

SEISMIC STRENGTH OF MOMENT END-PLATE CONNECTIONS WITH ATTACHED CONCRETE SLAB

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ABSTRACT

This paper presents the results of a test of an eight-bolt stiffened moment end-plate connection with a concrete slab attached. During a previous test of a moment end-plate connection with a concrete slab, premature tension bolt rupture occurred because the concrete slab was placed directly against the supporting column flanges. This successful test was conducted with a gap between the concrete and the column flanges. It is shown that with proper detailing, moment end-plate connections with floor slabs can be used in special moment frames. Recommendations for detailing concrete slabs at moment end-plate and similar bolted connections are made.

INTRODUCTION

A test was conducted to verify that a moment end-plate connection used in conjunction with a concrete structural slab could perform satisfactorily to pre-qualify the connection for use in Special Moment Frames provided that the concrete structural slab is properly detailed. A test reported by Sumner, et. al. (2000) of an end-plate connection with a concrete structural slab showed that the bolts at the flange opposite the structural slab could fail prematurely when compressive forces are developed in the structural slab at the connection. To prevent engagement of the structural slab near the connection, for this test, the slab was detailed to provide a gap between the structural slab and the column face at the connection and shear studs were eliminated for a distance of 1.5 times the beam depth from the expected plastic hinge location as recommended in Sumner and Murray (2002).

DESCRIPTION OF TEST SPECIMEN

The test specimen consisted of two W27x84 (A992) beams attached to each flange of a W14x311, A36 steel column in an interior configuration. (Because of funding constraints an available A36 section was used for the column.) The beam-column connections were made with eight-bolt, extended, stiffened end-plate connections. The

connections were designed using the procedures in the AISC Steel Design Guide 4, 2nd Ed, (Murray and Sumner 2004). The connection was designed such that the no-prying moment strengths (M_{PR}) of the bolts and the end-plate were as close as possible to the probable maximum beam moment (M_f). Because the measured yield strength of the beam was larger than anticipated, the probable maximum moment exceeded the no-prying moment capacity of the connection by 8.8%.

Each connection consisted of two rows of two bolts on the outside of each flange and two rows of two bolts on the inside of each flange. All bolts were 1-1/8 in. diameter, Grade A325. The bolts were on a 7-1/2 in. gage with 3-3/4 in. spacing between rows and located 2 in. from the face of each flange. Each end-plate was 1-1/4 in. thick of specified A36 steel. A 3/4 in. thick, A36 steel, stiffener in the plane of the girder web was provided to stiffen the extended portions of the end-plate outside. The layout of the end-plate is shown in Figure 1.

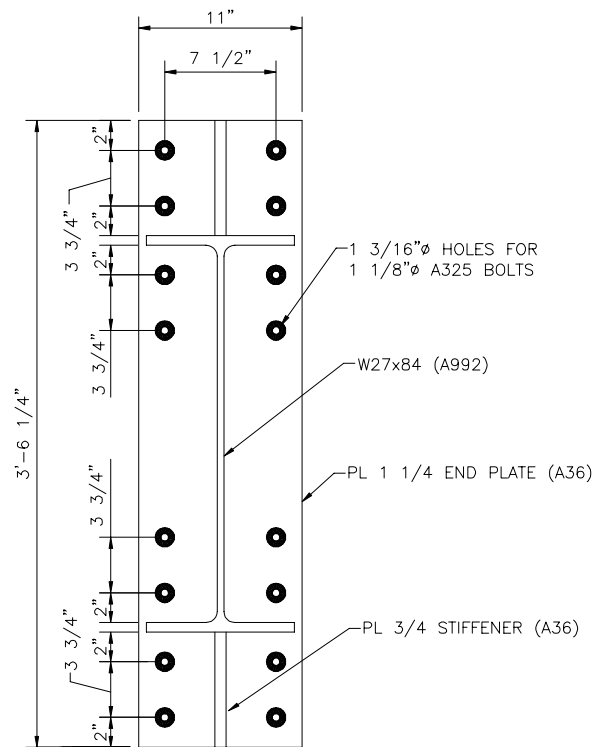


Figure 1. End-plate Layout

One-half of the bolts were instrumented with a “bolt” strain gauge to measure bolt strain during loading. The tension in the instrumented bolts was monitored during installation as the bolts were installed to the specified pretension force in the AISC *Specification for Structural Steel Buildings* (2005). Non-instrumented bolts were installed by the turn of the nut method based on number of turns required to pretension the instrumented bolts.

