing cracks occurred at later cycles with widths of about 1.5mm at ductility 4.0. Overall damage to the knee joint was considered negligible at the end of the test. (especially as compared to the tee joint)

The researchers determined that knee joint deformation was responsible for roughly 6% of total exterior column lateral displacement.

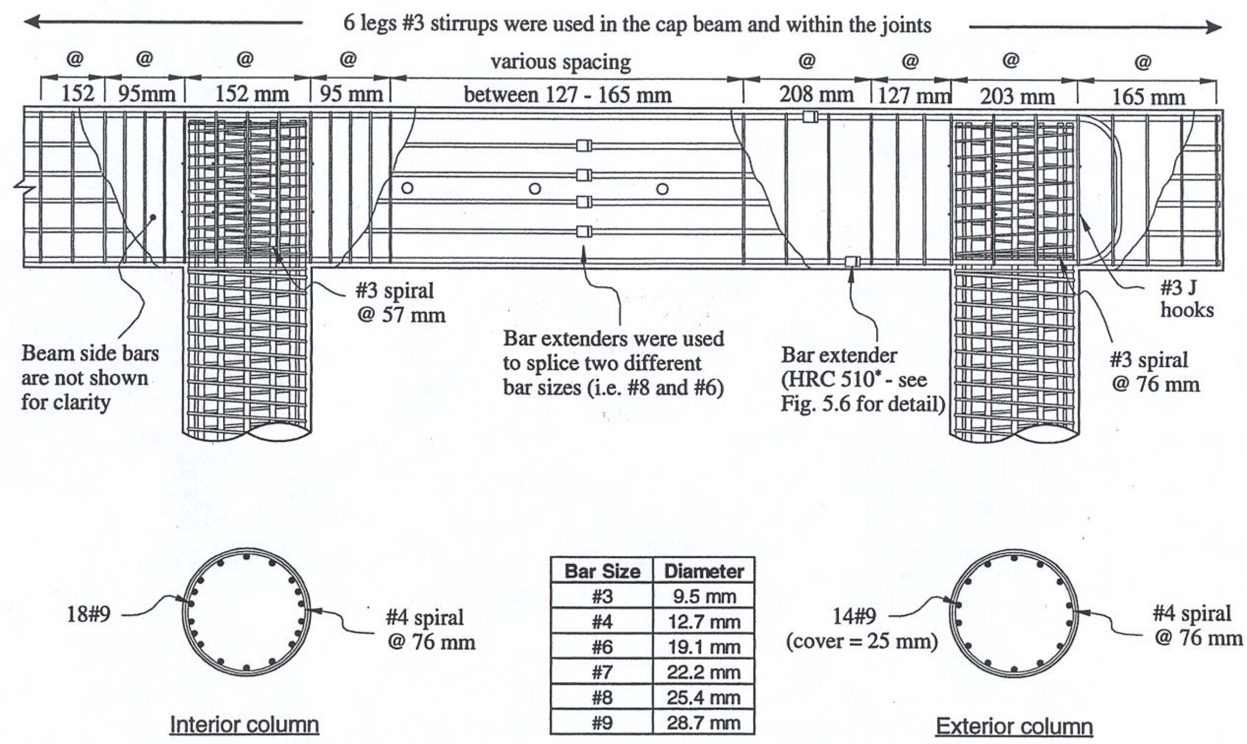
RESULTS/CONCLUSIONS:

Ductile detailing including proper confinement and embedment conditions allowed for good behavior of this specimen. The joint was nominally elastic for most of the test and joint deformations and damage were not excessive. The low level of contribution to system displacement suggests that joint deformations could be ignored in an analysis of this system.

Bibliography:

Sritharan, Sri, M.J. Nigel Priestley, Frieder Seible. "Seismic Design and Performance of Concrete Multi-Column Bents for Bridges (Report No. SSRP - 97/03)." La Jolla, CA: Structural Systems Research, University of California, San Diego, 1997.

R/C Bridge Beam-Column Joints
Tee-Sri-RND-2



Bar Size	Diameter
#3	9.5 mm
#4	12.7 mm
#6	19.1 mm
#7	22.2 mm
#8	25.4 mm
#9	28.7 mm

Figure 1 - Elevation and Column Cross Sections of Knee-Sri-RND-1

Note: All dimensions in (mm).

