RESPONSE OF CYCLICALLY LOADED EXTENDED END-PLATE MOMENT CONNECTIONS WHEN USED WITH WELDED BUILT-UP SECTIONS

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ABSTRACT

An experimental investigation was conducted to study the behavior of extended end-plate moment connections subjected to cyclic loading. Eleven specimens were tested, representing typical connection configurations used in the metal building manufacturing industry. Four of the beams were shallow (30 in. or less), and seven were deep (60 in. or more). Two of the beams had compact webs, two had non-compact webs, and seven had slender webs. All specimens were designed according to the "thick plate" procedure contained in AISC Design Guide 16, *Flush and Extended Multiple-Row Moment End-Plate Connections*. A displacement-controlled history was used to load the specimens. Experimental maximum moments were compared to analytical predictions of beam and connection strength. Also, each moment versus rotation relationship was analyzed for compliance with the requirements of Ordinary, Intermediate, and Special Moment Frames, as defined by AISC in the *Seismic Provisions for Structural Steel Buildings*.

The experimental results demonstrated that the thick plate procedure in Design Guide 16 is an accurate model for predicting the strength of the connection elements, and the procedure is recommended for designing connections subject to cyclic (seismic) loads. The connection design moment should be based on the expected plastic strength of the beam, regardless of the equations governing nominal beam strength.

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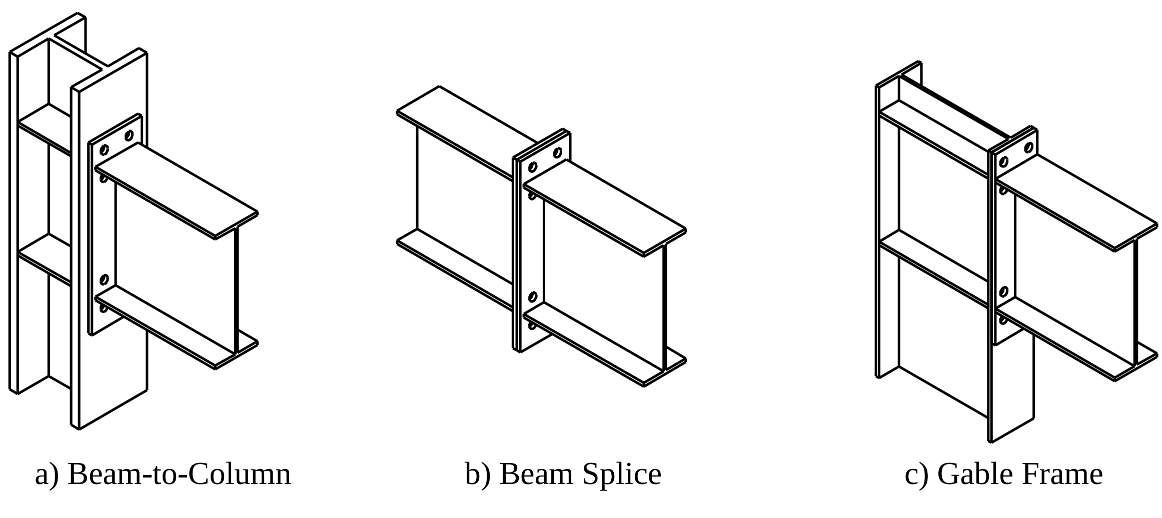
CHAPTER 1 - INTRODUCTION

1.1 BACKGROUND

The end-plate moment connection is used for beam-to-column connections, beam splices, and gable frame connections, as illustrated in Figure 1.1. The connection consists, first, of a plate with punched or drilled holes. The plate is shop-welded to a beam section and then field-bolted to a column, or to another end-plate in the case of a beam splice. An extended end-plate connection is one where the end-plate extends above or below the limits of the beam flange. In the case of seismic design, the end-plate usually is extended above and below the beam flanges and is symmetrical, because of load reversal. The scope of this research is restricted to the study of extended end-plate moment connections.



a) Beam-to-Column   b) Beam Splice   c) Gable Frame

FIGURE 1.1: TYPICAL USES FOR END-PLATE MOMENT CONNECTIONS

The use of end-plates in moment-resisting connections for steel buildings began in the 1960s. The end-plate moment connection was developed from research on tee-stub connections in the 1950s, and offered several advantages over other moment connections. No field welding was required, which reduced erection time. When using built-up members, end-plates could be cut from the same material used for the beam flanges, thereby simplifying fabrication. Also, because beam depth tolerances of rolled shapes did not influence the alignment of bolt holes, all holes could be punched or drilled in the shop in a reduced amount of time. During the 1980s, end-plate moment connections were adapted for use in multi-story buildings. More accurate fabrication techniques and research into end-plate and bolt behavior facilitated this expanded use.

Since the late 1980s, end-plate moment connections have been studied to determine their response to cyclic loading. Also, after the 1994 Northridge earthquake and the 1995 Kobe earthquake, investigators determined that bolted and riveted moment connections performed better than flange-welded moment connections in both of those seismic events. As a result of those findings, the end-plate moment connection is considered a viable, possibly preferable, alternative to connections utilizing field welding.

1.2 REVIEW OF PREVIOUS RESEARCH

Sumner (2003) recently summarized the results of the cyclic tests of extended end-plate moment connections performed over the past two decades. The experimental results of these tests demonstrate the following:

1. Connections designed for less than the beam capacity may not provide sufficient ductility. Refer to Johnstone and Walpole (1981), Fleischman et al. (1990), Ghoborah et al. (1990), Korol et al. (1990), Boorse (1999), and Ryan (1999).
2. Design procedures for monotonic loading are insufficient for seismic loading. Specifically, the bolts and end-plate should be designed for more than the plastic

moment capacity of the beam to prevent bolt degradation and loss of end-plate strength. Refer to Popov and Tsai (1989), Tsai and Popov (1990), Ghobarah et al. (1990), Leon (1995), and Adey et al. (1997, 1998, 2000).

3. Panel zone yielding can contribute to energy dissipation, with the end-plate helping to control the inelastic deformation of the panel zone. Refer to Ghobarah et al. (1992).
4. Weld access holes should not be used on end-plate moment connections. Stress concentrations in the flange, leading to flange tension rupture, are likely to occur if weld access holes are used. Refer to Meng and Murray (1997).
5. Shims can be used between the column flange and end-plate to mitigate bolt prying forces. Refer to Astaneh-Asl (1995) and Meng (1996).
6. The plastic moment strength of connecting beams frequently is greater than that predicted by the nominal plastic moment strength ($M_p = F_y Z_x$). The increased beam strength should be considered in connection design. Refer to Coons (1999) and AISC (2002).

Sumner (2003) incorporated the results of the aforementioned research into a unified design procedure for extended end-plate moment connections subject to cyclic loading. He recommended using yield-line theory to establish end-plate strength, designing the bolts assuming no prying action will occur, and selecting an end-plate thick enough to prevent the development of bolt prying forces. His review of previous research (Borgsmiller, 1995) led to the conclusion that bolt prying forces will not develop until the end-plate is loaded to at least 90% of its yield strength as predicted by yield line theory. Therefore, in order to prevent prying forces, he recommended dividing the moment resistance of the bolts by the factor 0.9, and using the resulting moment value to size the end-plate. This design procedure has been published in AISC Design Guide 16 (AISC, 2002b) for selected end-plate configurations.

In addition to the research summarized by Sumner, others have conducted cyclic tests of end-plate moment connections.

Dubina et al. (2002) conducted six tests of bare steel, double-sided, beam-to-column joints, and six tests of double-sided joints with a concrete slab. A mix of monotonic and cyclic testing was performed. All columns were flanged cruciforms in cross-section, built-up from welded plates, and the beams were I sections. They found that the panel zone could contribute significantly to energy dissipation due to the stiffening effect that the out-of-plane elements of the column would have on the panel zone. Welds connecting the flange to end-plate ruptured on many of the cyclically loaded specimens; weld access holes were used on all connections. Finite element modeling of the joint showed good correlation with experimental results.

Yorgun and Bayramoğlu (2001) conducted two tests on built-up beam and column sections with extended end-plate moment connections. The connections were designed weaker than the column and beam to observe connection behavior. One specimen had a 15 mm thick I-shaped shim between the end-plate and column flange, and the other specimen had no shim. They concluded that adding a shim could increase the ductility of the connection.

1.3 MOMENT CONNECTION CLASSIFICATION

Based on its relationship between moment resistance and rotational stiffness, a moment connection falls into one of two construction categories: partially restrained (PR) or fully restrained (FR). AISC provides guidelines for classifying connections based on the observed moment versus rotation curve (AISC, 1999). Extended end-plate moment configurations are sufficiently stiff to be classified as fully restrained. The fundamental assumption in the analysis of fully restrained construction is that the connections have sufficient rigidity to maintain the angles of intersecting members.

1.4 SEISMIC FORCE RESISTANCE CLASSIFICATION

The Seismic Provisions for Structural Steel Buildings (AISC, 2002) provide additional categories for moment connection classification. The categories are based on the severity of inelastic deformation expected in the seismic load resisting frame into which the connection is incorporated. The least restrictive category is the Ordinary Moment Frame (OMF). Connections in OMFs may be either PR or FR construction. There are no specific moment versus rotation requirements for this category. The Intermediate Moment Frame (IMF) category is more restrictive. Connections in IMFs must sustain an Interstory Drift Angle of at least 0.02 radians. The most restrictive category is the Special Moment Frame (SMF). Connections in SMFs must sustain an Interstory Drift Angle of at least 0.04 radians. For both the IMF and SMF, the flexural strength of the connection determined at the column face must equal at least 80 percent of the nominal plastic strength of the connected beam at the required Interstory Drift Angle.

1.5 PREQUALIFICATION OF BEAM-TO-COLUMN CONNECTIONS

1.5.1 Procedure

Appendices P and S of the Seismic Provisions for Structural Steel Buildings prescribe the procedure for testing (Appendix S) and prequalifying (Appendix P) a beam-to-column moment connection for use in Special Moment Frames and Intermediate Moment Frames.

Appendix P gives the authority for prequalifying a connection to a Connection Prequalification Review Panel (CPRP) recognized by the local building official. Based on available test data, the CPRP establishes upper and lower limits on the relevant connection parameters (i.e., beam depth, bolt diameter, etc.). Once the CPRP has prequalified a connection, the connection can be used on any project without project-specific testing, as long as the connection designer stays within the limits of the connection parameters established by the CPRP.

Appendix S prescribes the method of testing a beam-to-column connection. Connections that are considered for prequalificiation must satisfy the requirements of Appendix S. That is, the CPRP will consider for prequalification only those connections that have been tested in accordance with Appendix S. Additionally, Appendix S is used for project-specific testing and qualification of connections. The specimen tested in the laboratory (Test Specimen) must replicate the actual connection that will be used in the building (Prototype) in two major areas:

1. Test Specimen and Prototype points of inflection must coincide approximately, and locations of lateral bracing must coincide.
2. The Prototype design, detailing, construction features, and material properties must be replicated in the Test Specimen as closely as is practical. This includes sources of inelastic rotation, member sizes, material strengths, weld type and size, bolt type and size, etc.

Appendix S also prescribes the Loading History, Instrumentation, Materials Testing, and Test Reporting for the Test Specimen. Test Specimens are qualified based on the criteria described above in Section 1.4, and the Test Specimen must sustain the required Interstory Drift Angle for at least one complete load cycle.

1.5.2 Testing Limitations and Test Specimen Modifications

A significant problem arises when an attempt is made to create Test Specimens for a gable frame. Section S4.2 of the Seismic Provisions states that "points of inflection in the test subassemblage shall coincide approximately with the anticipated points of inflection in the Prototype under earthquake loading." For a gable frame, the points of inflection are at the column base and near the ridge. Such a Test Specimen would be huge, perhaps as long as 100 ft. Also, the load chain necessarily would be along the line of inflection points. The resulting Test Specimen, shown in Figure 1.2, would be quite impractical.

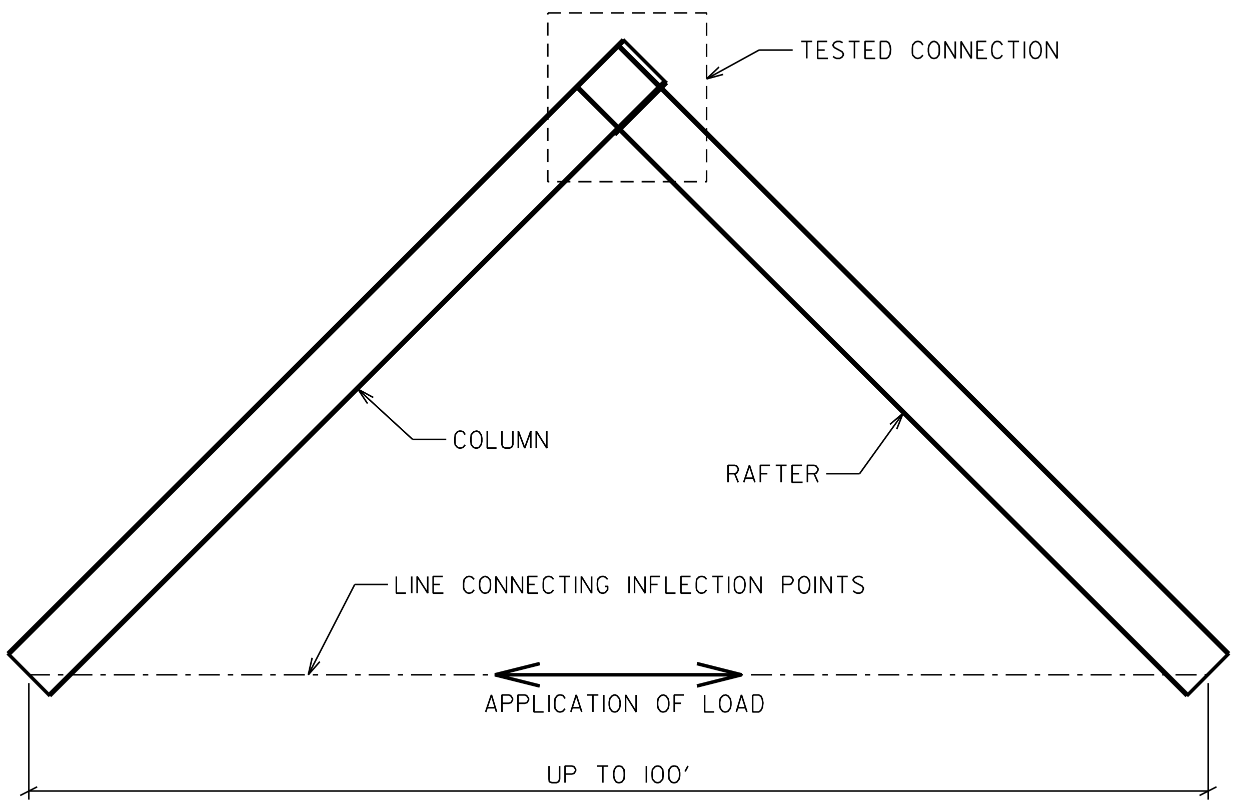


FIGURE 1.2: GABLE FRAME TEST SPECIMEN

The AISC seismic testing provisions are written for beam-to-column connections in multi-story frames. The provisions address neither the connections at the tops of columns in such frames, nor connections in portal or gable frames. Therefore, for the purposes of testing and qualification, the gable frame connection can be considered a derivative of the basic beam-to-column connection. In the basic beam-to-column connection, the column is supported at its ends, the beam is placed at mid-height of the column, and the load is applied at the beam tip. This simplified Test Specimen, shown in Figure 1.3, served as the model for all specimens tested during this study.

