6th Canadian Conf. Earthquake Engineering / Toronto / 1991 / 6ième Conf. Canadienne Sismique

Earthquake type loading on r/c beam-column connections: special cases of wide beams and eccentric beams

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ABSTRACT

Special reinforced concrete beam to column connections often found in framed structures have been tested under simulated earthquake loading at the University of Michigan Structural Engineering Laboratory. These connections include wide beam-column connections and eccentric beam-column connections.

Wide beam-columns, which are currently prohibited in high seismic zones (ACI-ASCE Committee 352 1985) will perform acceptably if design parameters are carefully controlled. The major parameters which must be controlled are the beam-width to column-width ratio, the fraction of the total longitudinal steel anchored in the column core, and the lateral stiffness of the structure.

The other special case considered is eccentric beam-column connections. In this type of connection the beam width is less than the column width, and the beam axes are eccentric to the column axis. The experimental program investigates the effect of varying the beam width, the beam depth, and the amount of longitudinal beam reinforcing on the behavior of the connection.

EXPERIMENTAL SETUP

Both the wide beam specimens and the eccentric beam specimens were tested by imposing a lateral displacement at the top of the column. The base of the column and the beam ends were pinned, consistent with the assumption that these locations were the inflection points for the lateral load moment diagram (see Fig. 1). Increasing story drifts from 0.5% to 5.0% were applied to the specimens. An axial load of 20 kips was applied to the column to preclude tension in the column during testing.

WIDE BEAM-COLUMN CONNECTIONS

Wide beam-column connections, whose beams are wider than their supporting columns, are often found in one-way concrete joist systems and in other buildings where floor-to-ceiling heights are restricted. Testing of these connections stems from the recommendation by ACI-ASCE Committee 352 that these connections be experimentally evaluated for use in high seismic zones (1985).

The wide beam-column experimental program consisted of the testing of four exterior 3/4-scale specimens, including transverse beam with reinforcement. The effects of joint shear stress level, fraction of beam longitudinal reinforcement anchored in the column core, and beam-width to column-width ratios (b_w/c_w) were explored as part of this experimental research. In addition to the experiments, computer simulation of R/C frames using wide beams and an analytical model of the wide beam force transfer mechanism were formulated. Information from interior

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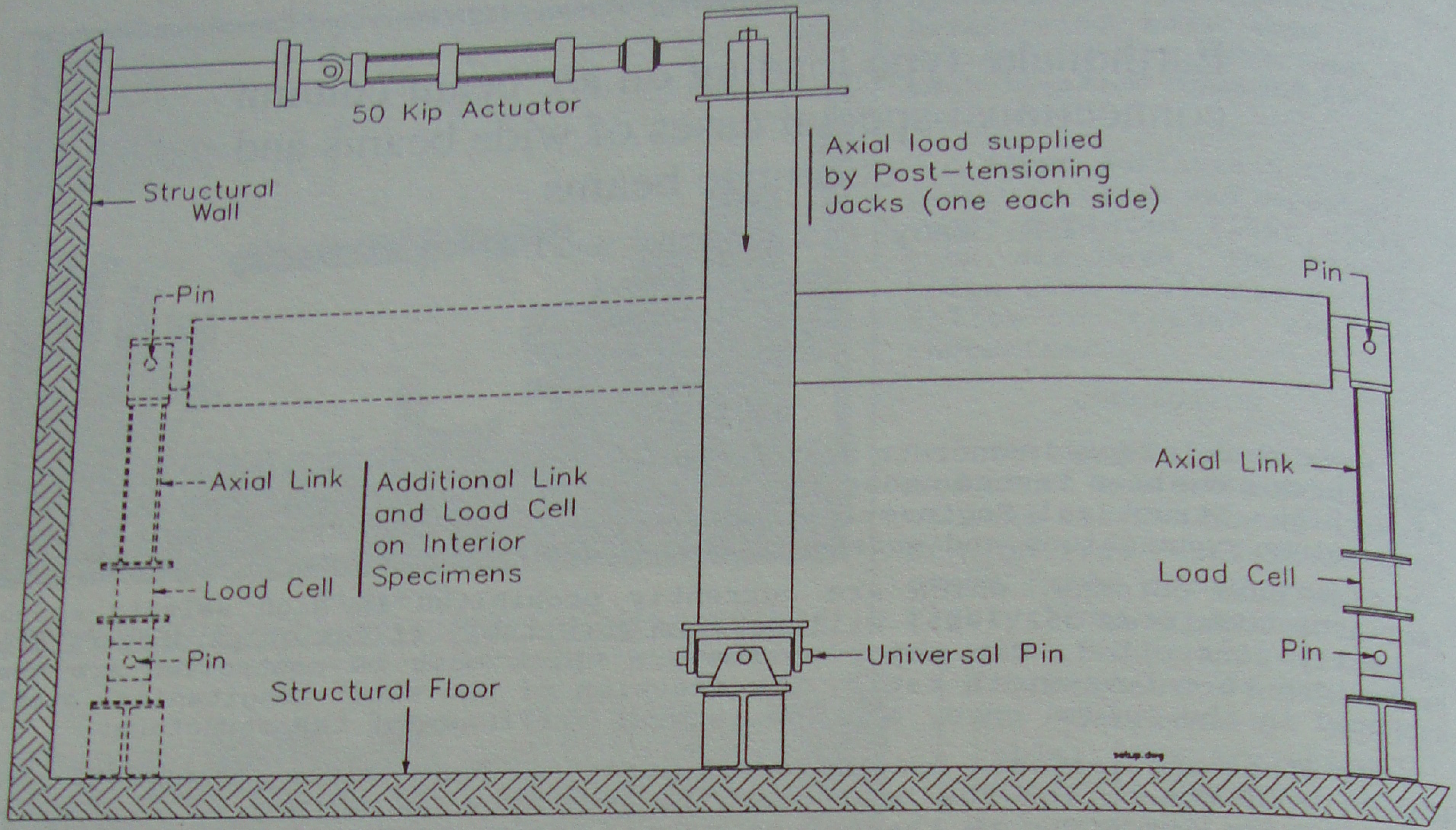


Figure 1. Experimental testing setup.

wide beam testing performed in Japan (Hatamoto 1990) supplemented the data from the testing performed at the University of Michigan. Table 1 and Fig. 2 outline the design parameters for the four specimens. All four specimens employed 14 in. square columns and 12 in. deep beams. At this date Specimens 1 and 2 have been tested; Specimens 3 and 4 will be tested during the Spring of 1991. Analytical and computer work will begin after testing is complete.

Specimen 1

Results from the test of Specimen 1 indicated that torsional distress in the transverse beam, along with anchorage loss for the wide beam flexural reinforcement, were the primary causes of wide beam connection failure. This specimen was too wide; its transverse beam did not have the capacity to transfer the torque applied by the wide beam to the column. The extensive torsional cracking in the transverse beam caused a loss of anchorage in the wide beam reinforcement.

Due to this anchorage loss, four of the six hooked top bars anchored in the

Table 1. Design parameters for wide beam specimens.