Two W14x22 filler beams attached to the web of the column and four W14x22 filler beams attached to the W27x84 girders supported a composite slab with special detailing in the area of the beam-column connection to minimize the impact of the composite slab on the performance of the connections. The layout of the test specimen is shown in Figure 2. Figure 3 is a photograph of the test setup.

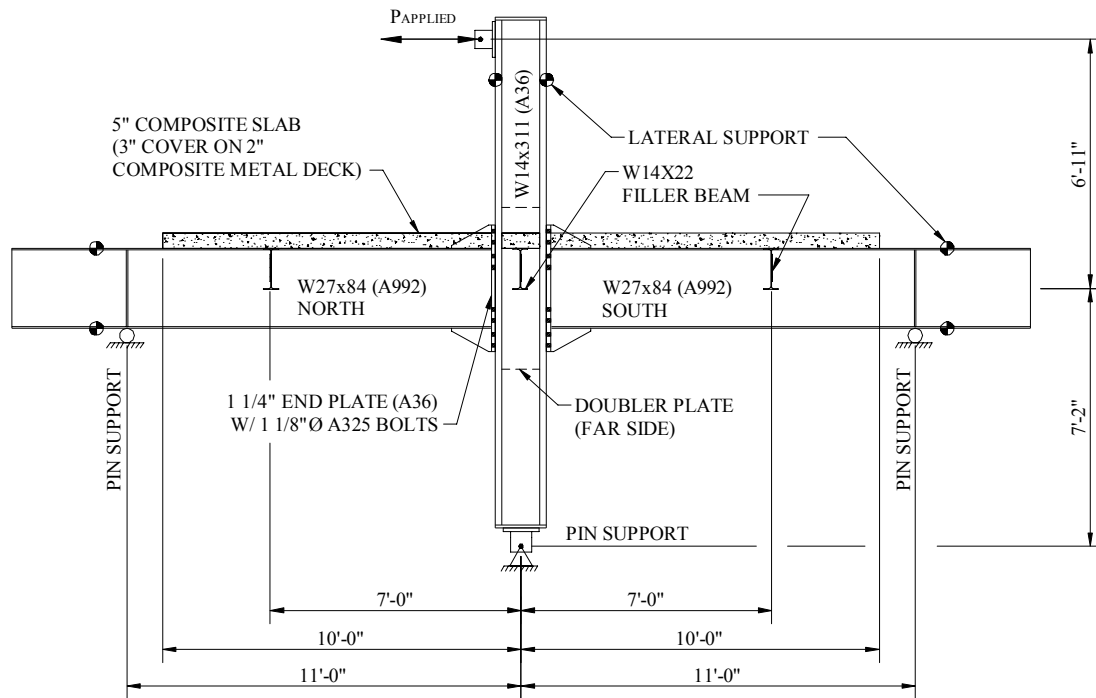


Figure 2. Elevation of Test Setup



Figure 3. Photograph of Test Setup

The beam-column sub-assembly included a 5 in. thick, 20 ft long by 6 ft wide normal weight composite slab. The slab was supported on the W27x84 girders and the six W14x22 filler beams. The filler beams supported the slab at the column centerline and in each direction at 7 ft from the column centerline. Twelve 3/4 in. diameter A108 shear studs ($F_U = 65$ ksi) with a 4-1/2 in. installed height were welded to the girders and the outer filler beams as shown in Figure 4. To protect the region of expected plastic hinging, studs were not welded to the girders for a distance of 1.5 times the beam depth (40.5 in.) from the end of the end-plate stiffener. Studs were not installed on the filler beams that were attached to the web of the columns. The 5 in. thick slab (3 in. cover on 2 in. ribbed deck) was placed on Vulcraft 2VLI, 20 gage, zinc coated composite steel deck. The deck was fastened to the beams by the stud welds only. The slab was reinforced with 4 x 4 - W2.9 x W2.9 welded wire fabric. A piece of 8 in. wide, 3/8 in. thick neoprene was used to form a gap between the slab and the end-plate and the column flanges. The gap provided a minimum clearance of 1/2 in. and the void was filled with spray foam insulation. The concrete slab was wet cured for 5 days and had a 40 days compressive strength of 3600 psi.