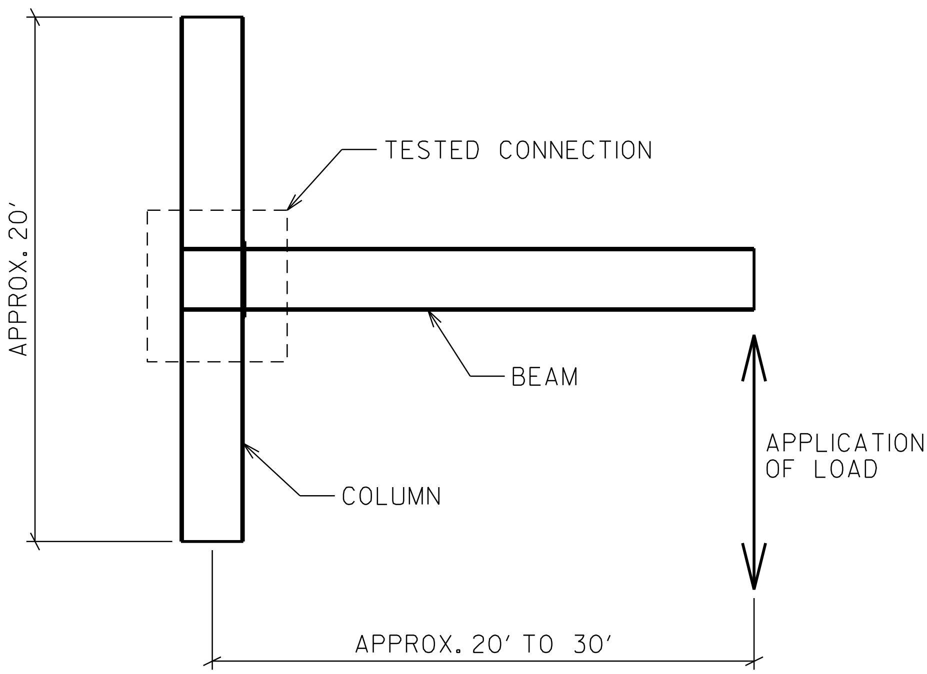
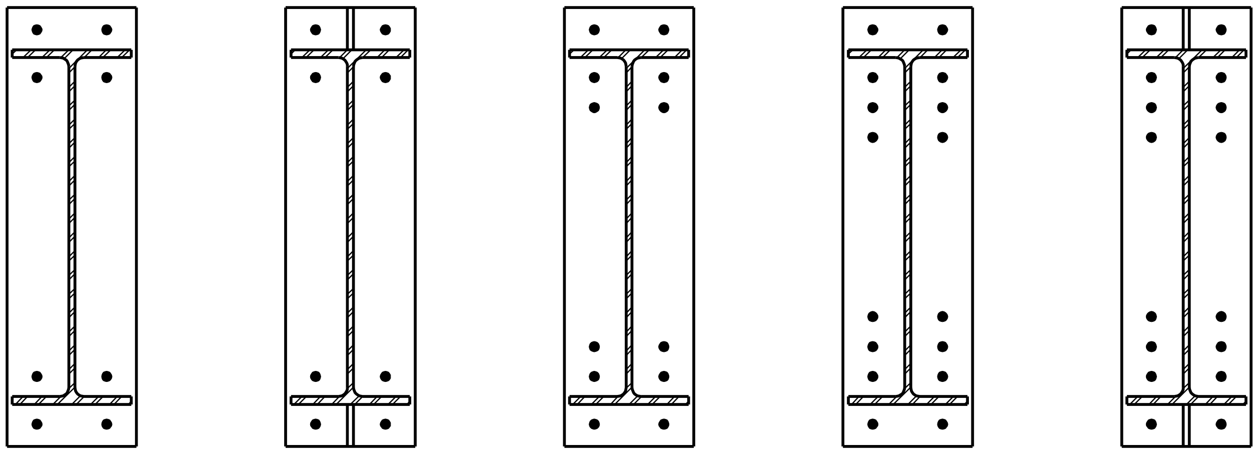


FIGURE 1.3: TYPICAL TEST SPECIMEN

1.6 RESEARCH OBJECTIVES

The purpose of this study is to subject five types of extended moment end-plate configurations, shown in Figure 1.4, to cyclic loading through failure, and observe their response. Observed connection strength is compared to the predicted strengths using the "thick plate" design procedure provided in AISC Design Guide 16. Observed beam strength is compared to the predicted strengths for sections with compact, non-compact, and slender webs.



a) 4E b) 4ES c) MRE 1/2 d) MRE 1/3 e) MRE 1/3S

FIGURE 1.4: TESTED EXTENDED END-PLATE CONFIGURATIONS

The experimentally determined moment versus rotation relationships provide the basis for classifying connections according to the categories provided in the Seismic Provisions for Structural Steel Buildings. Finally, the tests conducted as part of this study may form the basis for prequalifying these extended end-plate moment connections for use in seismic force resisting frames.

The design of the Test Specimens is discussed in Chapter 2. The testing program and experimental results are discussed in Chapters 3 and 4. The experimental results are analyzed in Chapter 5, and Chapter 6 includes conclusions, design recommendations, and future research needs.

CHAPTER 2 - TEST SPECIMEN DESIGN

2.1 DESIGN METHODOLOGY

The current methodology for design of steel frames in seismic regions is provided in the recommendations of FEMA 350 (FEMA, 2000). Frames should be proportioned so that the required Interstory Drift Angle, defined as the interstory displacement divided by story height, "can be accommodated through a combination of elastic deformation and the development of plastic hinges at pre-determined locations within the frame". It is the designer's choice as to where the plastic hinges develop. However, in multi-story frames, the greatest amount of energy dissipation occurs when plastic hinges develop in the beams. This is due to the fact that collapse mechanisms comprised of beam hinges require more elements than mechanisms composed of column hinges. Consequently, the most desirable approach is a strong column, strong connection, and weak beam design philosophy. All specimens tested during this study were designed following this philosophy.

2.2 DESIGN MOMENT

Plastic hinge location depends on the type of moment connection used. A beam of constant cross section, connected to the column using an extended end-plate, will develop hinges at a distance from the column face approximately equal to one-half the beam depth. From AISC Seismic Provisions, the expected yield stress R_yF_y must be used in calculating the expected beam plastic strength, M_{pe}:

$$M_{pe} = 1.1 R_y F_y Z_x \tag{2.1}$$

In the above expression, 1.1 is a factor to account for peak connection strength (including the effects of strain hardening, etc.), R_y is a coefficient obtained from the AISC Seismic Provisions and depends on material specification, Z_x is the plastic modulus at the hinge location, and F_y is the specified beam yield stress.

While it is reasonable to use Equation 2.1 for members with compact elements, it is not clear that Equation 2.1 applies to members with non-compact or slender elements. Two competing phenomena contribute to this uncertainty. First, according to the equations developed for nominal beam strength (AISC, 1999), unless all elements are compact, the individual elements will buckle locally before the section's plastic bending strength is achieved. Secondly, however, the stiffening effect of an end-plate may be sufficient to overcome the propensity of the elements to buckle locally. Consequently, the strength of a beam welded to an end-plate may be equal to the nominal beam strength (established by the LRFD Specification), the strength as defined by Equation 2.1, or somewhere between these two limits.

Members with non-compact or slender elements, particularly webs, are quite common in the pre-engineered metal building industry. Therefore, to more accurately establish the strength of beams when they are used in conjunction with extended end-plate moment connections, the connections in this testing program were designed assuming that the beam strength at the hinge equaled the nominal beam strength.

2.3 DESIGN PROCEDURE

2.3.1 Beam Design

Selecting the beam size and material was the first step in designing the Test Specimens. Tests conducted as part of this study are intended to form the basis for prequalifying the connections for use in either SMFs or IMFs. Because the CPRP will establish limits on beam depths for which the connections are prequalified, a wide range of beam depths was used in this study.

Table 2.1 shows the critical parameters for the beams used in this study. All beams were welded, built-up sections. Included in the table is a reference to the governing strength equations for each beam, which come from the LRFD Specification (AISC, 1999).

TABLE 2.1: BEAM SPECIMENS

Beam Depth (in.)	Flange				Web				Comments	Governing Equations
	Width b_f (in.)	Thickness t_f (in.)	$\frac{b_f}{2t_f}$	Material	Height h (in.)	Thickness t_w (in.)	$\frac{h}{t_w}$	Material		
24	7	5/8	5.6	A572 Gr 50	22 3/4	3/8	60.7	A572 Gr 50	Compact	Chapter F
30	6	3/8	8	A572 Gr 55	29 1/4	1/4	117	A572 Gr 55	Non-compact Web	Appendix F
60	8	5/8	6.4	A572 Gr 50	58 3/4	3/8	156.7	A572 Gr 50	Slender Web	Appendix G
72	12	1	6	A572 Gr 55	70	1/2	140	A572 Gr 50	Slender Web	Appendix G

Once the beam sizes and materials were selected, their nominal strengths were determined using the appropriate governing equations. Also calculated were their plastic and expected plastic strengths. However, in an attempt to predict the actual material yield stresses of the flanges and webs, nominal (M_n) and plastic (M_p) strengths were calculated by adding 5 ksi to the specified minimum yield stress. For example, A572 Grade 50 material was assumed to have a yield of 55 ksi. Expected plastic (M_{pe}) strengths were calculated using the yield and tensile stresses shown in Table 2-1 of the LRFD Manual (AISC 2001). Table 2.2 summarizes the resulting strength calculations, which are based on *predicted* properties.

TABLE 2.2: CALCULATED BEAM STRENGTHS

Beam Depth (in.)	Flange		Web		Governing Equations	Nominal Strength M_n (k*ft)	Plastic Strength M_p (k*ft)	Expected Plastic Strength M_{pe} (k*ft)
	Width b_f (in.)	Thickness t_f (in.)	Height h (in.)	Thickness t_w (in.)				
24	7	5/8	22 3/4	3/8	Chapter F	691	691	763
30	6	3/8	29 1/4	1/4	Appendix F	522	601	680
60	8	5/8	58 3/4	3/8	Appendix G	2210	2844	3161
72	12	1	70	1/2	Appendix G	6038	7067	7865

2.3.2 Connection Design

It was expected that the hinge would occur at a distance approximately equal to one-half the beam depth away from the connection faying surface. If that occurred, the connection moment would be larger than the moment at the hinge by a factor equal to the distance from the load point to the faying surface divided by the distance from the load point to the hinge location. Beam lengths were selected such that this factor, based on predicted hinge location, was approximately 1.1. Therefore, 110% of the nominal beam strength was used for the design connection strength.

Using the design connection strength, bolt size and grade were determined assuming a "thick" end-plate and no prying forces. Nominal bolt tensile stresses (90 ksi for A325 and 113 ksi for A490) were used. Then, an end-plate was selected that would be thick enough to prevent prying forces. In an attempt to predict the actual end-plate yield stress, 5 ksi was added to the specified minimum yield stress. The procedure for calculating bolt and end-plate strength is found in AISC Design Guide 16. The resulting connection configurations, determined using the aforementioned procedure, are summarized in Table 2.3.

The connection nomenclature is a combination of connection type, bolt diameter, end-plate thickness, and nominal beam depth. The five connection types are as follows: 4E for the four bolt extended unstiffened connection, 4ES for the four bolt extended stiffened connection, MRE1/2 for the six bolt extended unstiffened connection, MRE1/3 for the eight bolt extended unstiffened connection, and MRE1/3S for the eight bolt extended stiffened connection (see Figure 1.4). For example, a test designation of 4ES-1.0-1.0-24 indicates a four bolt extended stiffened connection with 1 in. diameter bolts, a 1 in. thick end-plate, and a nominal beam depth of 24 in.

TABLE 2.3: CALCULATED CONNECTION STRENGTHS

Specimen ID	Beam Depth (in.)	$1.1*M_n$ (k*ft)	No. of Bolts (Grade)	Bolt Dia. (in.)	Bolt Strength M_{np} (k*ft)	End-Plate Thickness (in.)	Expected End-Plate Yield (ksi)	Expected End-Plate Strength M_{pl} (k*ft)
MRE1/2 - 0.875 - 1.0 - 24	24	760	6 (A490)	7/8	734	1	55	871
4ES - 1.0 - 1.0 - 24	24	760	4 (A490)	1	692	1	55	1068
MRE1/3 - 0.75 - 0.75 - 30	30	574	8 (A325)	3/4	695	3/4	60	821
MRE1/3S - 0.75 - 0.625 - 30	30	574	8 (A325)	3/4	695	5/8	60	795
4E - 1.25 - 1.25 - 60 A	60	2431	4 (A325)	1 1/4	2177	1 1/4	55	3273
4E - 1.25 - 1.25 - 60 B	60	2431	4 (A490)	1 1/4	2733	1 1/4	55	3273
MRE1/2 - 1.0 - 1.0 - 60 A	60	2431	6 (A325)	1	2036	1	55	2452
MRE1/2 - 1.0 - 1.0 - 60 B	60	2431	6 (A490)	1	2557	1	55	2452
MRE1/3S - 1.25 - 1.25 - 72 A	72	6642	8 (A490)	1 1/4	6255	1 1/4	60	8359
MRE1/3S - 1.25 - 1.25 - 72 B	72	6642	8 (A490)	1 1/4	6255	1 1/4	60	8359
MRE1/3 - 1.25 - 1.5 - 72	72	6642	8 (A490)	1 1/4	6255	1 1/2	60	9844

In some instances, due to selection of bolt size and grade, the moment strength provided by the bolt group (M_{np}) was less than 110% of the beam nominal moment strength. This was due primarily to the fact that sizes and grades are not available to arrive very close to the desired value. For example, on the 4E - 1.25 - 1.25 - 60 connection, the needed bolt grade is 101 ksi, which is the average of the two available grades. This was less of a problem when selecting an end-plate thickness and grade. To ensure that prying forces would not develop, the decision was made to remain conservative with respect to required end-plate thickness, and a thickness was selected that was the closest available without going below the calculated required thickness. In all but one Test Specimen, predicted bolt strength was less than predicted end-plate strength.