No.	b (in.)	b_w/c_w	Beam Longitudinal Reinforcement Top	Beam Longitudinal Reinforcement Bot.	M_R	$\gamma = V_J/\sqrt{f'_c}$	% Steel in Core Top	% Steel in Core Bot.
1	34.0	2.43	9#5	7#5	1.50	15.18	33%	43%
2	30.0	2.14	8#5	6#5	1.69	13.47	50%	33%
3	34.0	2.43	2#6, 2#5, 6#4	3#5, 6#4	1.49	14.79	56%	44%
4	34.0	2.43	10#5, 2#4	8#5, 2#4	1.18	19.24	35%	35%

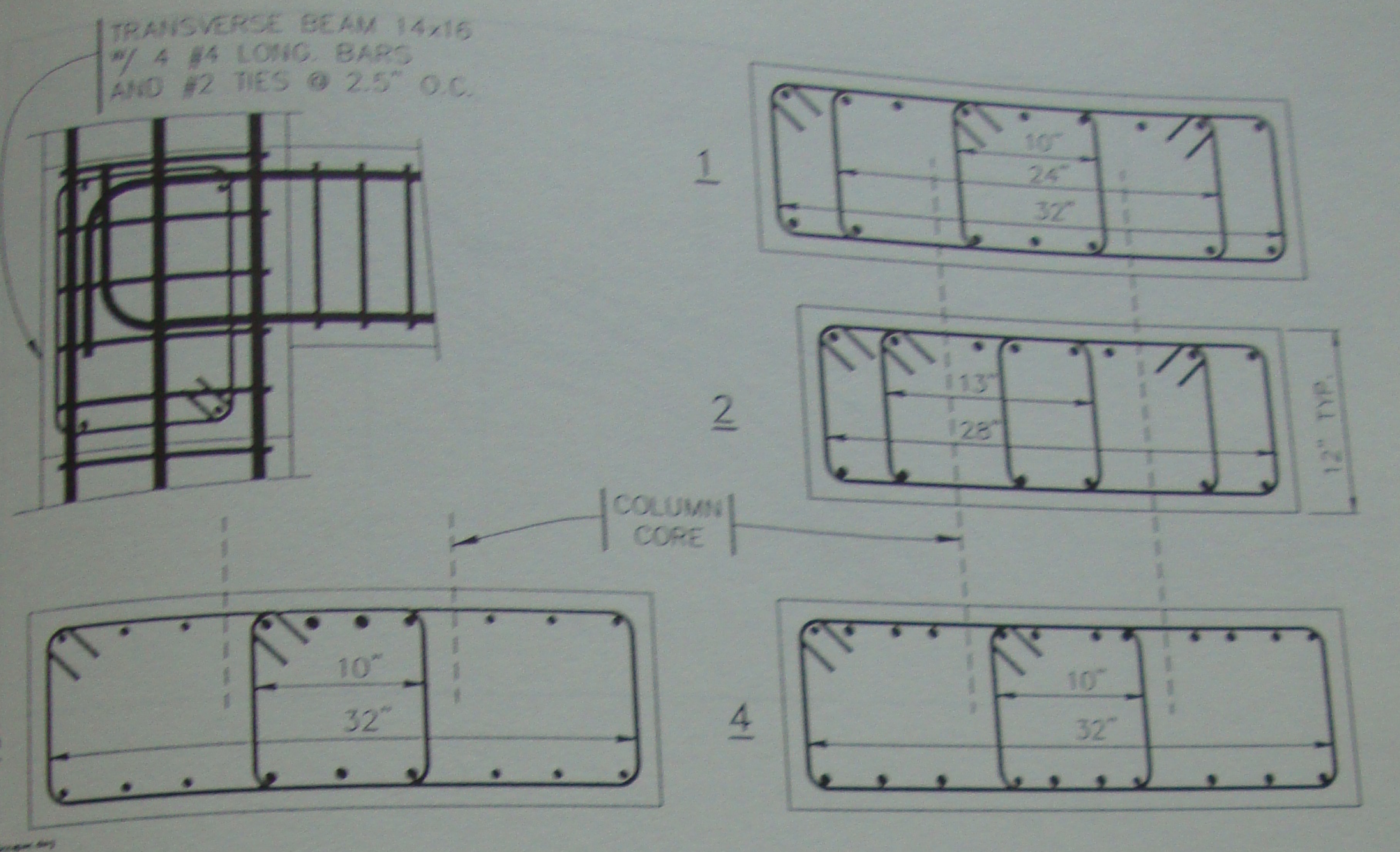


Figure 2. Wide beam specimens.

transverse beam never yielded during the testing of Specimen 1. The two bars that yielded did so only at a high lateral drift [3%, see Fig. 3]. The nominal story shear, that is, the story shear corresponding to full yielding of the beam plastic hinge, was not obtained from this specimen. At 2% lateral drift only 70% of the nominal story shear had been mobilized.