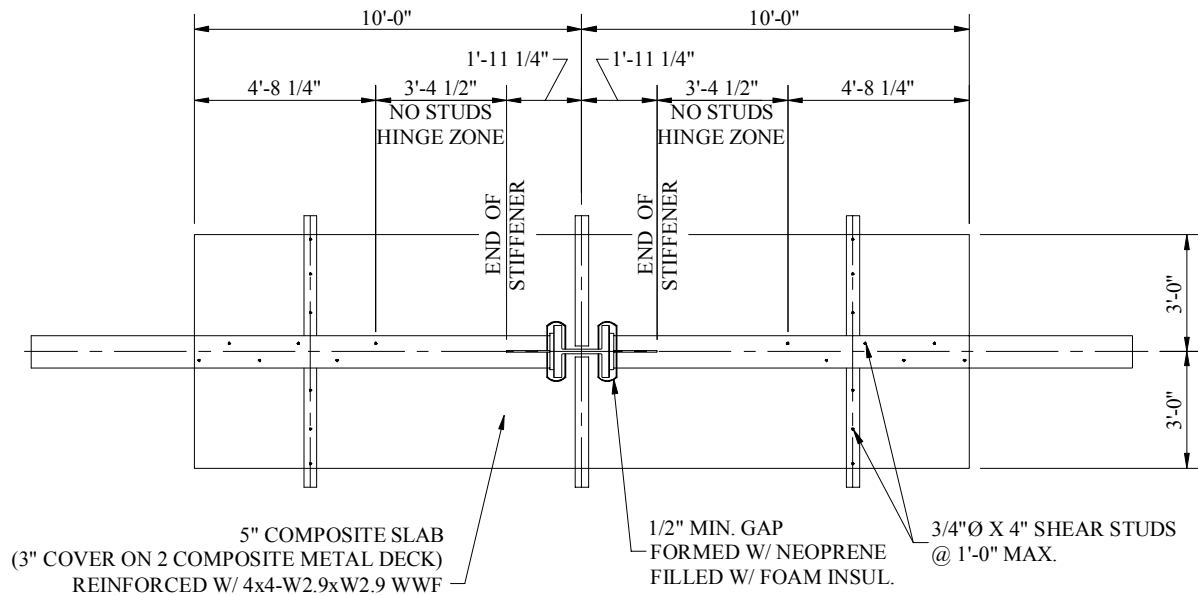


Figure 4. Plan View of Test Setup

End-plate welds were made using Gas Metal Arc Welding (GMAW) processes with 1/16 in. diameter Tri-Mark Metalloy 76 metal cored gas shielded carbon steel wire in accordance with AWS Specifications. The flange-to-end plate weld was made using a single-bevel-groove weld similar to AWS TC-U4b-GF. The beam flanges were prepared with a full depth 45 degree bevel with the root at the web side of the flange. The beam web was prepared with 45 degree half-depth bevels on each side. Weld access holes were not provided. The generic welding sequence for moment end-plate connections shown in Figure 5 was used. The web double bevel groove weld was placed first. Next 5/16 in. backing fillets were placed on the inside of each flange. The root of the bevel was then back-gouged and the flange groove welds placed. In the region of the flange-

web intersection, a partial penetration groove weld similar to AWS BTC-P4-GF was used. The stiffener was clipped to provide clearance from the flange weld and attached using double bevel groove welds.