2.3.3 Column Design

The columns were the last element of the Test Specimen to be designed, and were designed to remain elastic throughout testing. Continuity plates were used on all columns, following the recommendations of FEMA 350. Each continuity plate thickness matched the thickness of the flanges on the connecting beam. For a given bolt pattern, the yield line mechanism for an end-plate results in at least the same thickness as does the yield line mechanism for a stiffened column flange. Therefore, each column flange had the same thickness and grade as the connecting end-plate. By inspection, this resulted in a satisfactory column flange design. Column depth was selected for adequate moment resistance. For the 24 in. and 30 in. beams, a 20 in. column depth was used. A 30 in. deep column was used for the 60 in. beams, and a 40 in. column was used for the 72 in. beams.

The web thickness in the panel zone was selected on the basis of shear resistance. The applied shear was calculated by dividing the moment at the faying surface by the distance between the centers of the beam flanges. In some instances, a thinner web was provided outside of the panel zone, to save on material costs.

The weld connecting the continuity plate to the column web was designed to transmit the entire flange force into the panel zone. Within the panel zone, the weld connecting the column web to the column flange was designed to transmit the shear resulting from rigid body movement of the panel zone web. The free body diagram shown in Figure 2.1 illustrates the design forces, where F_{fu} is the predicted maximum applied flange force, and F_{pz} is the predicted force required to resist rigid body rotation caused by F_{fu}.

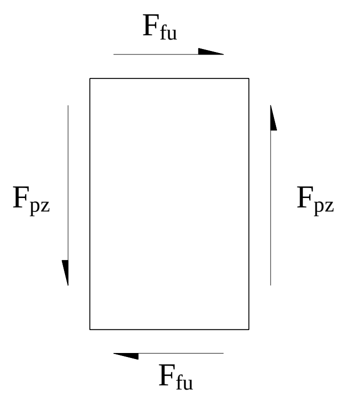


FIGURE 2.1: PANEL ZONE FREE BODY DIAGRAM

CHAPTER 3 - EXPERIMENTAL PROGRAM

3.1 General

The testing program consisted of six beam and column combinations. The specimens consisted of exterior sub-assemblages comprised of built-up beam and column sections. Typically each beam and column was used for two tests because a different connection type was installed on each beam end. However, the MRE1/3S-1.25-1.25-72 A connection was not paired with another connection. Therefore, a total of eleven extended end-plate moment connections were tested. The test matrix is shown in Table 3.1.

TABLE 3.1: TEST MATRIX

Specimen ID	Beam Depth (in.)	Column Depth (in.)	No. of Bolts (Grade)	Bolt Dia. (in.)	End-Plate Thickness (in.)
MRE1/2 - 0.875 - 1.0 - 24	24	20	6 (A490)	7/8	1
4ES - 1.0 - 1.0 - 24	24	20	4 (A490)	1	1
MRE1/3 - 0.75 - 0.75 - 30	30	20	8 (A325)	3/4	3/4
MRE1/3S - 0.75 - 0.625 - 30	30	20	8 (A325)	3/4	5/8
4E - 1.25 - 1.25 - 60 A	60	30	4 (A325)	1 1/4	1 1/4
4E - 1.25 - 1.25 - 60 B	60	30	4 (A490)	1 1/4	1 1/4
MRE1/2 - 1.0 - 1.0 - 60 A	60	30	6 (A325)	1	1
MRE1/2 - 1.0 - 1.0 - 60 B	60	30	6 (A490)	1	1
MRE1/3S - 1.25 - 1.25 - 72 A	72	40	8 (A490)	1 1/4	1 1/4
MRE1/3S - 1.25 - 1.25 - 72 B	72	40	8 (A490)	1 1/4	1 1/4
MRE1/3 - 1.25 - 1.5 - 72	72	40	8 (A490)	1 1/4	1 1/2

3.2 TEST SETUP

For the first nine tests, the setup shown in Figure 3.1 was used. The ninth test, MRE1/3S-1.25-1.25-72 A, caused damage to the support framing. While the damage did not introduce errors into the testing or data collection process, a decision was made to build a stronger frame for the remaining tests. For these last two tests, MRE1/3S-1.25-1.25-72 B and MRE1/3-1.25-1.5-72, the setup shown in Figure 3.2 was used. The sub-assemblages were tested in a horizontal position for ease of construction and safety during testing. The test columns were bolted to support beams. The support beams were bolted to posts (in the case of Figure 3.1) or floor beams (in the case of Figure 3.2), which in turn were bolted to the reaction floor.

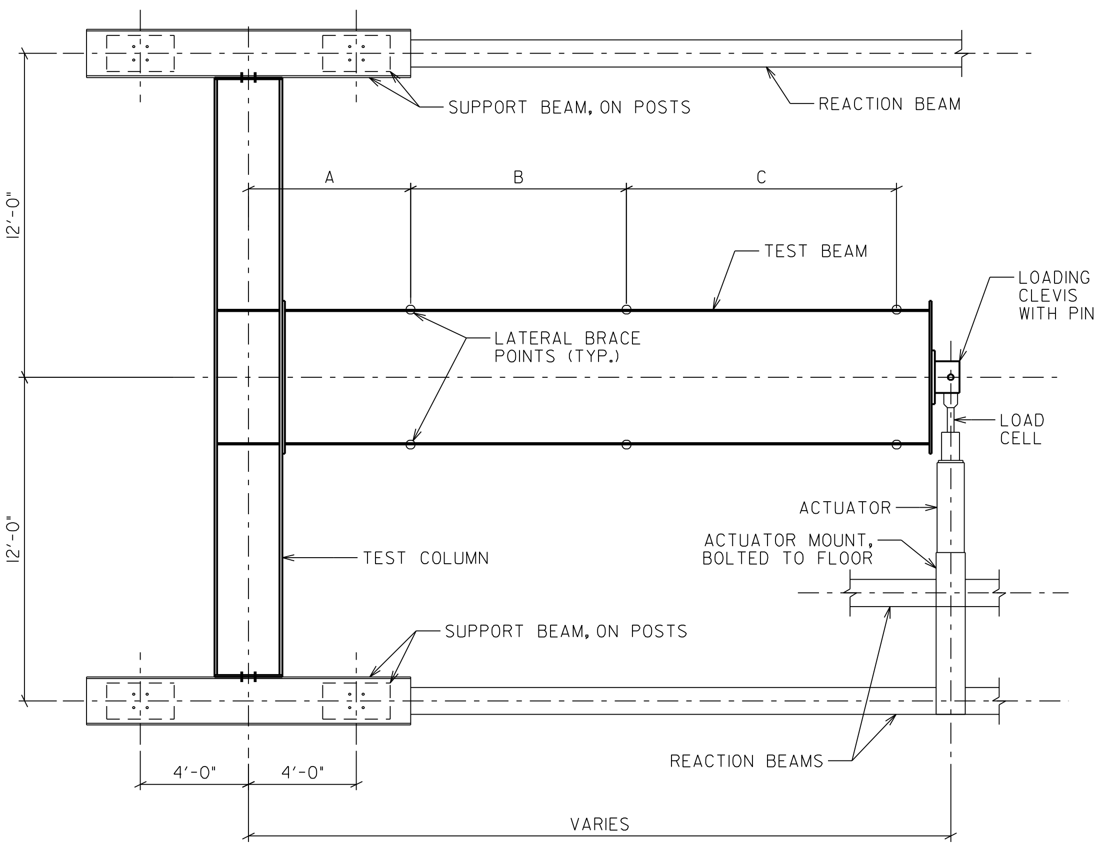


FIGURE 3.1: TYPICAL SETUP FOR FIRST NINE TESTS

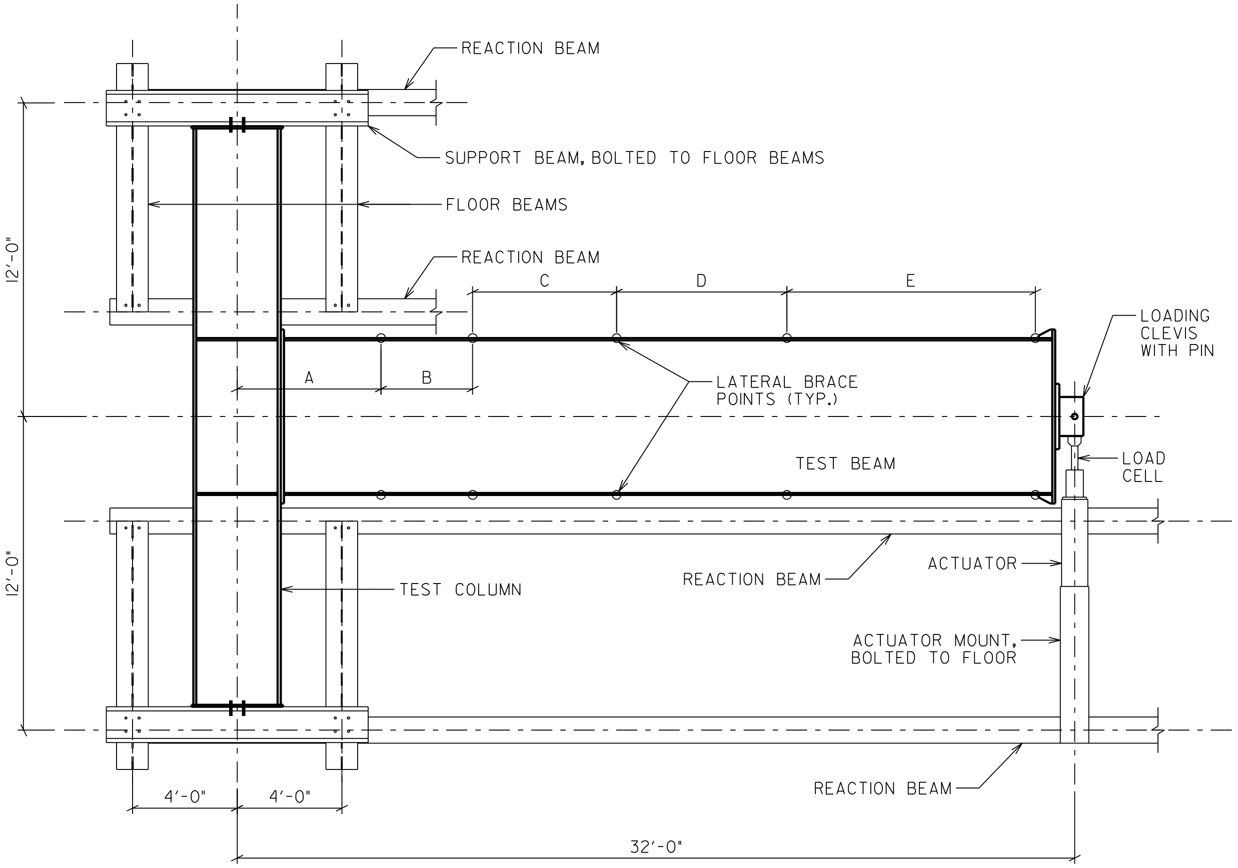


FIGURE 3.2: SETUP FOR MRE1/3S-1.25-1.25-72 B AND MRE1/3-1.25-1.5-72

The column ends were considered to be simply supported. Lateral supports were provided above and below the beams to prevent lateral torsional buckling of the beam. Dimensions for beam bracing locations, as well as top bracing size, are shown in Table 3.2. Channels, oriented horizontally, were used for the first two tests. However, during test 4ES-1.0-1.0-24, the channels were not sufficiently stiff enough to prevent the beam from buckling after the beam's plastic strength had been achieved. In subsequent tests, therefore, W21 sections were used to brace the beams on top. Below the beams, heavy W8 sections were used for bracing on all tests. Lateral supports also were provided below the columns, near the connection, to prevent out-of-plane column movement due to beam deformations. At the outset of testing, the clear distance between the members and braces was approximately 1/4 in. A roller was used to

TABLE 3.2: BEAM BRACING DATA

Specimen ID	Top Bracing Size	BRACING DIMENSIONS [1]				
		A (ft)	B (ft)	C (ft)	D (ft)	E (ft)
MRE1/2 - 0.875 - 1.0 - 24	C10x15.3	4.5	5	7	-	-
4ES - 1.0 - 1.0 - 24	C10x15.3	4.5	5	7	-	-
MRE1/3 - 0.75 - 0.75 - 30	W21x62	3.83	5	7	-	-
MRE1/3S - 0.75 - 0.625 - 30	W21x62	3.83	5	7	-	-
4E - 1.25 - 1.25 - 60 A	W21x62	6	8	10	-	-
4E - 1.25 - 1.25 - 60 B	W21x62	6	8	10	-	-
MRE1/2 - 1.0 - 1.0 - 60 A	W21x62	6	8	10	-	-
MRE1/2 - 1.0 - 1.0 - 60 B	W21x62	5.5	7.5	11	-	-
MRE1/3S - 1.25 - 1.25 - 72 A	W21x62	9	9	12	-	-
MRE1/3S - 1.25 - 1.25 - 72 B	W21x62	5.5	3.5	5.5	6.5	9.5
MRE1/3 - 1.25 - 1.5 - 72	W21x62	5.5	3.5	5.5	6.5	9.5

1. Refer to Figures 3.1 and 3.2 for dimensions 'A' through 'E'.

support the loaded end of the beam, so that the beam was not bearing on the braces at the onset of testing. A clevis was bolted to the end-plate not being tested (i.e., the end-plate that was not bolted to the column). For the 60 in. and 72 in. deep beams, additional holes were required in the end-plate for attachment of the clevis. These additional holes were located within the middle third of the beam depth, thereby eliminating their influence on end-plate strength. For the 24 in. and 30 in. deep beams, the clevis was bolted to the end-plate using the connection bolt holes.

3.2.1 Instrumentation

The instrumentation layout is shown in Figure 3.3. Support framing, reaction floor, and loading equipment are not shown for clarity. Loading was applied to the beam tip using an Enerpac RR30036 double-acting hydraulic ram. Screwed into the end of the ram was a 200 kip capacity tension-compression load cell. The linear displacement transducer, Δ_L, measured beam tip displacement at the end-plate on the loaded end of the beam. The transducer was attached to the end-plate and not the clevis. Displacement potentiometers, Δ_{CT} and Δ_{BT}, were positioned at the ends of the column to measure rigid body rotation. Another displacement potentiometer, Δ_T, was placed on one of the support beams along the column centerline to measure rigid body translation. Displacement potentiometers, Δ_{CTF} and Δ_{CBF}, were placed on the column across from the beam flanges to measure panel zone deformation. Displacement potentiometers, Δ_{PZ1} and Δ_{PZ2}, measured change in length diagonally across the panel zone. Calipers, CLP1 and CLP2, were placed around the column flange and end-plate at the beam flanges to measure separation of the end-plate from the column flange.