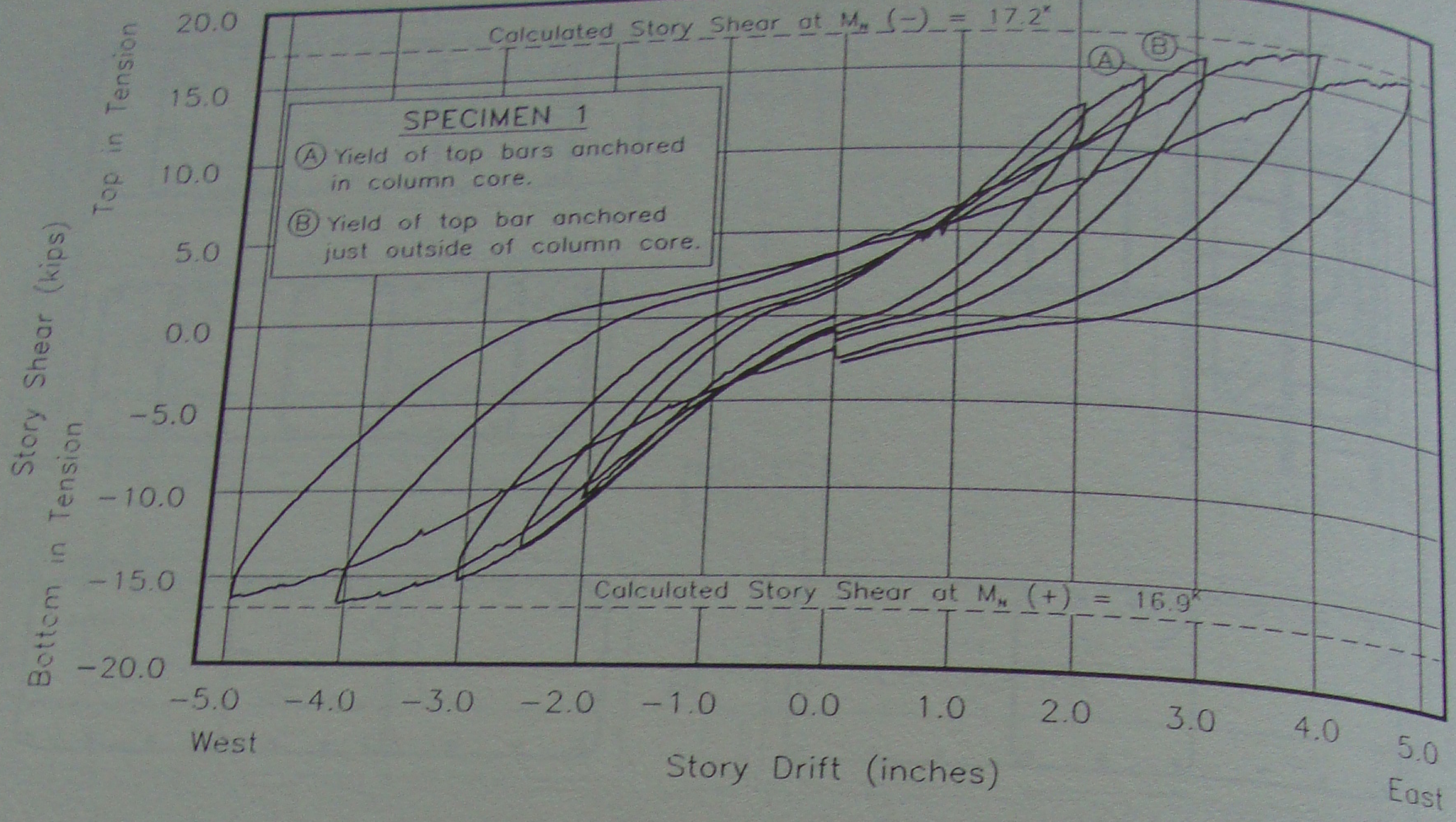
Specimen 2

Testing of Specimen 2, whose beam-width to column-width ratio [b_W/c_W] was 2.14, demonstrated that wide beam connections can perform well if torsional distress and anchorage loss of bars anchored in the transverse beam are eliminated. For Specimen 2, as compared to Specimen 1, this transverse beam distress was controlled by reducing its b_W/c_W ratio and by reducing the amount of steel anchored in the transverse beam [see Fig. 3]. Full yielding of the wide beam reinforcement occurred early in the displacement history for Specimen 2, between 1.5% and 2% story drift. The yielding of the exterior-most bars, which were anchored in the transverse beam, showed only a slight lag behind those bars anchored in the column core.

Specimen 2 reached a higher story shear than Specimen 1, even though its predicted design strength was 12% less than that of Specimen 1. Specimen 2 also exhibited increasing story shear with increasing drift up to the maximum drift of 5% imposed on the specimens. At 2% lateral drift Specimen 2 had mobilized 95% of its nominal story shear.

Specimens 3 and 4

As this paper is written, Specimens 3 and 4 have not yet been tested. These specimens were designed to probe the limits established during the testing of Specimens 1 and 2. Specimen 3 has dimensions identical to Specimen 1, with modifications made to improve the behavior. A higher fraction of the total wide beam reinforcement is placed in the column-core in an attempt to reduce the demand on the transverse beam. Specimen 4 utilizes an increased beam steel ratio



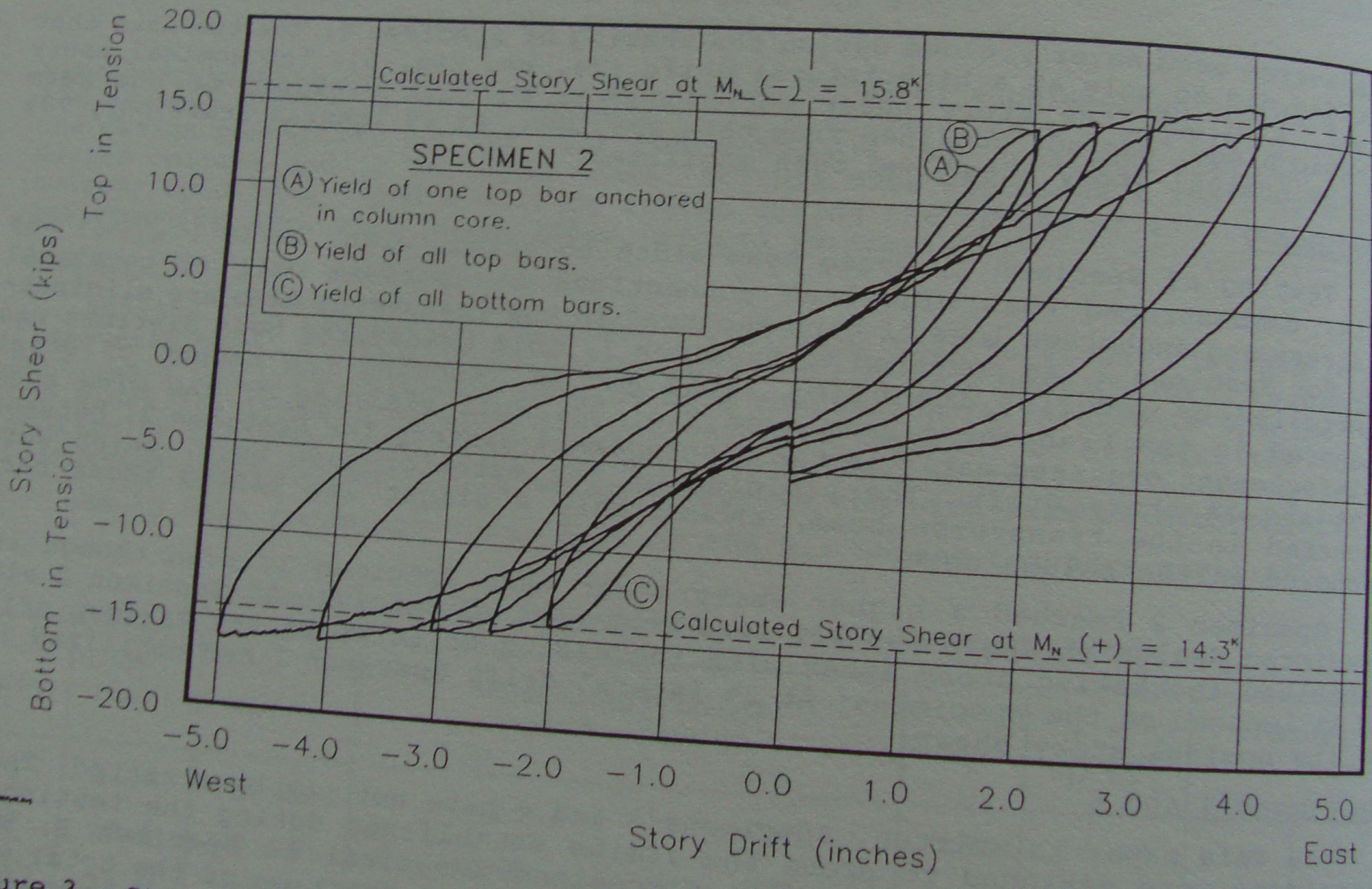


Figure 3. Story shear versus drift for Specimens 1 and 2. Drift cycles 2%, 2.5%, 3%, 4%, and 5% extracted from total curve.

[ρ] to anchor more wide beam reinforcement both in the column core and in the transverse beam. Both specimens benefit from increased torsional reinforcement in the transverse beam. Also, for both specimens, the anchorage of the wide beam reinforcement in the transverse beam is enhanced by increasing the cover over the hooked bars.

Analytical Analyses

Results from the experimental testing of Specimens 1 and 2 indicate that wide beam frames may have as much as 40% less lateral stiffness than comparable conventional R/C frames. Results from testing performed in Japan confirms this finding (Hatamoto 1990). The analytical component of this research, not yet complete, will probe the ramifications of this reduced lateral stiffness on wide beam frame behavior.

ECCENTRIC BEAM-COLUMN CONNECTIONS

A related program investigated the behavior of reinforced concrete eccentric beam-column connections subjected to earthquake-type loading. The specimens represent a beam-column subassemblage from an exterior moment-resisting frame in which, for architectural purposes, the beams were made flush with the exterior face of the column. This eccentric type connection is quite common yet is recognized by ACI-ASCE Committee 352 as an area of needed research (ACI-ASCE Committee 352 1985).