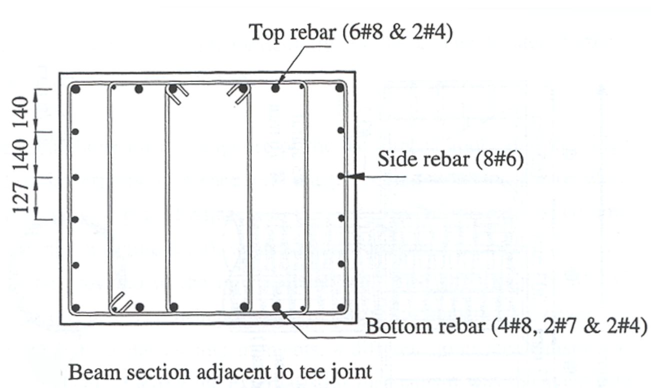


Figure 2 - Beam Cross Section

Note: All dimensions in (mm).

Reference: Tee-Sri-RND-2
Sritharan, et al. (SSRP 97/03)

Type: Tee (part of a multi-column bent specimen)
Column: Round - $\rho_l = 3.98\%$
Scale: 1/2
Testing Protocol: Pseudo-static, early cycles under load control, later by displacement control

TEST OBJECTIVE:

To examine the most efficient beam-column joint detail for use in multi-column bridge bents.

TEST SCOPE:

The test specimen was loaded first with gravity loads in a sequence intended to minimize cracking. Initial cycles were force controlled up to initial yield and further cycles were conducted under displacement control. Displacement ductility was increased from μ_Δ=1.0 up to 8.0. At least three cycles were applied at each displacement step.

DESIGN PHILOSOPHY:

Knee-Sri-RND-1 was designed to meet current (as of 1997) seismic design criteria while also satisfying capacity design concepts. Rational joint force transfer mechanisms were used in the design – the main joint compression strut was expected to anchor half of the tension force and the total compression force in the column bars and the remainder was expected to be resisted by a clamping strut anchored outside the joint region. For opening moments, this clamping strut is developed by additional vertical ties and beam bottom longitudinal steel. For closing moments, the strut was resisted by the continuous beam longitudinal reinforcement in the beam stub just outside of the joint.

SPECIMEN DETAILS:

f_c': Column = 5000 psi (34.5 MPa)
Cap Beam = 4000 psi (27.6 MPa)
Joint = 4000 psi (27.6 MPa)

COLUMN:
24" (61 cm) diameter spirally reinforced circular column
L_c = 9'-0" (2.75 m) top of footing to bottom of cap beam
Longitudinal Reinforcement:
18 #9 with straight cut offs - $\rho_l = 3.98\%$
Transverse Reinforcement:
#4 spiral @ 3" (7.6 cm) pitch - $\rho_s = 1.2\%$

Axial Load: 7.32% Ag*f_c' constant gravity load

BEAM:
Rectangular - 3'-0"b x 2'-6"h (91.4cm x 76.2 cm)
Longitudinal Reinforcement:
Top: 6 #8, 2 #4
Bottom: 4 #8, 2 #7, 2 #4
Transverse Reinforcement:
6-leg #3 stirrups @ 2 ½" - 6 ½" (varied) (6.4 cm – 16.5 cm)
Skin Reinforcement:
8 #6 (4 each side)

JOINT:
Horizontal Reinforcement:
#3 spiral at 2¼" (5.7 cm) pitch, $\rho_s = 0.86\%$
Vertical Reinforcement:
6-leg #3 stirrups @ 6" (15.2 cm)

QUANTIFIED RESPONSE:

stress quantities in ***psi (MPa)***	Push	Pull
Max Joint Shear Stress	$9\sqrt{f_c'}$ $(0.75\sqrt{f_c'})$	$9\sqrt{f_c'}$ $(0.75\sqrt{f_c'})$
Joint Strain at Max Shear Stress	0.011 rad.	0.017 rad.
Max Joint Shear Strain	0.035 rad.	0.035 rad.
Joint Stress at Max Shear Strain	$5.66\sqrt{f_c'}$ $(0.47\sqrt{f_c'})$	$6.0\sqrt{f_c'}$ $(0.5\sqrt{f_c'})$
Max Principal Tension	$7.3\sqrt{f_c'}$ $(0.61\sqrt{f_c'})$	$10\sqrt{f_c'}$ $(0.83\sqrt{f_c'})$
Max Principal Compression	$11\sqrt{f_c'}$ $(0.91\sqrt{f_c'})$	$9\sqrt{f_c'}$ $(0.75\sqrt{f_c'})$
% Deformation due to joint	41%	

FAILURE MODE:

Buckling of column longitudinal reinforcement in knee joint, deterioration of joint and buckling of beam longitudinal bars in tee joint.