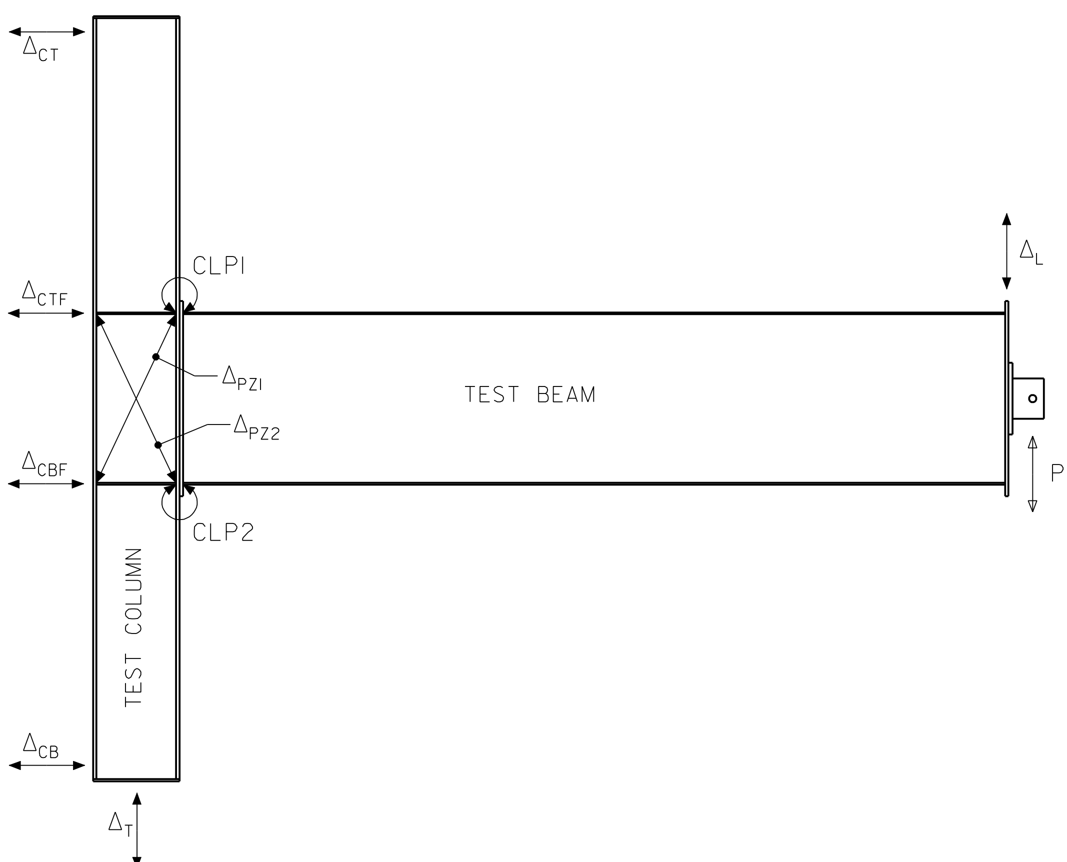
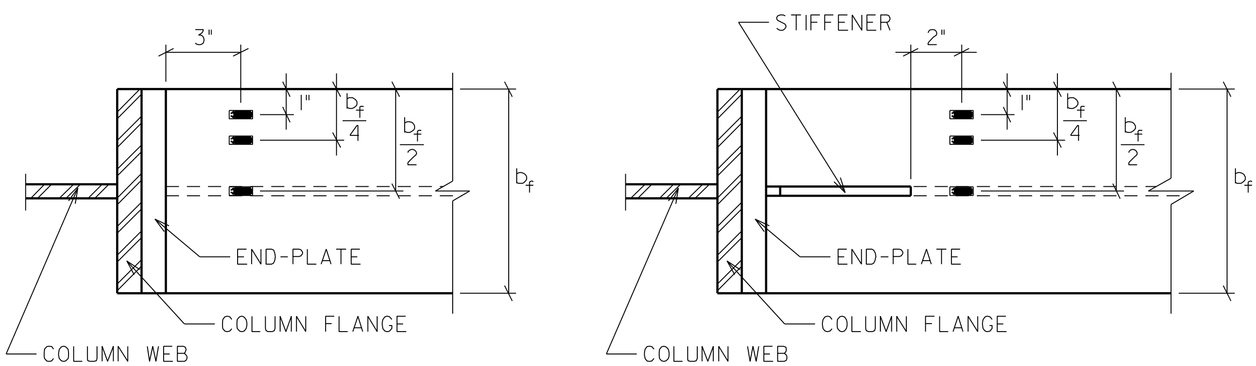


FIGURE 3.3: TYPICAL INSTRUMENTATION

Uniaxial strain gages were placed on the beam flanges to measure strains near the end-plate. With unstiffened end-plates, strain gages were placed 3 in. from the edge of the end-plate. With stiffened end-plates, strain gages were placed 2 in. from the end of the stiffener. The two beam flange strain gage configurations are shown in Figure 3.4. Uniaxial strain gages also were placed inside half of the bolts to measure bolt forces during pre-tensioning and testing. The gages were placed in a hole drilled through the head into the unthreaded portion of the bolt shank. Prior to connection assembly, the instrumented bolts were calibrated in a universal testing machine.



a) Unstiffened Beam Flange

b) Stiffened Beam Flange

FIGURE 3.4: STRAIN GAGE CONFIGURATIONS ON BEAM FLANGES

3.3 TESTING PROCEDURE

The column was installed between the support beams. The instrumented bolts were connected to the PC-based data acquisition system and the system was zeroed. A beeswax-based lubricant was applied liberally to the threaded portion of the bolts. The beam was bolted to the column, and the bolts were tightened using an air impact wrench. The instrumented bolts were tightened first, beginning at one beam flange and ending at the other. The bolts were tightened until the minimum pretension load specified by the AISC LRFD Specification was reached

(Section J3.6, AISC, 1999). During tightening, bolt tension was monitored using the data acquisition system, and the time to tighten each bolt was noted. In some instances where 1 1/4 in. diameter bolts were used, no amount of impacting was sufficient to attain the specified pretension. In these cases, the air impact wrench was stopped once the increase in bolt pretension dropped below the rate of 500 lb per 15 seconds. An Ingersoll-Rand IR290 1” Super-Duty Air Impact Wrench (1,600 ft-lb maximum torque) was used to tighten the 1 1/4 in. bolts.

After the instrumented bolts were tightened, the non-instrumented bolts were tightened, working across the beam in the direction opposite to that used for the instrumented bolts. The non-instrumented bolts were tightened for a period of time equal to the average time required to tighten the instrumented bolts.

The remaining instrumentation then was installed and connected to the data acquisition system. Then all instrumentation was zeroed except for the bolts. Calibration values were checked for all potentiometers and transducers, and the calibration values were stored in the system. A preload of approximately one-quarter of the expected beam strength was applied, and the behavior of the instruments was observed. The system stiffness was compared to the theoretical stiffness, and adjustments were made to the instrumentation if necessary. Finally, so that yielding could be observed in the connection region, “whitewash” was applied to the column panel zone, end-plate, and beam between the end-plate and first lateral brace.

The test began with the recording of an initial zero-load reading. The specimen was loaded according to the protocol prescribed by the AISC Seismic Provisions (AISC, 2002). The loading protocol prescribes a series of load steps and the number of cycles for each step, as shown in Figure 3.5. Each load step corresponds to an interstory drift angle. The protocol was executed accordingly, with data points collected at regular intervals. Observations were recorded

Load Step Number	Interstory Drift Angle, θ (rad)	Number of Loading Cycles
1	0.00375	6
2	0.005	6
3	0.0075	6
4	0.01	4
5	0.015	2
6	0.02	2
7	0.03	2
Continue with increments in θ of 0.01, and perform two cycles at each step		

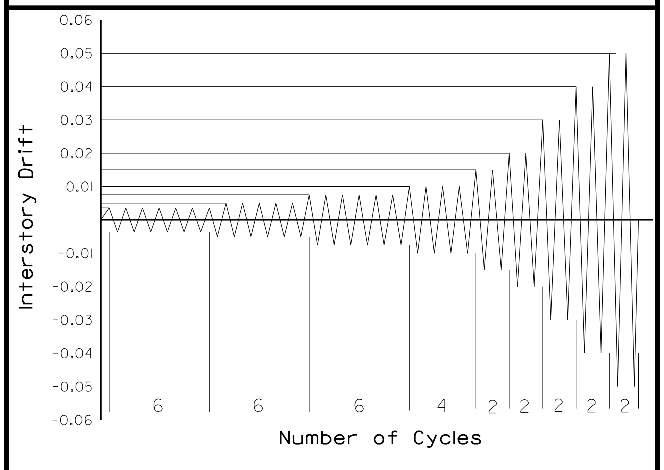


FIGURE 3.5: AISC LOADING PROTOCOL (AISC, 2002)

and photographs taken at noteworthy benchmarks (e.g., first yield, local buckling, etc.). The load steps were continued until there was a brittle failure, the limits of the test frame were reached, or the specimen strength decreased to less than 40 percent of the maximum strength.

The interstory drift angle was monitored in real time to ensure that the specified protocol was followed accurately. Rigid body translation was subtracted from the beam tip deflection, and the resulting value was divided by the distance from the column centerline to the beam tip. Rigid body rotation was calculated by dividing the difference between the two column end potentiometers by the distance between them. Finally, the interstory drift angle was calculated by subtracting rigid body rotation from the modified beam tip rotation.

CHAPTER 4 - EXPERIMENTAL RESULTS

4.1 GENERAL

The parameters used to evaluate the specimen performance were the maximum applied moment and the maximum interstory drift angle with corresponding moment. The applied moment was calculated by multiplying the applied load by the distance from the load application point to the face of the column. For a typical moment frame, the interstory drift angle is calculated by dividing the interstory drift by the story height. For the tested exterior sub-assemblages, the interstory drift angle was calculated by dividing the beam tip displacement by the distance from the beam tip to the column centerline, and subtracting rigid body rotation and translation. The inelastic story drift angle was calculated by subtracting the elastic response from the total response. The elastic response was calculated by multiplying the applied load by a constant stiffness coefficient. The stiffness coefficient was determined prior to testing by performing a frame analysis using the measured specimen properties.

4.2 TEST RESULTS

Fabrication drawings of all test specimens are found in Appendix A. Detailed summaries for all tests are found in Appendices B through L. Included in each summary are design drawings, measured properties of the test specimens, predicted connection and beam strengths, description of specimen performance, graphs showing the specimen response, and photographs before and after testing.

4.2.1 MRE1/2-0.875-1.0-24

The MRE1/2-0.875-1.0-24 specimen began to exhibit inelastic behavior in the 0.02 rad Load Step. The total inelastic behavior, shown in Figure 4.1, was a combination of beam yielding, end-plate separation from the column flange, and panel zone yielding. Panel zone yielding contributed the least to the total inelastic behavior. The end-plate separation from the column flange occurred due to bolt stretching, not end-plate yielding.

Loss of bolt pretension was negligible throughout the elastic load steps, yet increased substantially at and above the 0.02 rad Load Step. During the second half of the last cycle of the 0.03 rad Load Step, all six bolts ruptured on the tension side of the connection at an interstory drift of approximately 0.028 rad. Figure 4.2 shows the Test Specimen after bolt rupture. Figure 4.3 shows a typical plot of bolt behavior on the ruptured side of the connection.