Experimental Program

All of the specimens have identical column sections and two beams framing into opposite sides of the column in such a way that one side of the beam is flush with the adjacent column face. The major design parameters varied were the beam width, the beam depth, and the amount of top and bottom longitudinal reinforcing steel in the beam. By varying the beam width, the eccentricity between the column centerline and the beam centerline and the effective joint shear area also varies. The amount of reinforcing steel in the beam influences the horizontal shear demand on the joint and changes the beam-to-column moment strength ratio. The design parameters for the four specimens are given in Table 2. The following items will be examined to determine the adequacy of these eccentric type connections: joint shear strength, deterioration of anchorage of longitudinal reinforcement, plastic hinge rotations, hysteretic behavior and energy dissipation, and loss of stiffness due to cyclic loading.

Test Results

At the time of this writing, only the results for the first two specimens are available. The spandrel beams on the first specimen had a cross-section of 10 in. x 15 in. and were longitudinally reinforced with 3-#6 and 3-#5 in the top and bottom of the beam, respectively. The column was 14 in. wide. Therefore, an eccentricity of 2 in. existed between the column centerline and the beam centerline. The load-displacement response of Specimen 1 to cyclic loading is shown in Fig. 4. The strength of the specimen continued to increase as the cycles reached story drifts of 4%. The stiffness, however, deteriorated rapidly as shown

Table 2. Design parameters for eccentric connections.

Specimen No.	Beam Size	Column Size	Eccen-tricity	Top Bars	Bottom Bars	Joint Shear Factor	Moment Strength Ratio
1	10"x15"	14"x14"	2"	3-#6	3-#5	14.0	1.42
2	7"x15"	14"x14"	3.5"	2-#6	2-#5	10.6	2.13
3	7.5"x15"	14"x14"	3.25"	3-#5	2-#5	10.7	2.03
4	7.5"x22"	14"x14"	3.25"	3-#5	2-#5	9.9	1.30

in Fig. 8. Torsional cracks on the sides of the column where the beams framed in were evident within the joint depth. Diagonal cracking in the joint was more extensive on the outside face of the column than on the inside face.

The second specimen had 7 in. x 15 in. spandrel beams with 2-#6 and 2-#5 in the top and bottom of the beam respectively. The same column section as in the first specimen was used. Since the beam was only 7 in. wide, an eccentricity of 3-1/2 in. existed between the beam and column centerlines. The load-displacement response of Specimen 2 is shown in Fig. 5. The hysteresis loops show excessive pinching, and consequently small amounts of energy were dissipated. The pinching is primarily caused by the closing of cracks and the slippage of the beam bars through the joint. The development of cracking in the joint was similar to that of the first specimen. Torsional cracks in the column within the beam depth developed early in the loading sequence, but did not grow larger as the cycles progressed.

The stiffness of Specimen 1 is compared to the stiffness of Specimen 2 in Fig. 8. The stiffness of both specimens decreased rapidly as the drift, which is given in percent of the story height, increased. The stiffness of Specimen 2, however, deteriorated at a rate faster than Specimen 1. A reason for this faster rate of deterioration was partially do to a quicker loss of anchorage of the beam bars in the joint.

The measured strains in the reinforcement indicate that slippage of the beam bars did occur. The strains recorded in one of the top longitudinal beam bars at the beam-column interface for Specimen 1 and Specimen 2 are shown in Fig. 6 and Fig. 7, respectively. From simple beam theory it would be expected that the bar would be in compression for positive story drifts and in tension for negative story drifts. The strains in the figures for both specimens show that the bar is in tension for negative story drifts, but for positive story drifts the strain changes from the expected compression to tension as the percent story drift increases. The change from compression to tension indicates the anchorage of the beam bar in the joint has been partially lost. For Specimen 1, the bar did not go into tension until 2% story drift was reached and at 3% story drift the tensile strain was still very small. For Specimen 2, the bar was in tension after 0.5% story drift and reached a tensile strain of approximately one-half the yield value at 1.5% story drift. This indicates that the beam bars experienced more slippage in Specimen 2 than in Specimen 1. The slippage of the beam bars may be the predominate cause of the pinching in the hysteresis loops and the rapid rate of stiffness deterioration in Specimen 2.

The premature loss of anchorage in Specimen 2 may be explained by examining the joint distortion on both sides of the connection. Displacement transducers were mounted on the faces of the column in an arrangement designed to measure the shear deformation. Fig. 9 shows, for Specimen 2, the joint deformation measured on the face of the joint flush with the spandrel beams (flush face) and on the opposite face where an offset exists between the column face and beam face (offset face). As may be expected, the figure shows the flush face of the joint experienced large deformations and significant diagonal cracking. The beam bars passed through the column close to the flush face of the joint where most of the damage was concentrated. This is believed to have been the cause of the anchorage loss of the beam bars in the joint.

The rotations over the plastic hinging zones in the beams were also measured. Figs. 10 and 11 show the beam end load versus the rotation in the west beam for Specimen 1 and 2, respectively. Similar behavior was also recorded for the east beams. Comparing the behavior of Specimen 1 to Specimen 2 it can be seen that the beams in Specimen 1 were stiffer and stronger. This would be expected because larger and more heavily reinforced spandrel beams were used in Specimen 1. Comparing the rotations for cycles with the same story drifts it can be seen that Specimen 2 underwent larger rotations. This would indicate larger fixed end rotations occurred in Specimen 2 as a result of bar slippage.