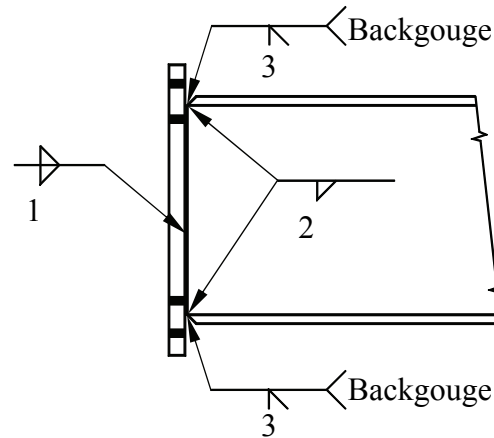


Figure 5. Welding Sequence

The design of the column panel zone was in accordance with the provisions of the AISC Seismic Provisions for Structural Steel Buildings (2002) and the recommendations of the AISC Prequalified Connections for Special and Intermediate Moment Frames for Seismic Applications (2005a). The panel zone of the W14x311 column was reinforced with a 3/4 in., A36 steel doubler plate on one side of the column web. The doubler plate was beveled full depth 15 degrees with a 3/16 in. root opening along the intersection with the column flange, and a complete joint penetration single bevel groove weld was placed using a GMAW process. A 1/4 in. fillet weld was placed between the doubler plate and column web. Continuity plates were not used.

Tensile coupons were taken from the bottom flange of each beam from portions of the beam that did not sustain significant stresses during the cyclic testing. A tensile coupon representative of the end-plate was taken from the same plate material as the end-plates. Tensile coupon test results of the beam and end-plate material are listed in Table 1.

Table 1. Tensile Coupon Test Results

Coupon	Thickness (in.)	Width (in.)	Area (in.)	Upper Yield Stress (ksi)	Dynamic Yield Stress (ksi)	Ultimate Stress (ksi)	Elongation (%)
North Beam	0.653	1.499	0.979	65.2	64.6	81.7	28.8
South Beam	0.652	1.499	0.977	64.7	64.3	82.1	28.1
End Plate	1.283	1.500	1.924	51.1	49.9	78.2	30.9

INSTRUMENTATION

One-half of the bolts used in the moment end-plate connection were instrumented with strain gauges. Two instrumented calipers were used to measure the separation between the north end-plate and the column. The calipers were placed across the thickness of the end-plate and column flange at the inside of the top and bottom beam flanges. To measure the deformations in the column panel zone, a pair of plunger type potentiometers was placed diagonally in the panel zone on the side of the column that was not reinforced with the doubler plate. Additionally, each side of the panel zone was instrumented with two 120 ohm, three gauge, 45 degree rosettes. One of the rosettes was placed at the center of the panel zone and the other placed in the lower left hand corner of the panel zone. Slip in the concrete slab relative to the beams was measured using two plunger type potentiometers that were placed approximately 3.5 ft and 9 ft from the centerline of the column on each beam.

LOADING PROTOCOL

The specimen was loaded according to the protocol specified in Appendix E of the *Protocol for Fabrication, Inspection, Testing and Documentation of Beam-Column Connection Tests and Other Experimental Specimens* Report No. SAC/BD-97/02 from the SAC Joint Venture (1997). The report specifies stepwise increasing deformation cycles based on the inter-story drift angle, θ , of the specimen. Load was applied to the column tip and the total inter-story drift angle was derived from the displacement of the column tip by dividing the column tip displacement by the height of the column (169 in.). Rigid body movements were eliminated from the net column tip displacement by subtracting the displacements at the column base and each beam tip.

DESCRIPTION OF PERFORMANCE

During the cycles of 0.015 radians of story drift, light flaking of the whitewash on the beam bottom flange was observed, indicating the onset of yielding in the bottom flange. Yielding of the bottom flange became more apparent during the cycles of 0.02 radians of story drift and cracking in the slab was observed. During the cycles of 0.03 radians of story drift, the bottom flange of each beam buckled as shown in Figure 6. Some yielding of the end-plate stiffener was observed as well as yielding in the panel zone on the doubler plate side of the column. Crushing of the concrete at the column in the area of the stiffener was observed as shown in Figure 7. Longitudinal cracks propagating from the stiffener parallel to the beams offset about 3 in. from the beam centerline were also observed as shown in Figure 7. Several of the bolts on both the inside and outside of the bottom flange had loosened to where they could be turned by hand. The specimen completed only 1/2 cycle at 0.04 radians of story drift before failure of the test frame was eminent. Two attempts to complete the first full cycle were made but large deformations of the pin support at pin at the column base prevented the achievement of the desired story drift. A third attempt to reach 0.04 radians of story drift was made by extending the ram at the south beam tip. In each case, loading was stopped short of