HYSTERESIS DESCRIPTION:

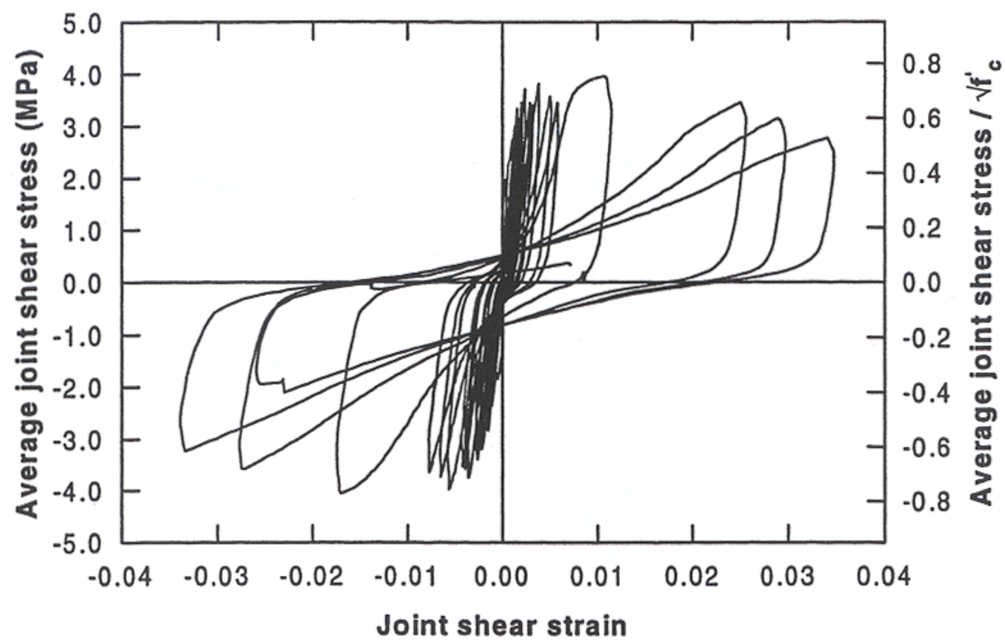


Figure 3 - Joint Shear Stress vs. Shear Strain for Tee-Sri-RND-2 (SI Units)

Severe stiffness and strength degradation are readily apparent, especially as compared to the knee joint (Knee-Sri-RND-1) in the same frame. Strains are as much as 5-6 times larger than in the knee joint but stresses are roughly equivalent.

Shear stress calculated based on full joint length equal to column dimension (24" (61 cm)) and width equal to h_c*√2, as suggested by Priestly, (33.9" (86.1 cm)) based on moments at column-joint interface.
Cracking in the joint was minor at low cycles but increased rapidly at larger cycles, especially starting around a system ductility level of 2.0. Relative movement along a large diagonal crack in the joint was noted in cycles at a ductility of 4.0 and cracks did not close when lateral load passed through the zero point. Width of the large center joint crack increased to 4.5mm in the first cycle to a ductility of 6.0 and continued to increase. Damage concentrated in the plastic hinge region of the column but continued to accumulate in the joint until the end of the test.

Punching failure of the column through the tee joint was evident at test's end with movement of about 29mm above the top of the joint. Severe spalling was noted on the tee joint as well. Failure of the system was due to buckling of column longitudinal bars in the exterior column; no buckling of column bars anchoring into the tee joint was evident.

Tee joint deformation was calculated to be responsible for about 41% of total interior column displacement due to the large strains it experienced.

RESULTS/CONCLUSIONS:

The behavior of this tee joint was not satisfactory but it must be noted that, according to the researchers, the joint was pushed beyond the level of displacement (to a system ductility of about 6.0) it would be expected to withstand in a real earthquake event (a real event might impose deformations consistent with a system ductility of about 4.0). Furthermore, target concrete strength in the joint was not reached resulting in higher than expected joint shear stresses. Also, overstrength in a real design would compensate for some of the poor behavior this joint exhibited. In any case, deformations in this joint should not be ignored.

Bibliography:

Sritharan, Sri, M.J. Nigel Priestley, Frieder Seible. "Seismic Design and Performance of Concrete Multi-Column Bents for Bridges (Report No. SSRP - 97/03)." La Jolla, CA: Structural Systems Research, University of California, San Diego, 1997.