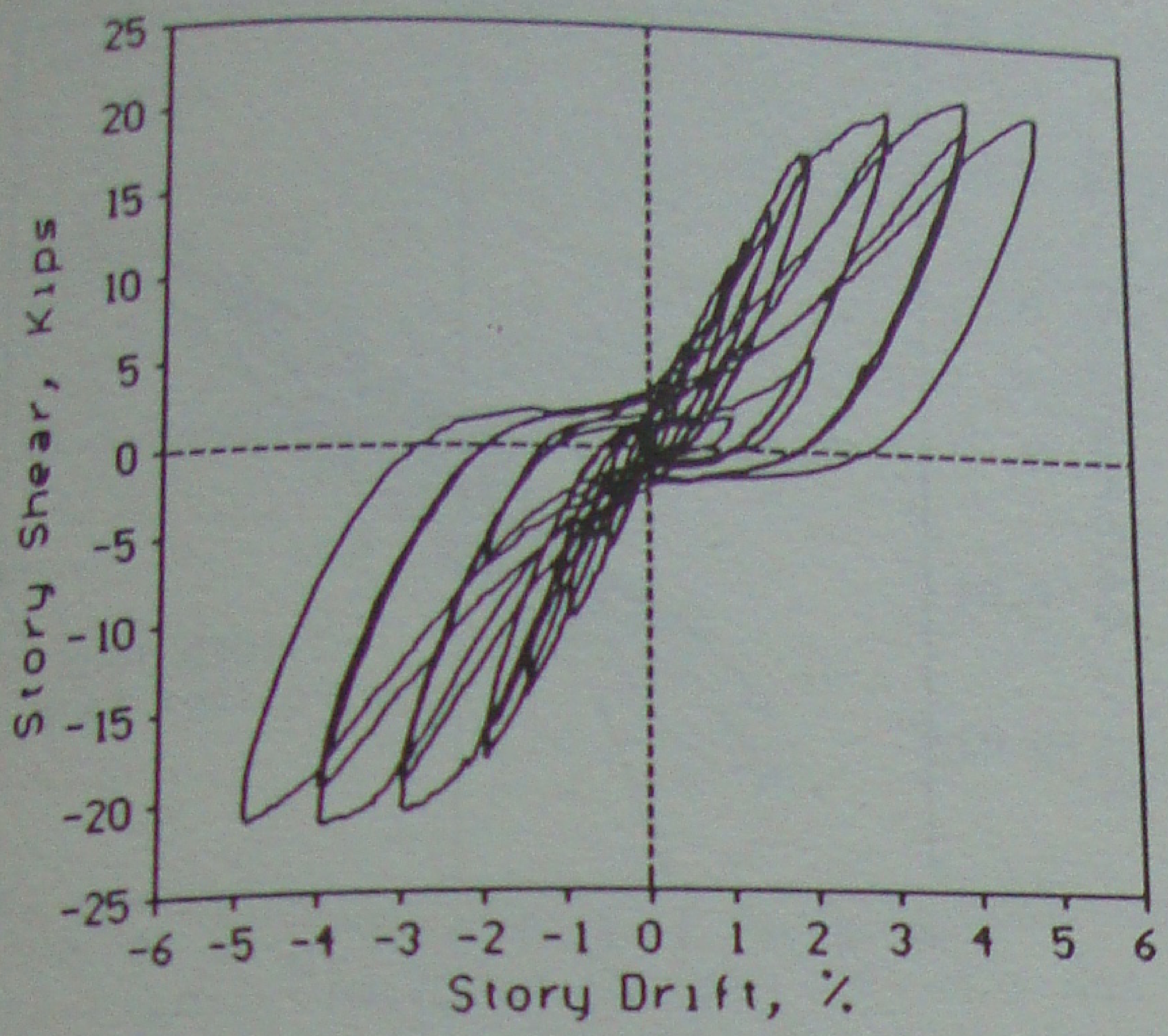


Figure 4. Load-displacement response of specimen 1.

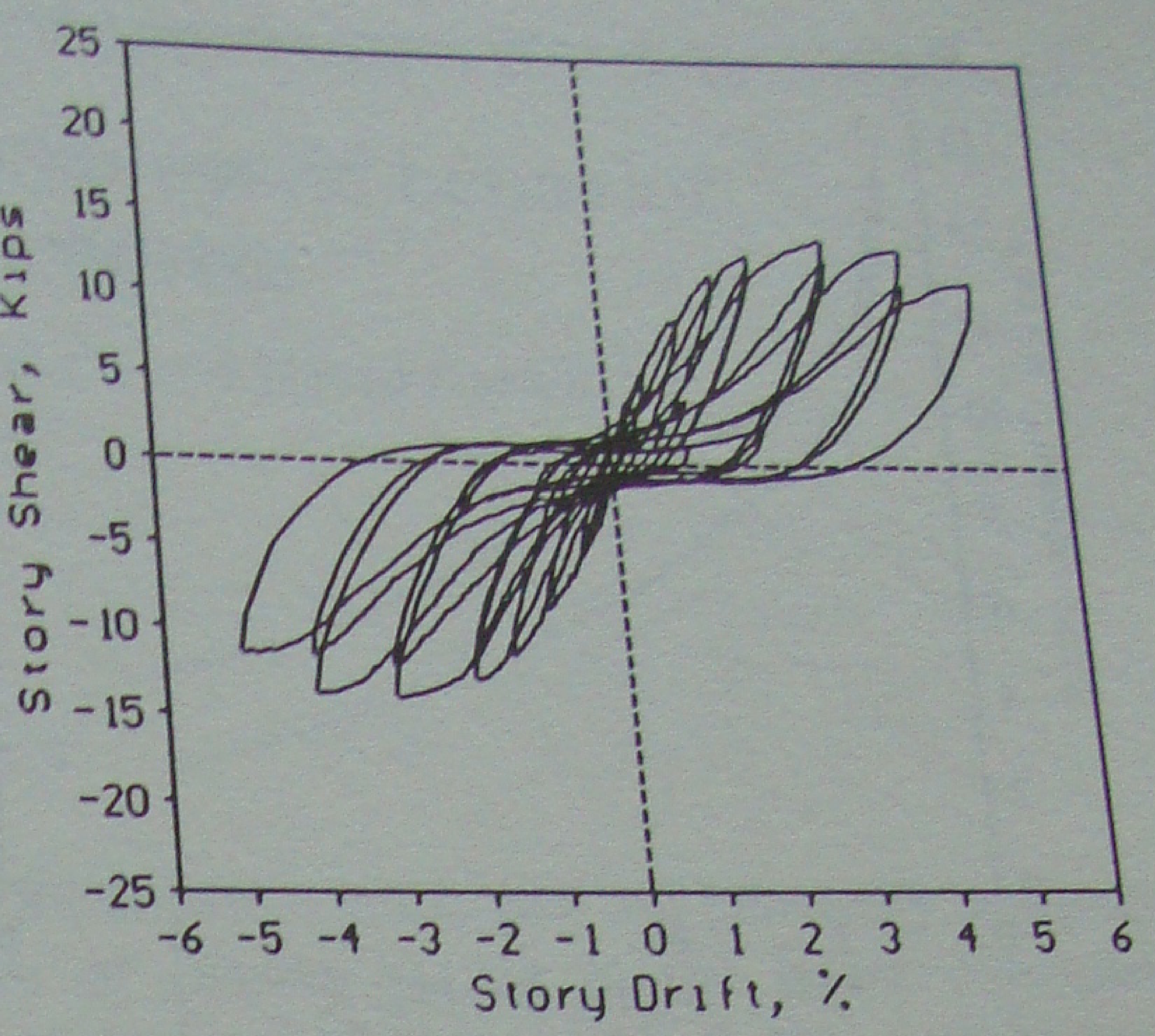


Figure 5. Load-displacement response of specimen 2.