the desired drift angle for fear of failure of the test frame. At the time the test was aborted, the end-plates had shown no signs of distress. Very little separation of the end-plate from the column flange was observed and only minor flaking of whitewash was observed on the edges of the endplate adjacent to the bottom flange.

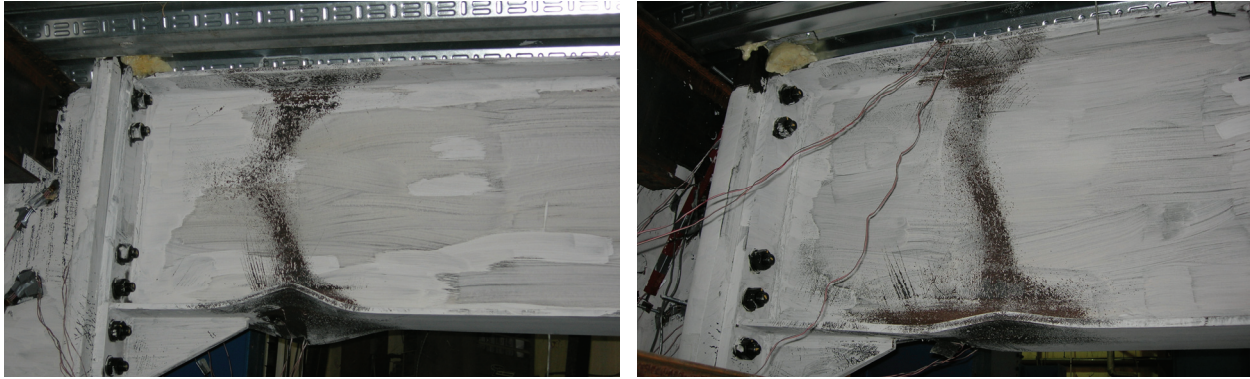


Figure 6. Yielding and Bottom Flange Buckle

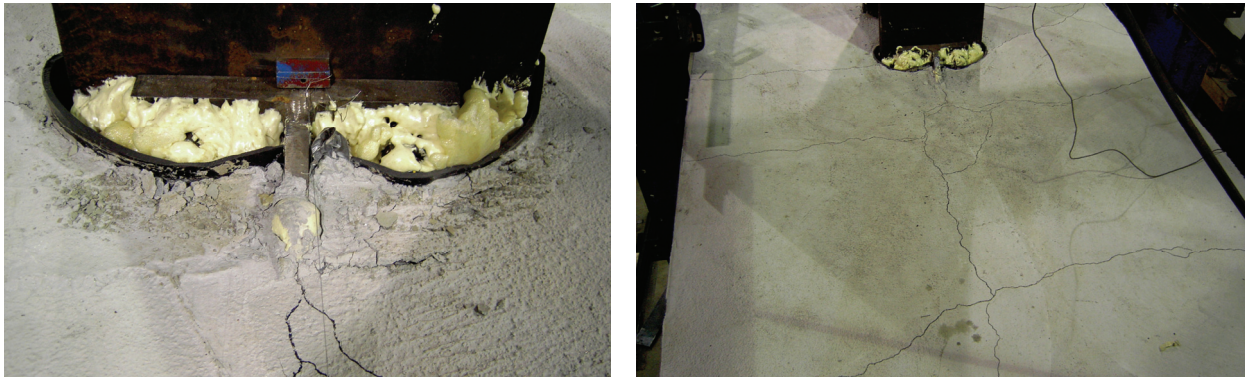
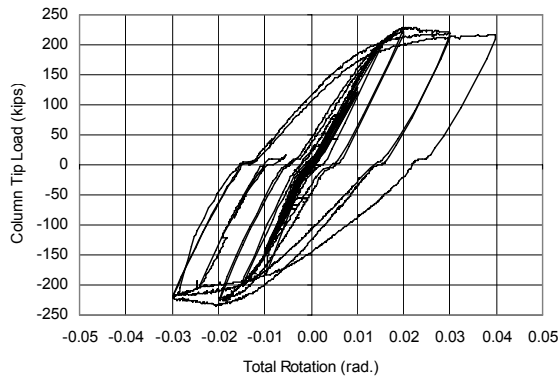
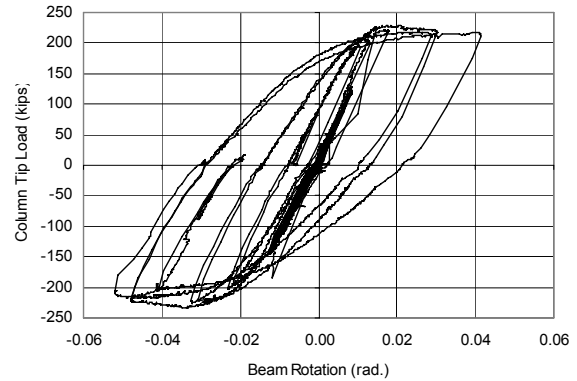