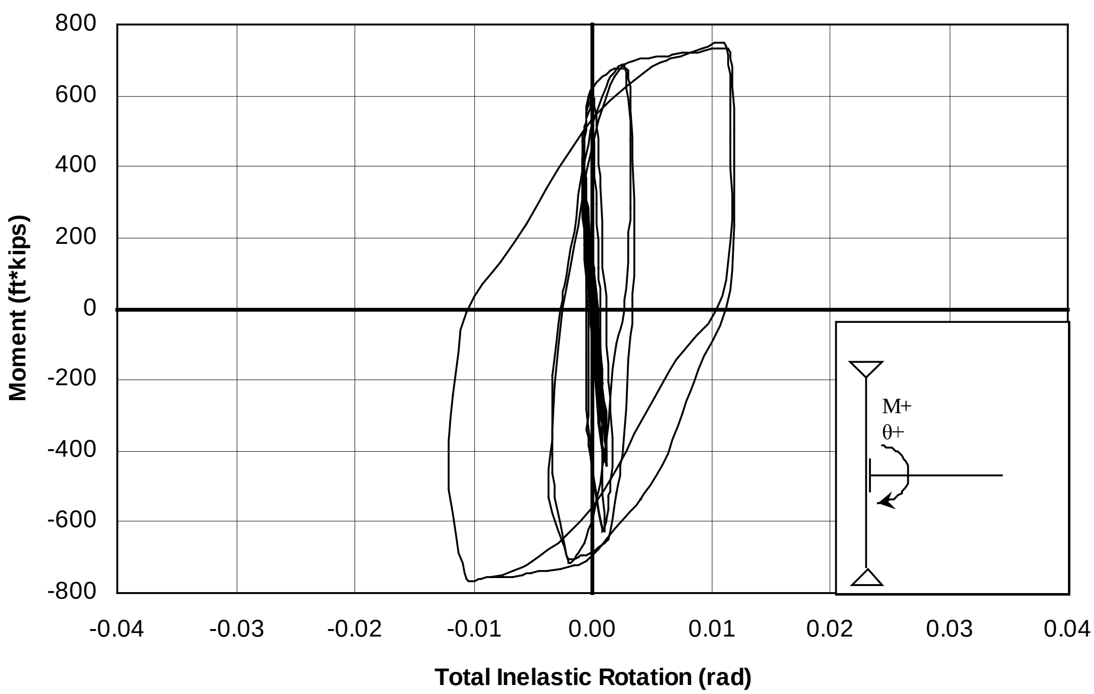


FIGURE 4.1: MRE1/2-0.875-1.0-24 TOTAL INELASTIC RESPONSE

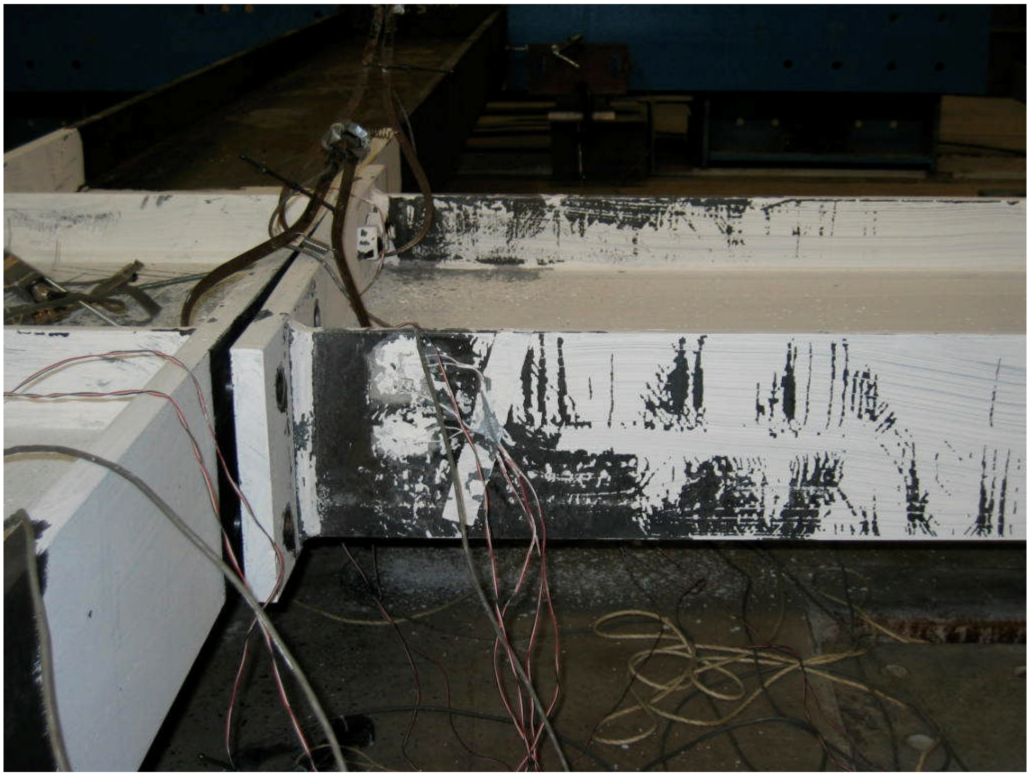


FIGURE 4.2: MRE1/2-0.875-1.0-24 AFTER TESTING

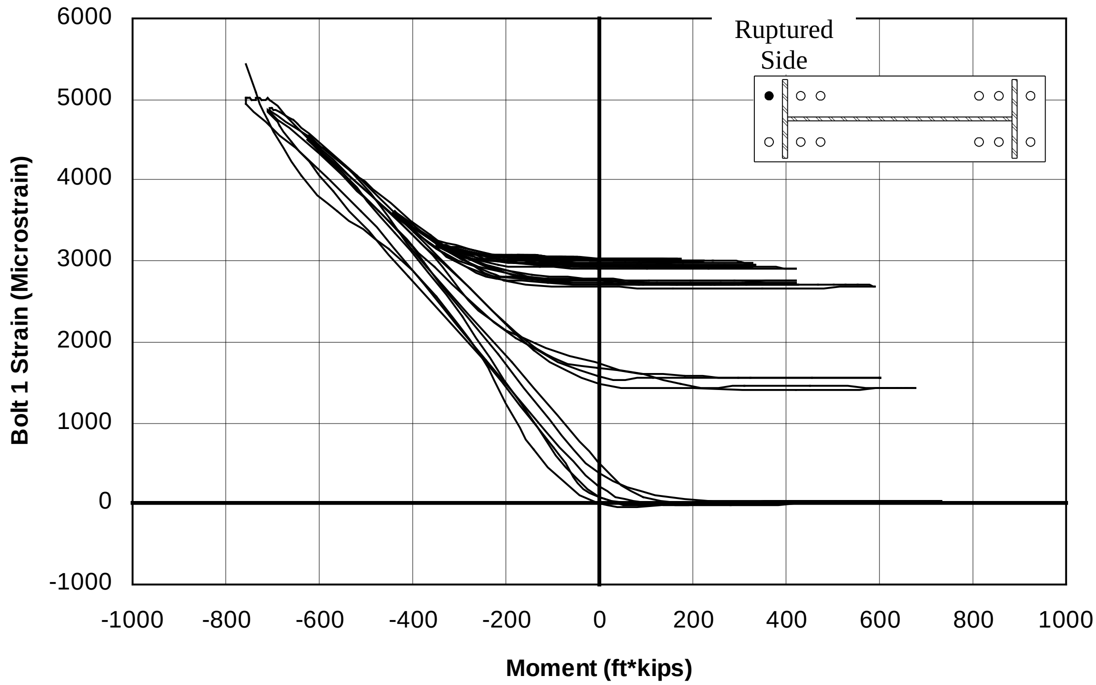


FIGURE 4.3: MRE1/2-0.875-1.0-24 BOLT RESPONSE

4.2.2 4ES-1.0-1.0-24

The 4ES-1.0-1.0-24 specimen began to exhibit inelastic behavior in the 0.02 rad Load Step. The total inelastic behavior, shown in Figure 4.4, was a combination of beam yielding, end-plate separation from the column flange, and panel zone yielding. Nearly all of the inelastic behavior came as a result of beam yielding.

Loss of bolt pretension occurred at and above the 0.02 rad Load Step, eventually going to zero by the end of the 0.03 rad Load Step. Bolt rupture did not occur, however, and it appears that the end-plate did not yield. Flange local buckling was observed during the 0.04 rad Load Step. The test was stopped during the first cycle of the 0.05 rad Load Step, because the bracing could no longer keep the specimen from buckling laterally. Figure 4.5 shows the specimen after testing, and Figure 4.6 is a plot of typical bolt behavior.

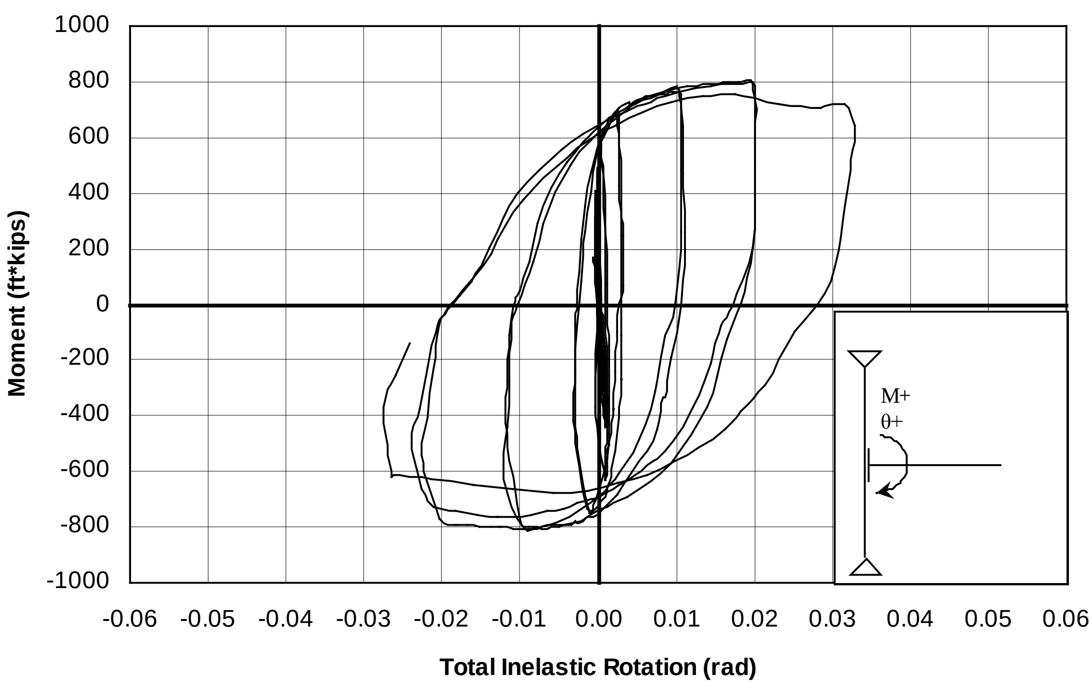


FIGURE 4.4: 4ES-1.0-1.0-24 TOTAL INELASTIC RESPONSE

FIGURE 4.5: 4ES-1.0-1.0-24 AFTER TESTING

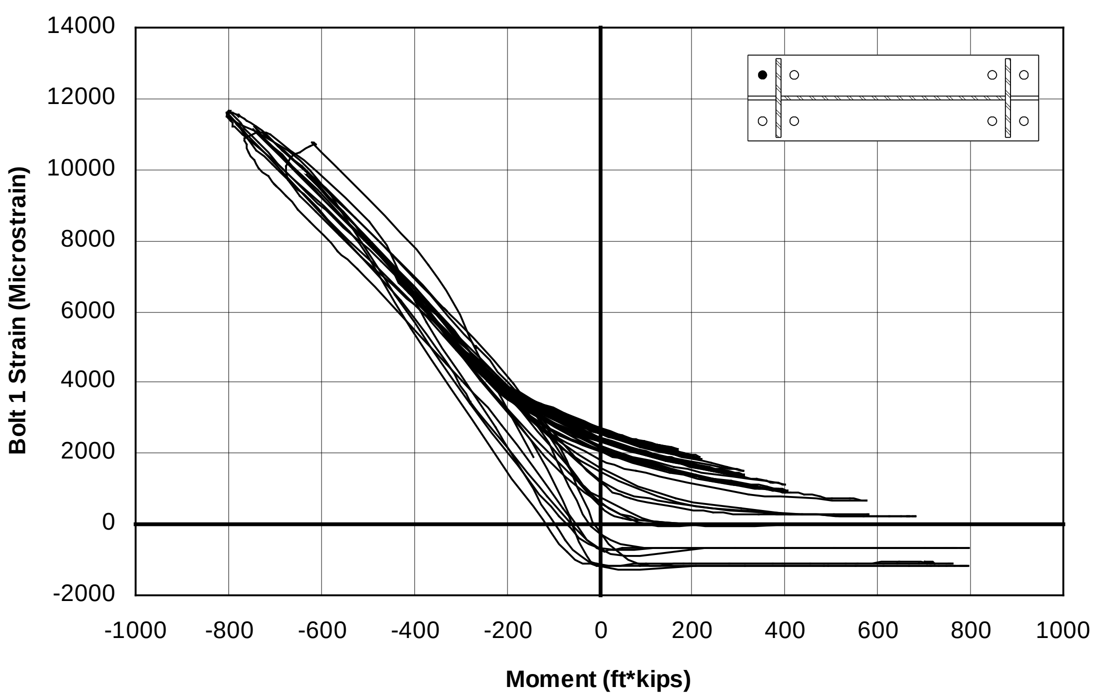


FIGURE 4.6: 4ES-1.0-1.0-24 BOLT RESPONSE

4.2.3 MRE1/3-0.75-0.75-30

The MRE1/3-0.75-0.75-30 specimen began to exhibit inelastic behavior in the 0.02 rad Load Step. The total inelastic behavior, shown in Figure 4.7, consisted of beam yielding and end-plate separation from the column flange, with beam yielding comprising the majority of the inelastic behavior. The panel zone exhibited negligible inelastic behavior.

Loss of bolt pretension occurred at and above the 0.02 rad Load Step, eventually going to zero in the four outermost bolts at each flange by the end of the 0.03 rad Load Step. The four innermost bolts at each flange exhibited an average total loss of approximately half of the original pretension. Flange local buckling was observed during the 0.02 rad Load Step, and web local buckling was observed during the 0.03 rad Load Step. The test was stopped at the end of the 0.04 rad Load Step, because the specimen's moment strength had been halved. Figure 4.8 shows the specimen after testing, and Figure 4.9 is a plot of typical bolt behavior.