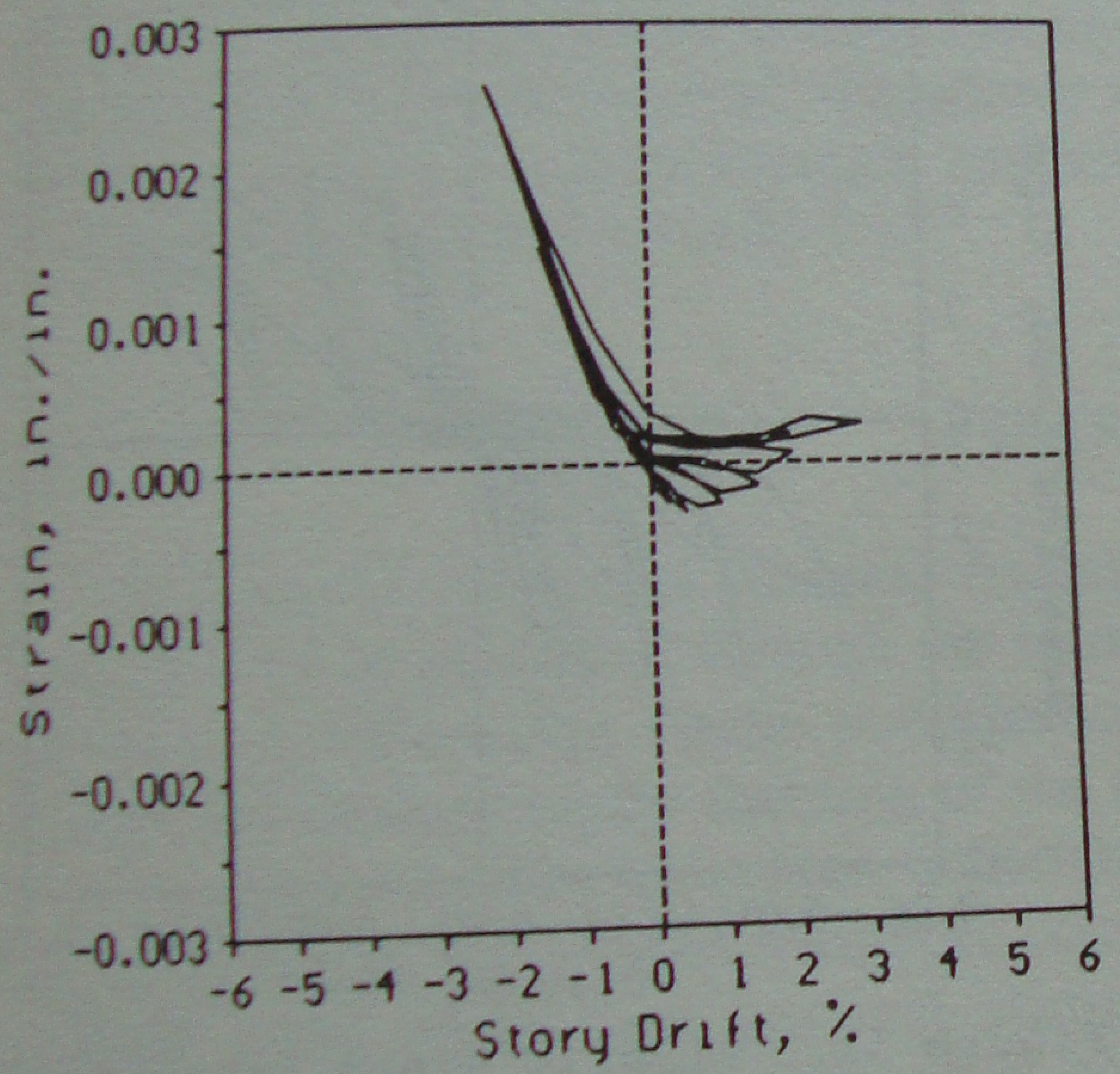


Figure 6. Strain in beam top longitudinal bar at column face in specimen 1.

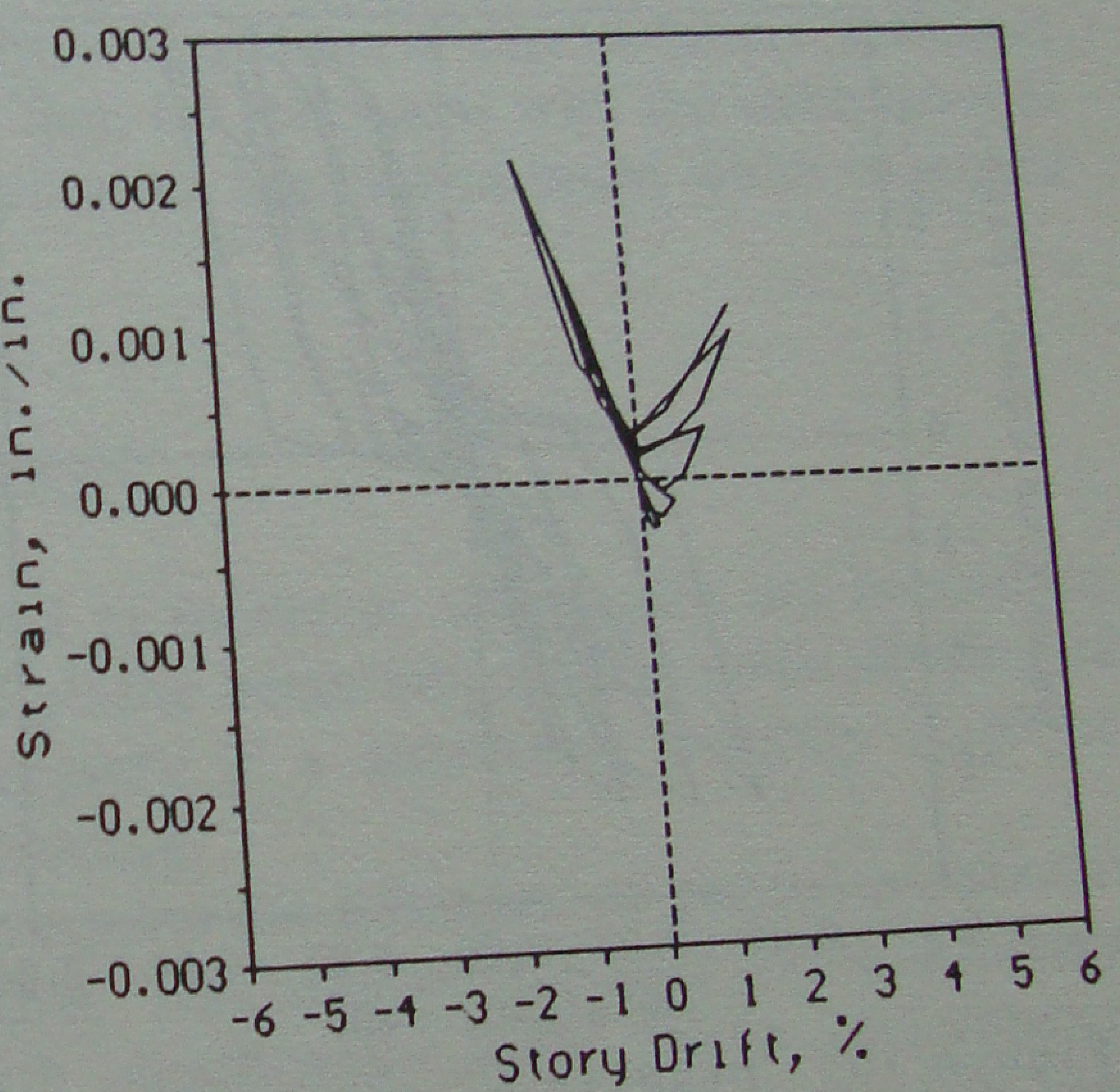


Figure 7. Strain in beam top longitudinal bar at column face in specimen 2.

Summary of Eccentric Beam-Column Tests

The poor performance of the eccentric beam-column connections are primarily a result of the longitudinal beam bars losing anchorage within the joint. The joints showed some torsional distress at the beginning of the reversed cyclic loading pattern, but this damage did not increase as the cycles progressed. The poor anchorage condition is a result of the beam bars passing through the side of the joint where most of the deformation and cracking are concentrated. Comparing the stiffness deterioration for the two specimens, the stiffness deteriorated more rapidly for the narrower spandrel beams.