Figure 7. Cracking of Slab

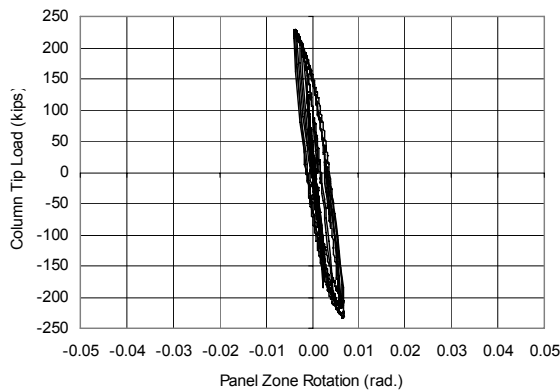
A peak column tip load of 229.3 kips was applied. The maximum net column tip displacement, δ_c , achieved was 6.750 in. which corresponds to an inter-story drift angle, $\theta = 0.04$ radians. The corresponding contribution of the beams, column and panel zone to the total total tip displacement is $\delta_{beam} = 5.568$ in., $\delta_{col} = 1.687$ in. and $\delta_{pz} = -0.506$ in. The corresponding rotations of the beams, column and panel zone are $\theta_{beam} = 0.051$ radians, $\theta_{col} = 0.010$ radians, and $\theta_{pz} = -0.030$ radians respectively. The total rotation response and the contributions of the individual components relative to the column tip load are shown in Figure 8. The strain in the bottom flange bolts of the south north beam is shown relative to the south beam centerline moment in Figure 9.