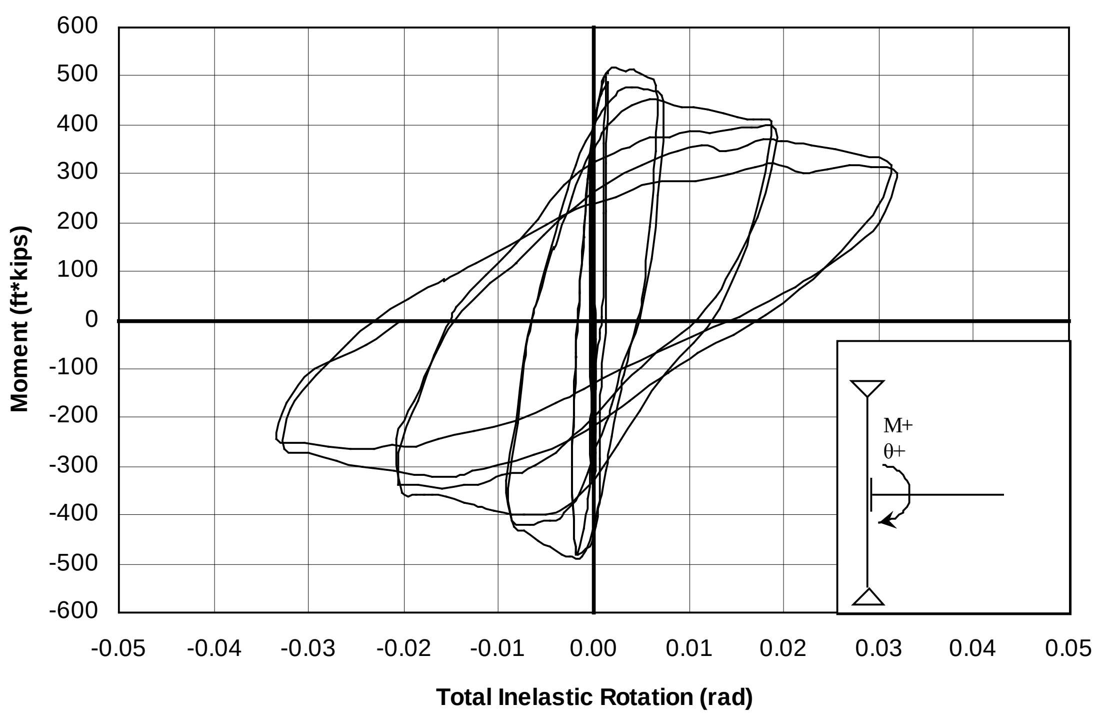
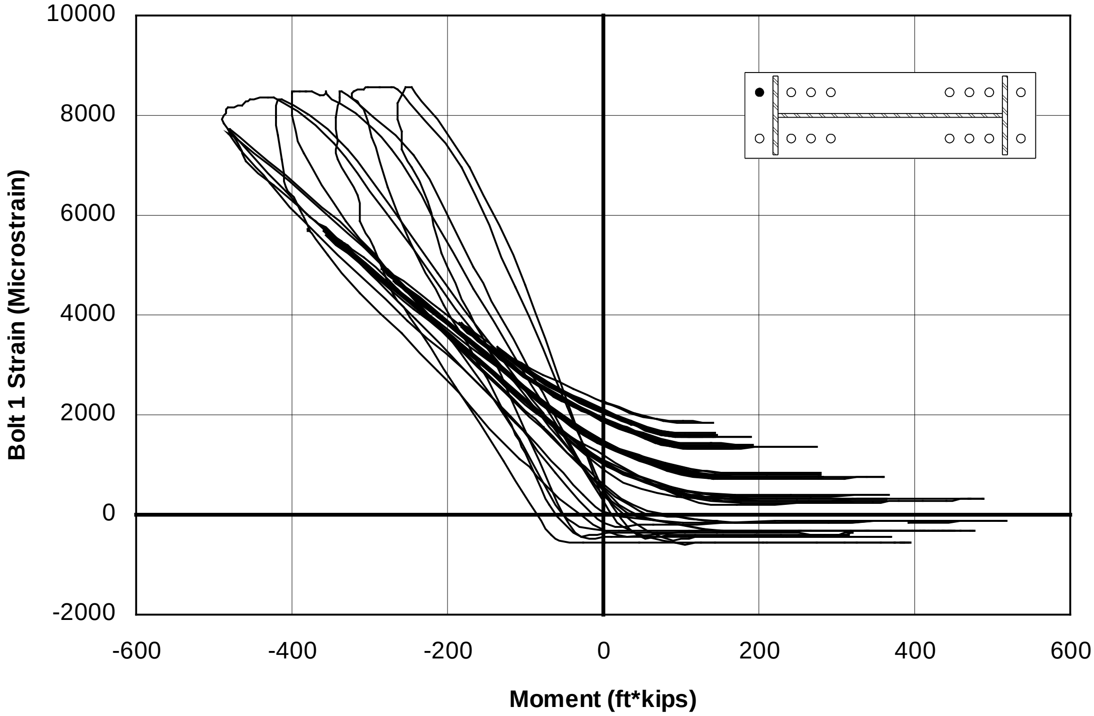
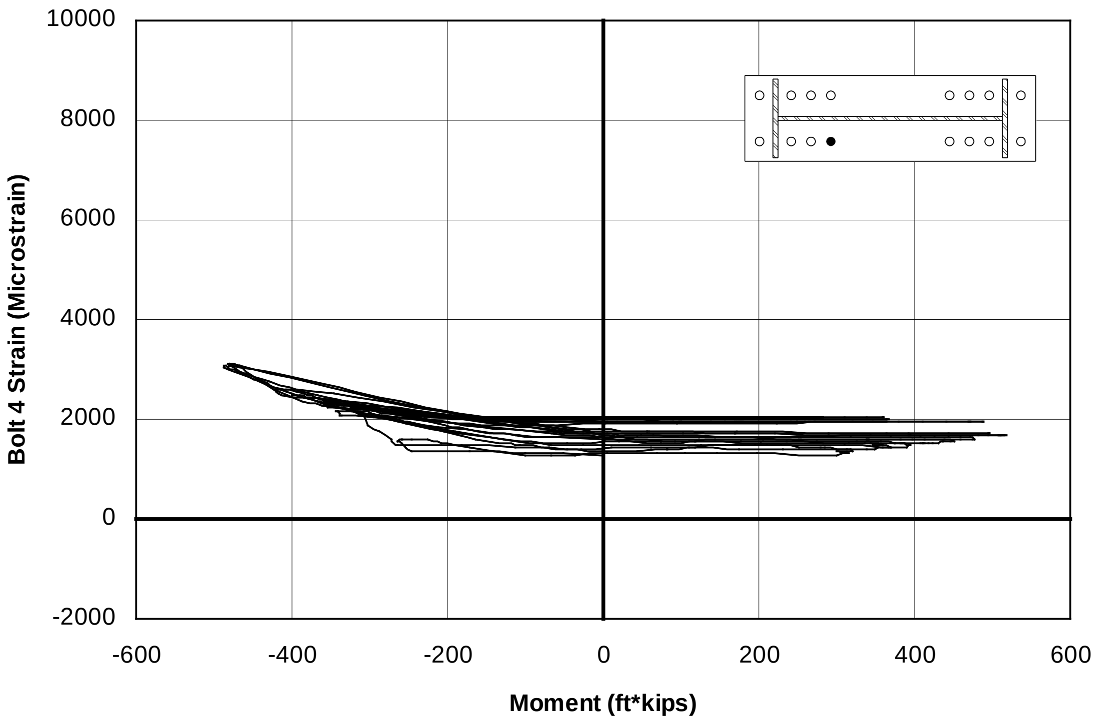


FIGURE 4.7: MRE1/3-0.75-0.75-30 TOTAL INELASTIC RESPONSE

FIGURE 4.8: MRE1/3-0.75-0.75-30 AFTER TESTING



a) Typical Outermost Bolt



b) Typical Innermost Bolt

FIGURE 4.9: MRE1/3-0.75-0.75-30 BOLT RESPONSE

4.2.4   MRE1/3S-0.75-0.625-30

The MRE1/3S-0.75-0.625-30 specimen began to exhibit inelastic behavior in the 0.02 rad Load Step. The total inelastic behavior, shown in Figure 4.10, was comprised of beam yielding and end-plate separation from the column flange, with beam yielding comprising the majority of the inelastic behavior. The panel zone exhibited negligible inelastic behavior.

Loss of bolt pretension occurred at and above the 0.02 rad Load Step, eventually going to zero in the four outermost bolts at each flange by the end of the 0.03 rad Load Step. The four innermost bolts at each flange exhibited an average total loss of between one-quarter and one-half of the original pretension. Flange local buckling was observed during the 0.02 rad Load Step, and web local buckling was observed during the 0.03 rad Load Step. During the second half of the first cycle of the 0.04 Load Step, one of the outermost bolts ruptured on the tension side of the connection, at an interstory drift of 0.033 rad. Figure 4.11 shows the specimen after testing, and Figure 4.12 is a plot of typical bolt behavior.

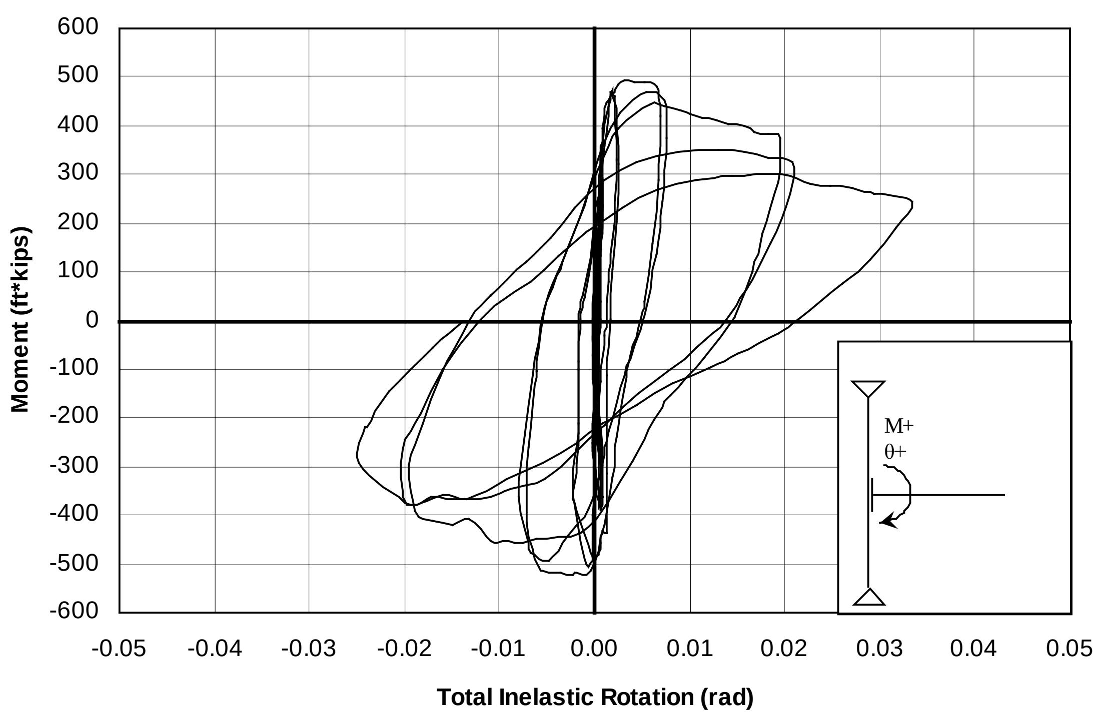


FIGURE 4.10: MRE1/3S-0.75-0.625-30 TOTAL INELASTIC RESPONSE

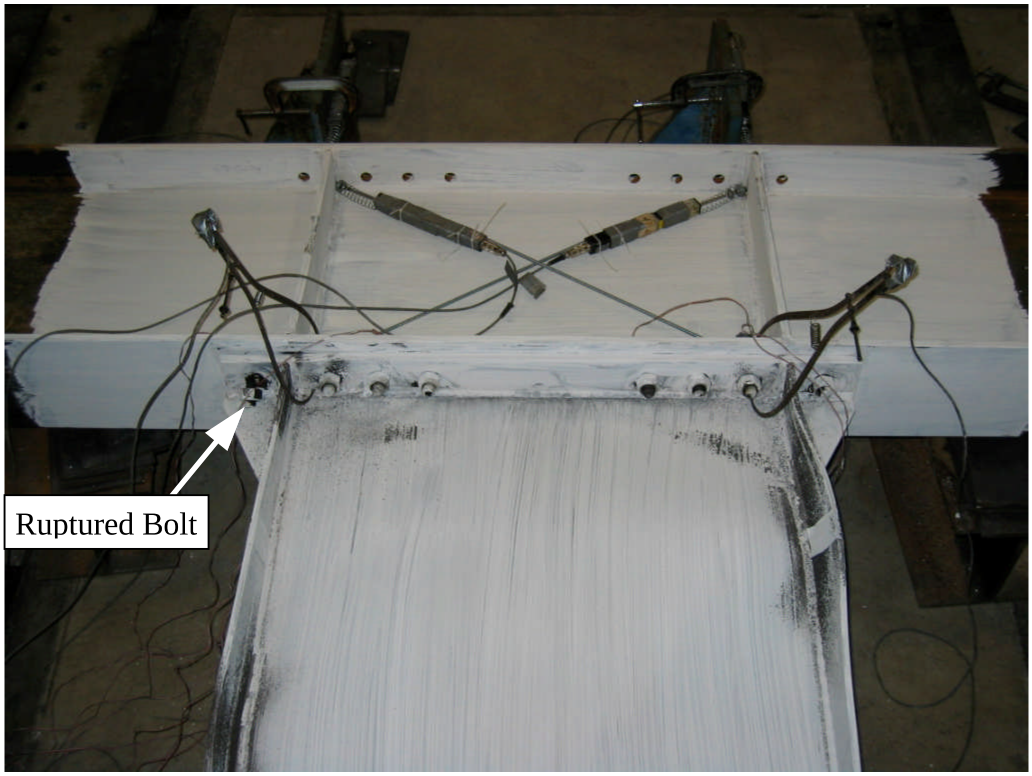
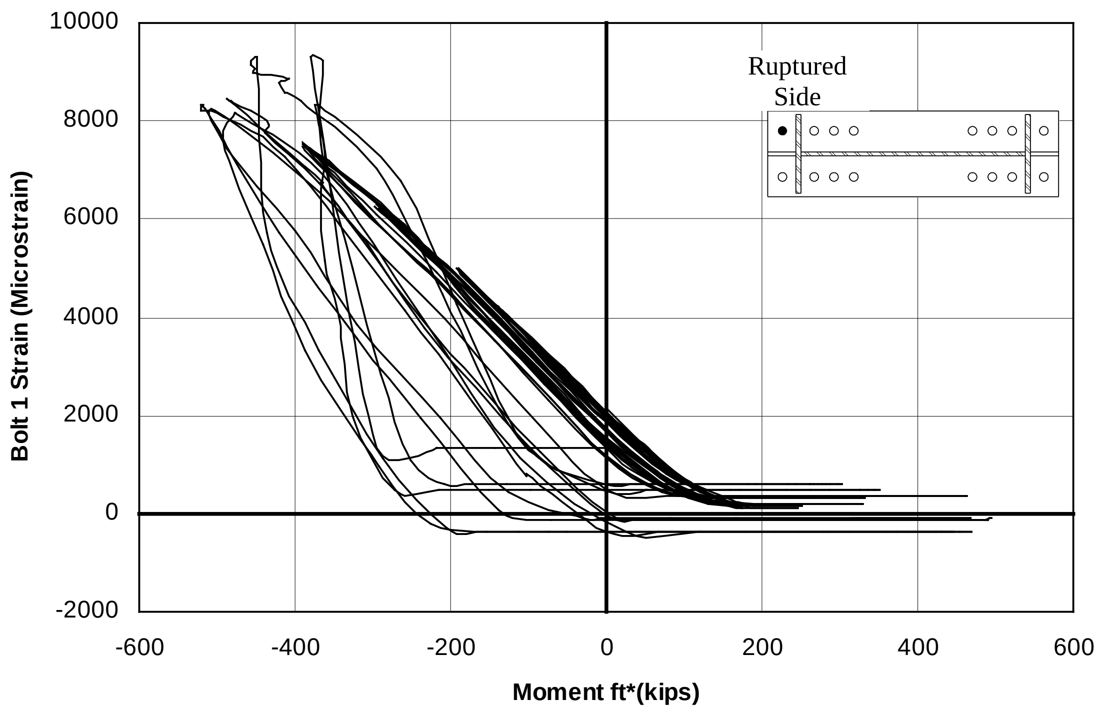
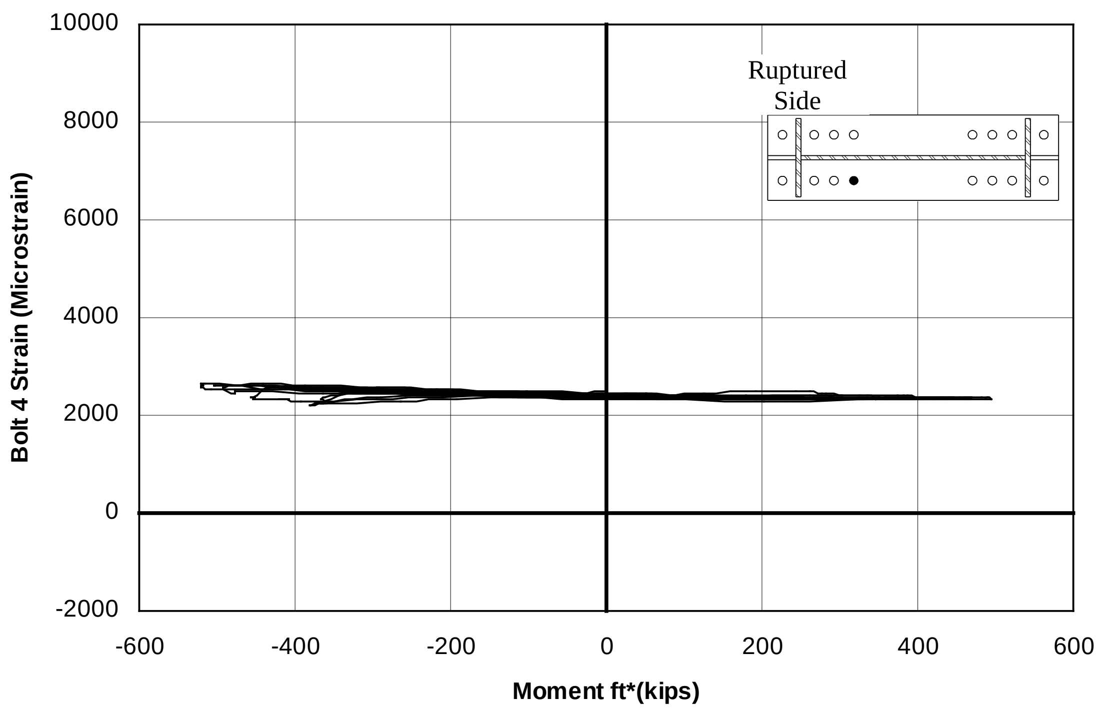


FIGURE 4.11: MRE1/3S-0.75-0.625-30 AFTER TESTING



a) Typical Outermost Bolt



b) Typical Innermost Bolt

FIGURE 4.12: MRE1/3S-0.75-0.625-30 BOLT RESPONSE

4.2.5 4E-1.25-1.25-60 A

The 4E-1.25-1.25-60 A specimen began to exhibit inelastic behavior in the 0.015 rad Load Step. The total inelastic behavior, shown in Figure 4.13, was comprised of beam yielding, end-plate separation from the column flange, and panel zone yielding. Beam yielding and end-plate separation comprised the majority of the inelastic behavior. End-plate deformation, shown in Figure 4.14, was evident at the end of testing.