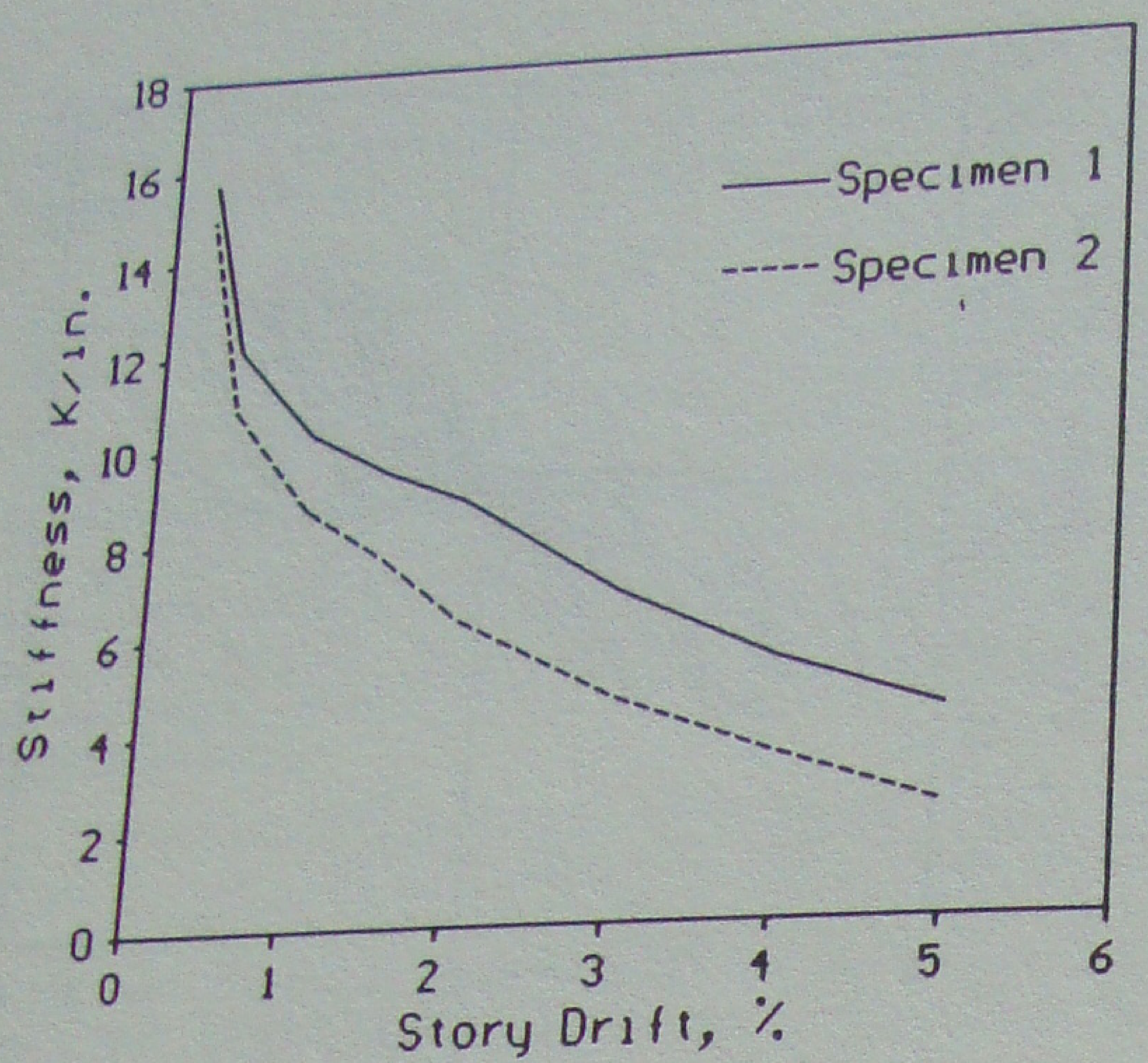


Figure 8. Stiffness deterioration of specimen 1 and 2.

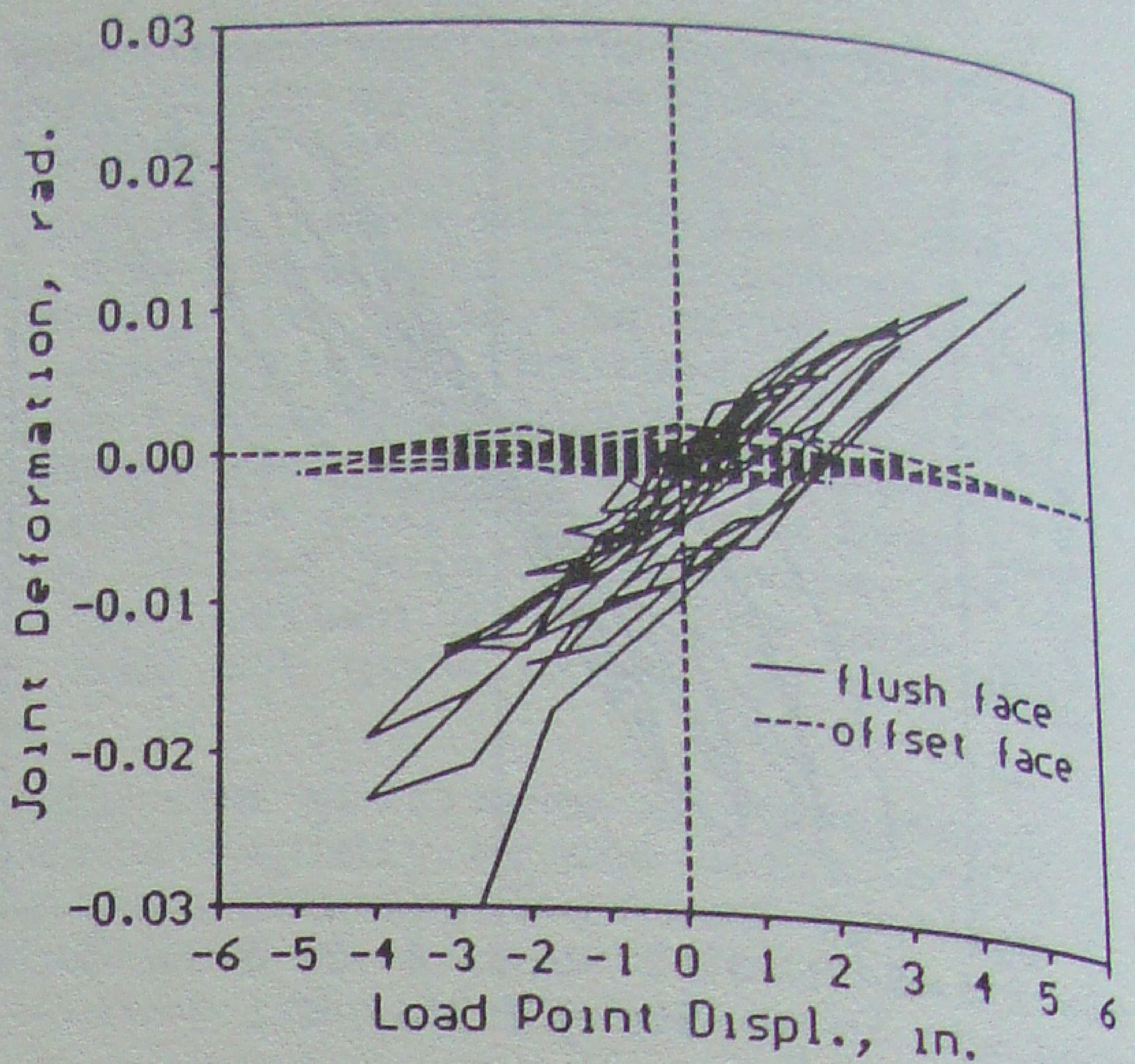


Figure 9. Joint shear deformation of specimen 2.