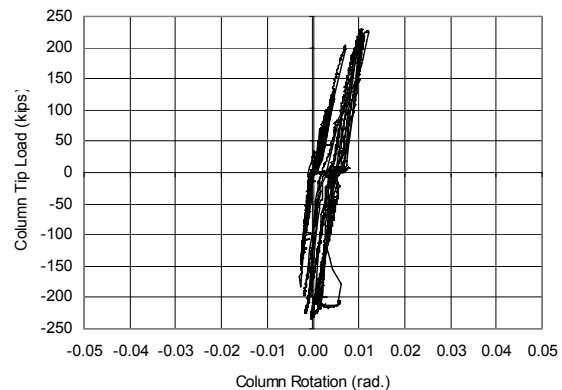
(a) Column tip Load vs Total Rotation



(b) Column Tip Load vs Beam Rotation



(c) Column tip Load vs Panel Zone Rotation

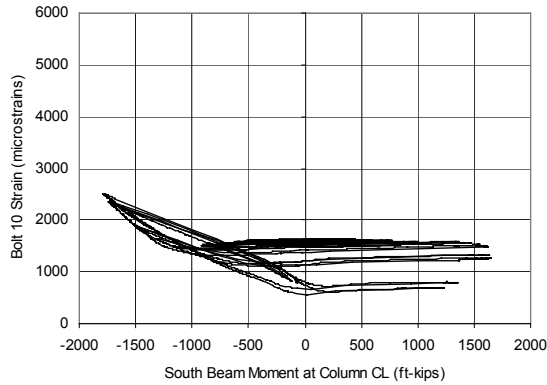


(b) Column Tip Load vs Column Rotation

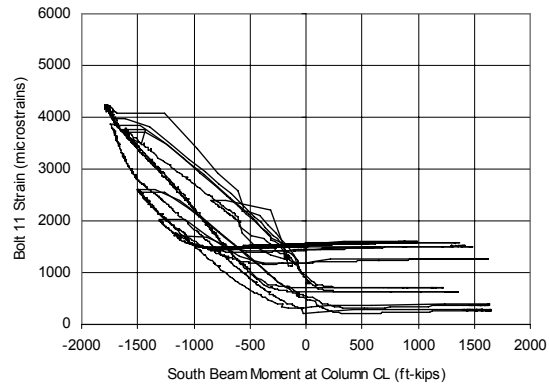
Figure 8. Specimen Rotation Response Relative to Column Tip Load

CONCLUSIONS

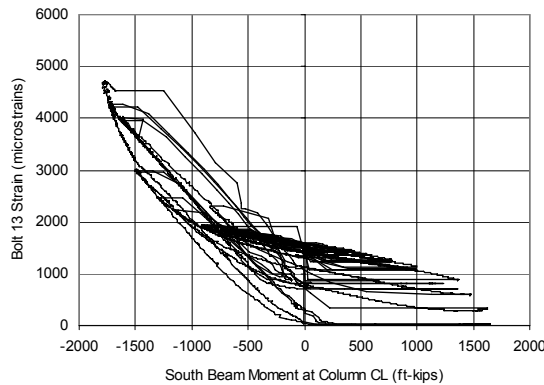
The test showed that when a ½ in. minimum gap is provided between the structural slab and the face of the column and when shear studs are eliminated for a distance of 1.5 times the beam depth from the expected hinge location, a composite slab does not negatively affect the performance of a bolted moment end plate connection. The test specimen was subjected to only ½ cycle at 0.04 radians instead of the desired 2 full cycles due to eminent failure of the test frame. Because both beam bottom flanges buckled during the cycles at 0.03 radians resulting in slight strength degradation and the connection did not exhibit signs of distress, it is believed that the specimen could have endured the full 2 cycles at 0.04 radians had the test frame not failed.



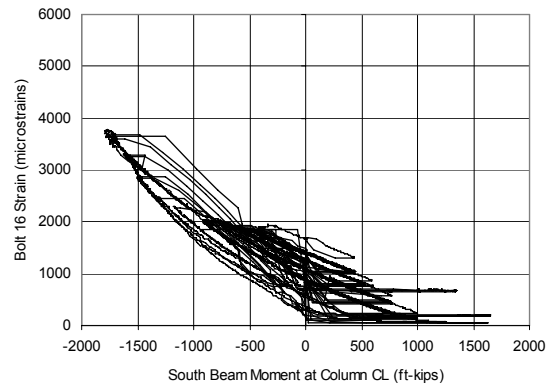
(a) Bolt 10 Strain vs S. Beam Moment



(b) Bolt 11 Strain vs S. Beam Moment



(a) Bolt 13 Strain vs S. Beam Moment



(b) Bolt 16 Strain vs S. Beam Moment

Figure 9. South Beam Bottom flange Bolt Strain vs Beam Moment

RECOMMENDATIONS

At moment end-plate and similar bolted moment connections, to eliminate the composite action between the slab and beam in the regions of the beam where plastic hinges are expected, the following slab and shear stud detailing is recommended:

- Shear studs should not be placed along the top flange of the connecting beams for a distance from the face of the column, one and a half times the depth of the connecting beam.
- Compressible expansion joint material, at least $\frac{1}{2}$ in. thick, should be installed between the slab and the column face.

- The slab reinforcement in the area within two times the depth of the connecting beam from the face of the column should be minimized.

REFERENCES

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