Loss of bolt pretension occurred at and above the 0.015 rad Load Step, eventually going to zero in all bolts by the end of the 0.02 rad Load Step. During the first half of the first cycle of the 0.03 Load Step, all four bolts ruptured on the tension side of the connection at an interstory drift of 0.022 rad. Then, the side of the specimen that did not rupture was loaded with half cycles through the end of the 0.04 rad Load Step. At this point flange local buckling was evident, and the specimen exhibited 50 percent strength loss. The test was stopped at this point due to considerable column twisting. Figure 4.15 is a plot of typical bolt behavior.

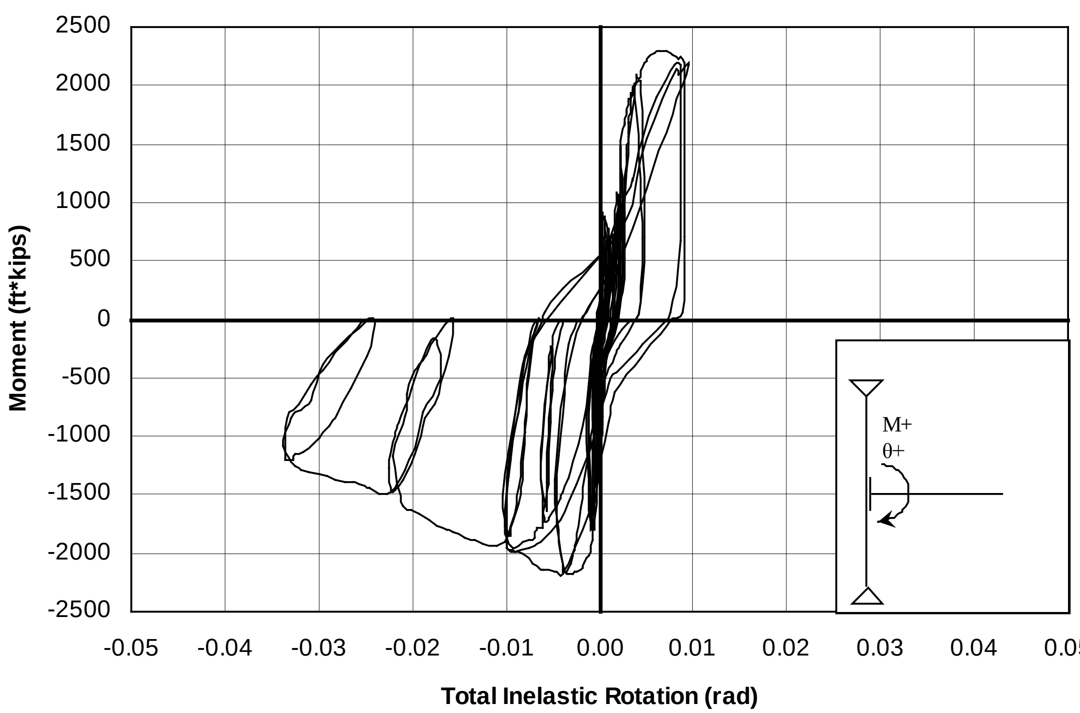


FIGURE 4.13: 4E-1.25-1.25-60 A TOTAL INELASTIC RESPONSE

FIGURE 4.14: 4E-1.25-1.25-60 A AFTER BOLT RUPTURE

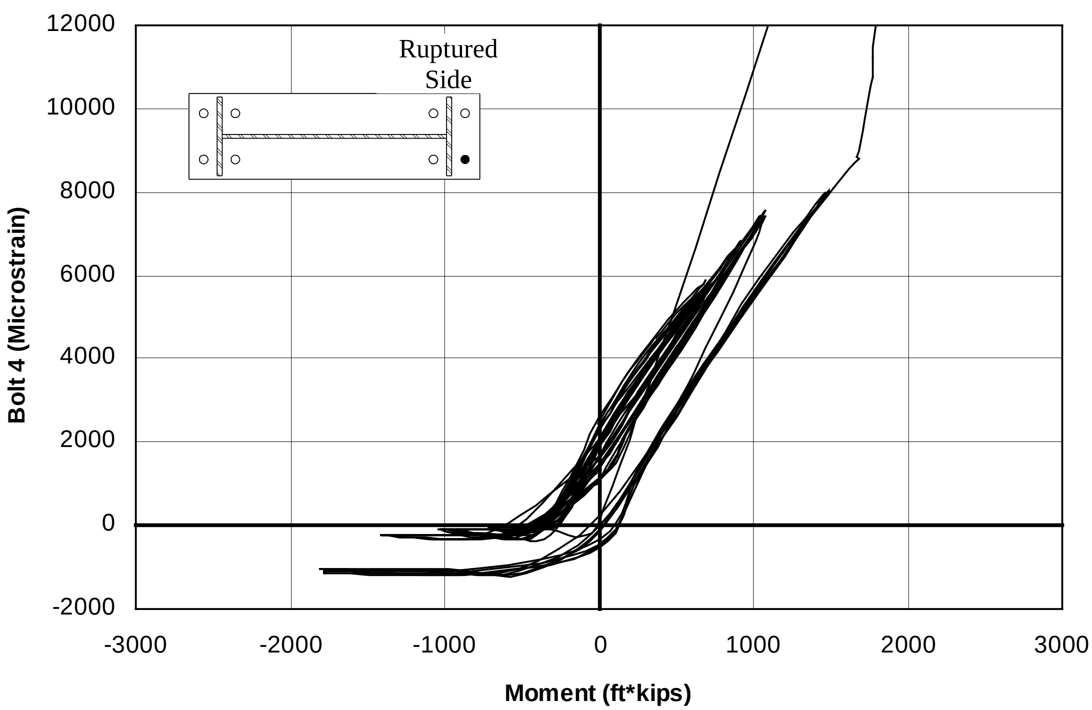


FIGURE 4.15: 4E-1.25-1.25-60 A RUPTURED BOLT RESPONSE

4.2.6 4E-1.25-1.25-60 B

The 4E-1.25-1.25-60 B specimen began to exhibit inelastic behavior in the 0.015 rad Load Step. The total inelastic behavior, shown in Figure 4.16, was comprised almost entirely of beam yielding; end-plate separation from the column flange and panel zone yielding were negligible.

Loss of bolt pretension occurred at and above the 0.015 rad Load Step, eventually going to zero in the exterior instrumented bolts by the end of the 0.02 rad Load Step. During the 0.01 rad and 0.015 rad Load Steps, whitewash flaking was evident on the beam flanges. Flange local buckling was evident during the 0.02 rad Load Steps, and the specimen exhibited a 10 percent loss of strength. During the 0.03 rad Load Step, local flange buckling increased, and strength loss was rapid. By the end of this step, the specimen exhibited a 50 percent loss of strength. The test was stopped at this point. Figure 4.17 shows the specimen after testing, and Figure 4.18 is a plot of typical bolt behavior.

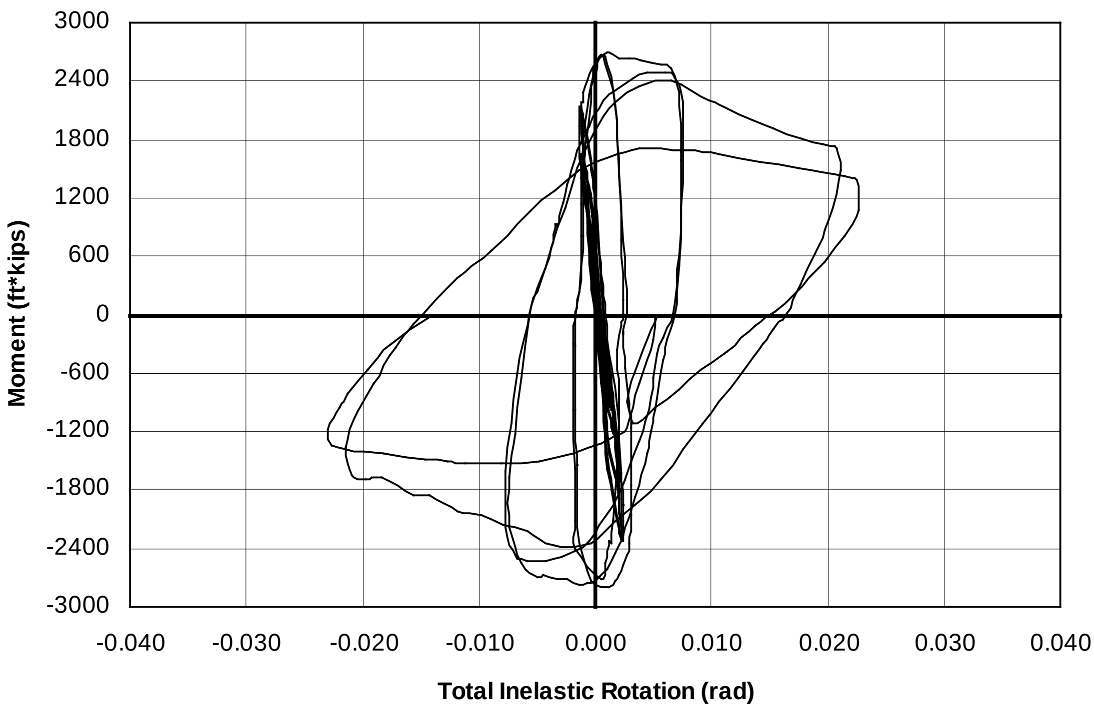


FIGURE 4.16: 4E-1.25-1.25-60 B TOTAL INELASTIC RESPONSE

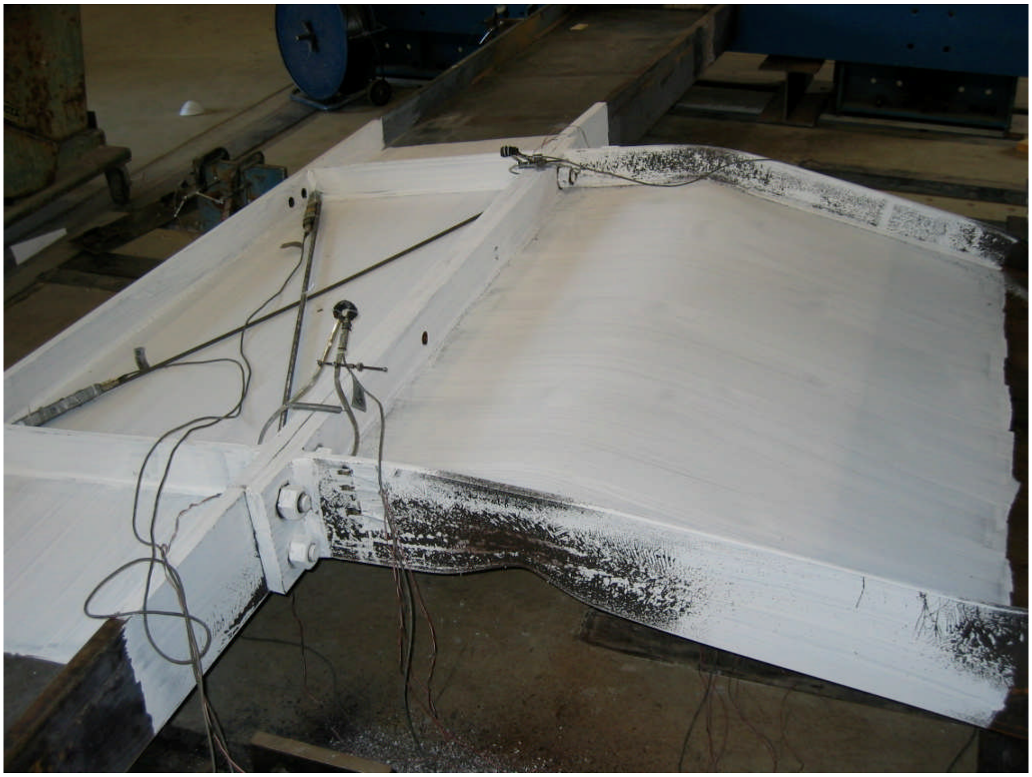


FIGURE 4.17: 4E-1.25-1.25-60 B AFTER TESTING

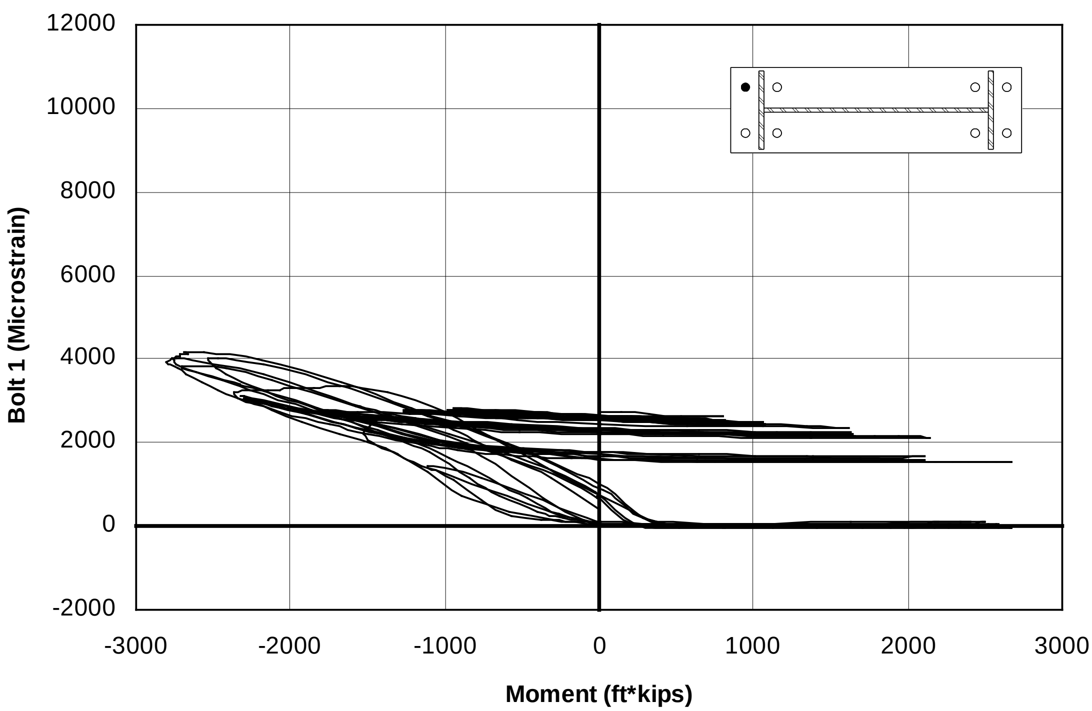


FIGURE 4.18: 4E-1.25-1.25-60 B BOLT RESPONSE

4.2.7 MRE1/2-1.0-1.0-60 A

The MRE1/2-1.0-1.0-60 A specimen began to exhibit inelastic behavior in the 0.015 rad Load Step. The total inelastic behavior, shown in Figure 4.19, was negligible and comprised almost entirely of end-plate separation from the column flange.