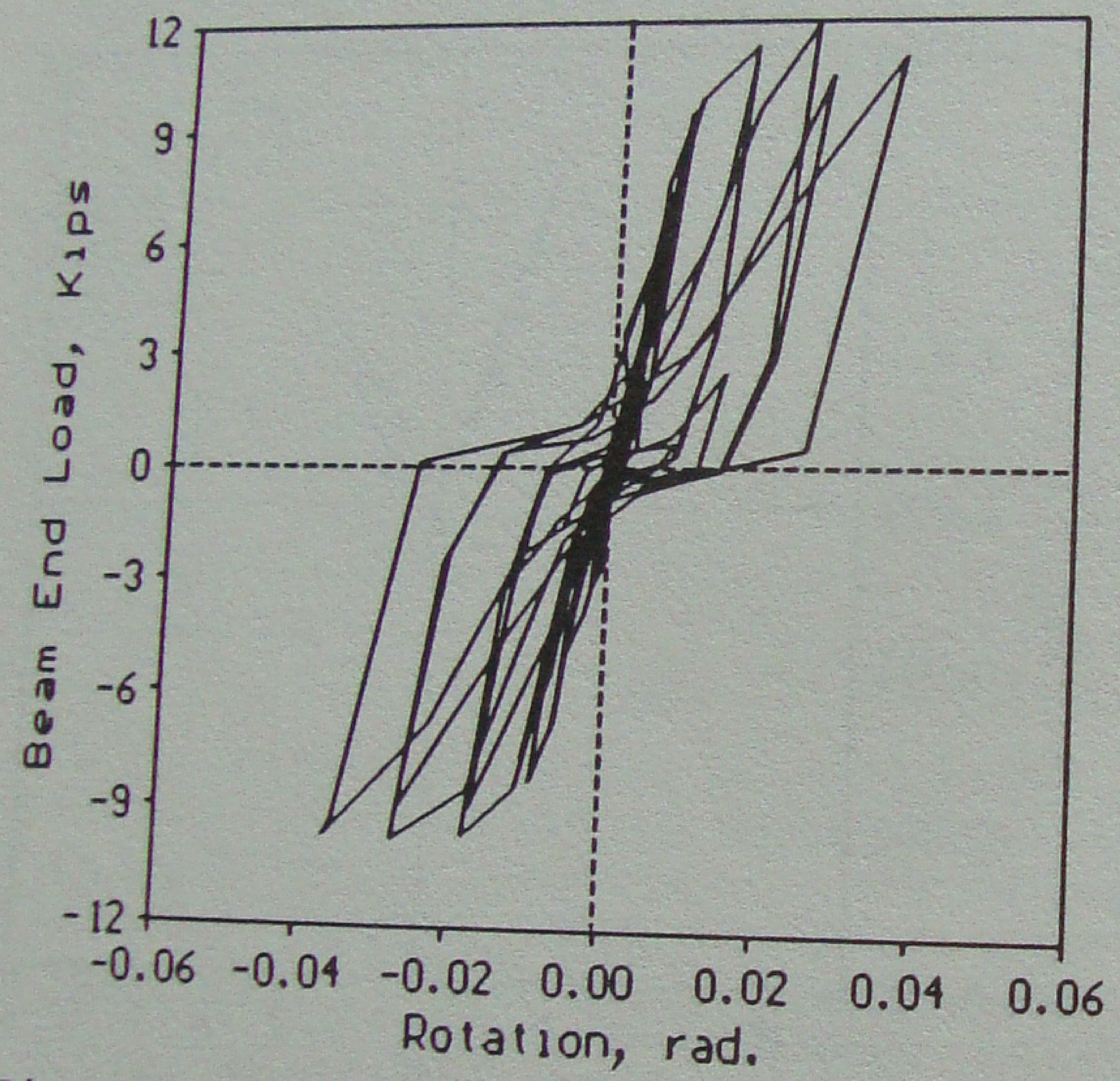


Figure 10. Plastic hinge rotation in west beam of specimen 1.

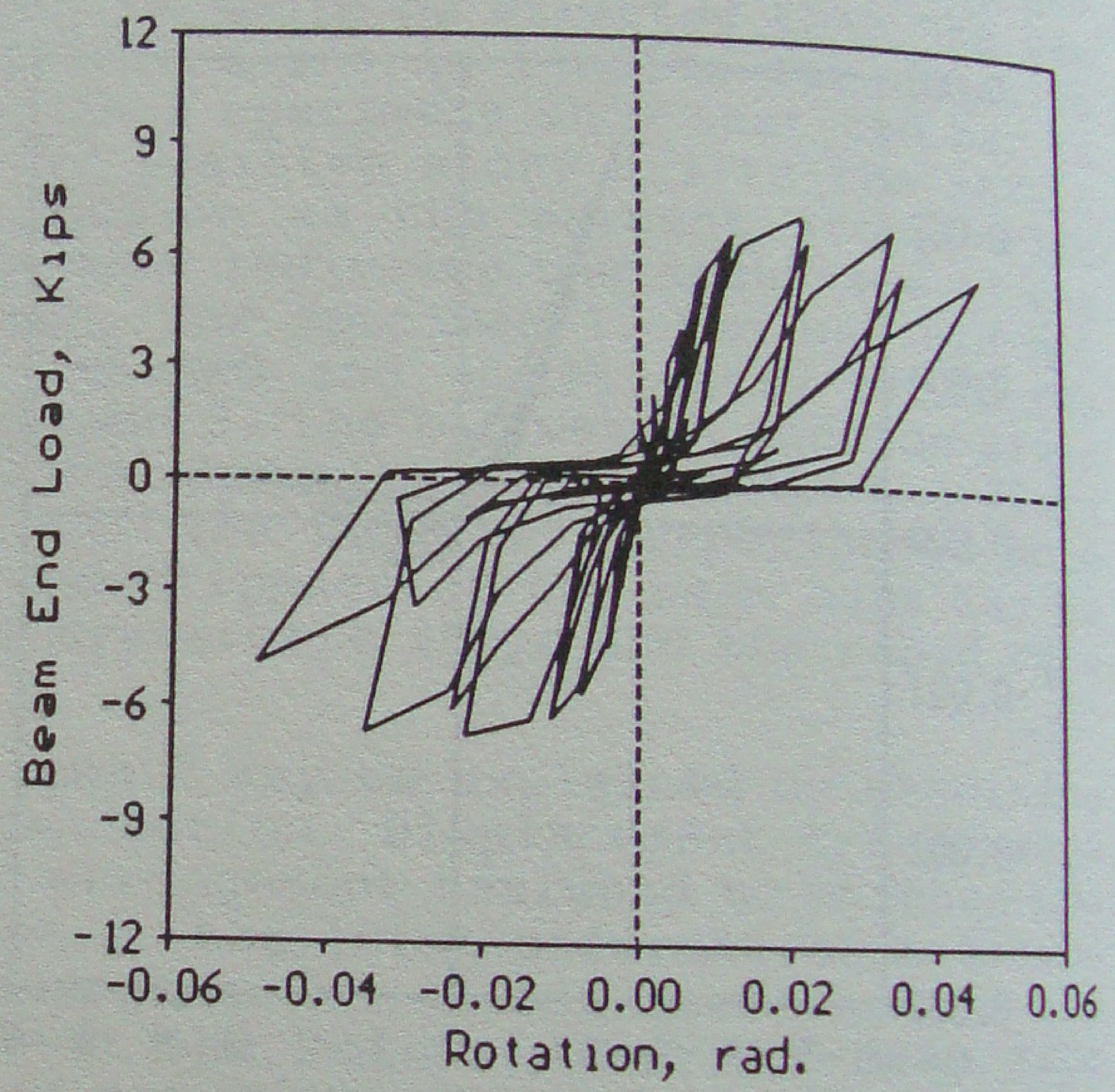


Figure 11. Plastic hinge rotation in west beam of specimen 2.

ACKNOWLEDGEMENTS

Support for this research by the United States National Science Foundation is gratefully acknowledged. The results and conclusions expressed here are those of the authors and do not necessarily reflect the views of the sponsor. Information from Hitoshi Hatamoto at the Kajima Construction Corporation in Japan, along with his colleagues Satoshi Bessho at Kajima and Prof. Yasuhiro Matsuzaki at the Tokyo Science University, is also gratefully acknowledged.

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