Loss of bolt pretension occurred at and above the 0.01 rad Load Step. Yielding of the outermost bolts prior to failure was indicated by permanent set of the bolt strains during the 0.01 rad Load Step. At no point during the test was whitewash flaking or local buckling evident. During the first half of the first cycle of the 0.015 rad Load Step, the four outermost bolts ruptured on the tension side of the connection. At the same time, approximately 13 in. of the weld connecting the column panel zone web to the column flange ruptured in the vicinity of the ruptured bolts. The test was stopped at this point. Figure 4.20 shows the specimen after testing, and Figure 4.21 is a plot of typical bolt behavior on the ruptured side of the connection.

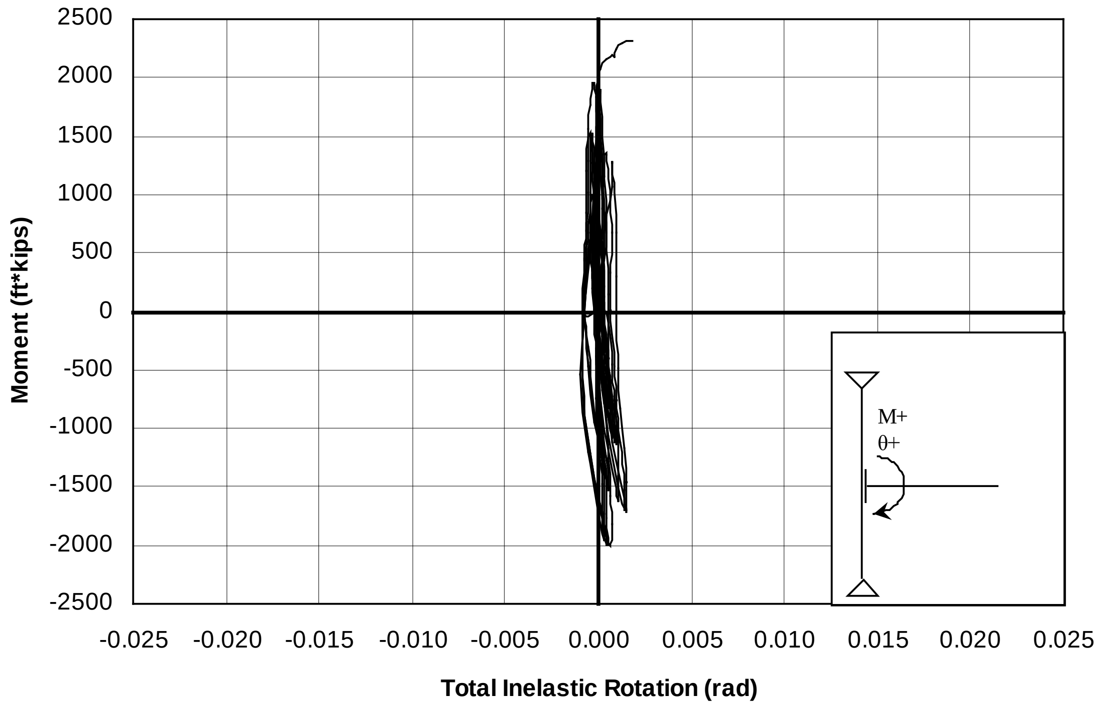


FIGURE 4.19: MRE1/2-1.0-1.0-60 A TOTAL INELASTIC RESPONSE

FIGURE 4.20: MRE1/2-1.0-10-60 A AFTER BOLT AND WELD RUPTURE

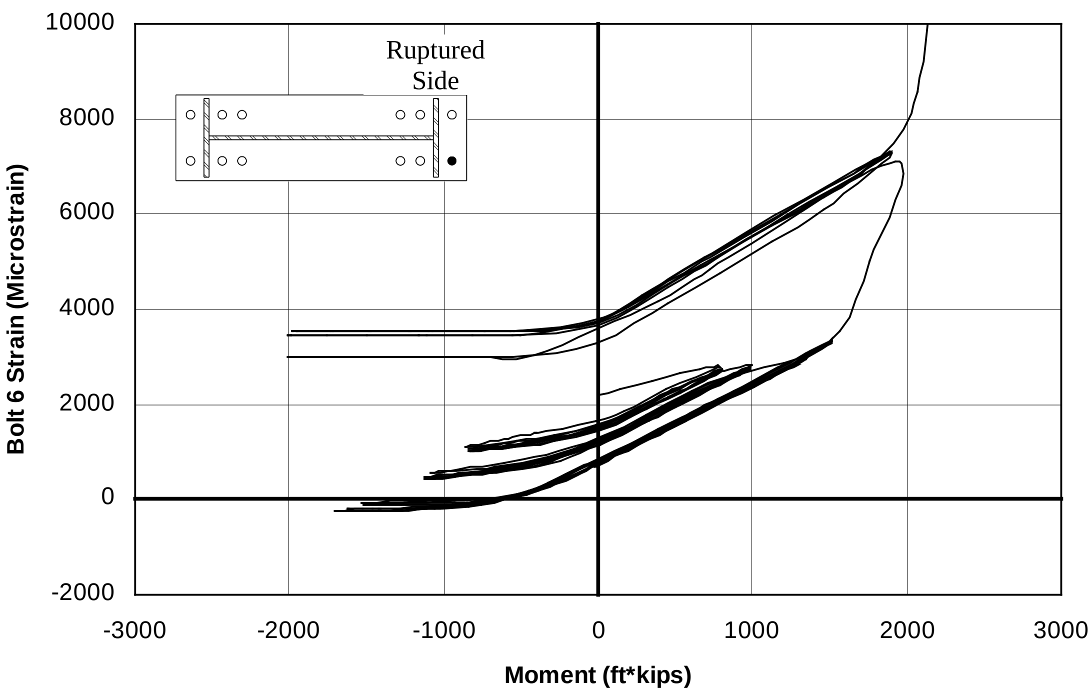


FIGURE 4.21: MRE1/2-1.0-1.0-60 A RUPTURED BOLT RESPONSE

4.2.8 MRE1/2-1.0-1.0-60 B

The MRE1/2-1.0-1.0-60 B specimen began to exhibit inelastic behavior in the 0.015 rad Load Step. The total inelastic behavior, shown in Figure 4.22, was comprised by beam yield and end-plate separation from the column flange.

Loss of bolt pretension occurred at and above the 0.01 rad Load Step, decreasing to zero by the end of the 0.015 rad Load Step. Whitewash flaking was evident during the 0.015 rad Load Step. During the first half of the first cycle of the 0.02 rad Load Step, slight flange local buckling was evident. During the second half of the same cycle, all six bolts ruptured on the tension side of the connection at an interstory drift of 0.016 rad. At this point, there was significantly less whitewash flaking on the flange opposite of the side where the bolts ruptured compared to the flange that had buckled. The non-ruptured side of the connection was loaded with half cycles through the first half of the first cycle of the 0.03 rad Load Step. The specimen exhibited a 20 percent loss of strength, and the test was stopped. Figure 4.23 shows the specimen after bolt rupture; Figure 4.24 is a plot of typical bolt behavior on the ruptured side.

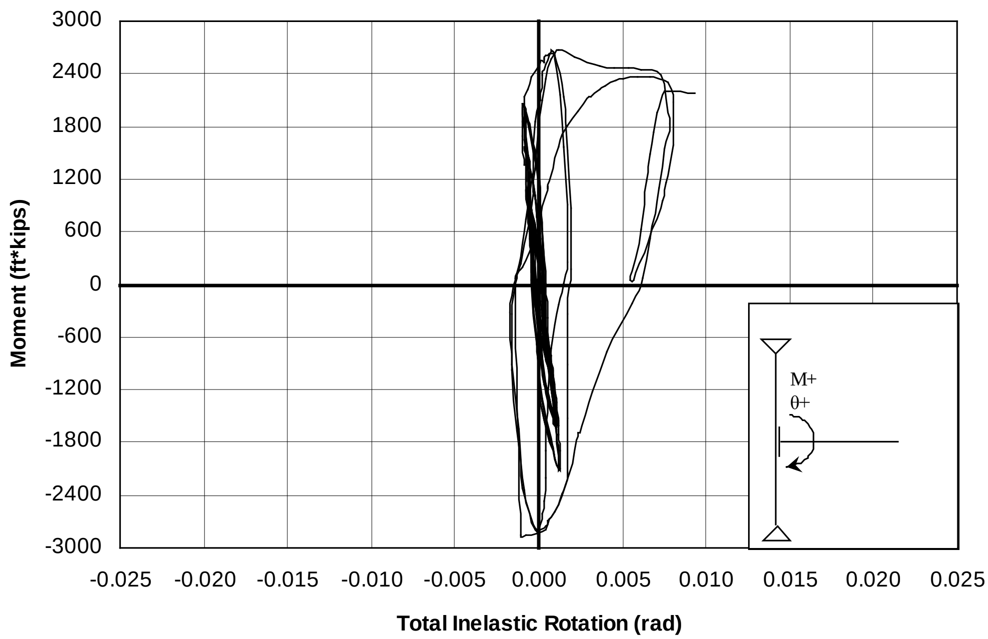


FIGURE 4.22: MRE1/2-1.0-1.0-60 B TOTAL INELASTIC RESPONSE

FIGURE 4.23: MRE1/2-1.0-1.0-60 B AFTER BOLT RUPTURE

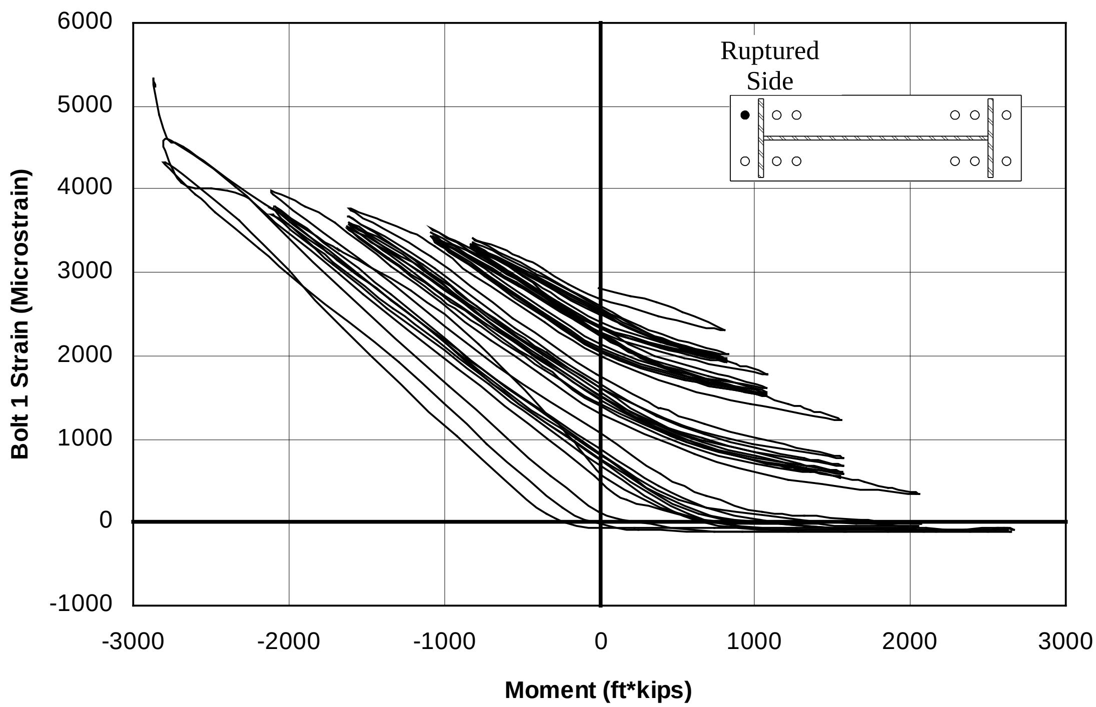


FIGURE 4.24: MRE1/2-1.0-1.0-60 B RUPTURED BOLT RESPONSE

4.2.9 MRE1/3S-1.25-1.25-72 A

The MRE1/3S-1.25-1.25-72 A specimen began to exhibit inelastic behavior in the 0.015 rad Load Step. The total inelastic behavior, shown in Figure 4.25, was negligible and comprised almost entirely of end-plate separation from the column flange.

Loss of bolt pretension occurred at and above the 0.0075 rad Load Step, with the bolts losing approximately 50 percent of the original pretension by the end of the 0.01 rad Load Step. Whitewash flaking on the beam flanges became evident during the 0.01 rad Load Step. During the first half of the first cycle of the 0.015 rad Load Step, one of the outermost bolts ruptured at an interstory drift angle of 0.012 rad. The specimen immediately exhibited a 10 percent strength decrease. At this point the specimen was unloaded, and then loaded in the opposite (negative) direction. The specimen attained an interstory drift of 0.015 rad in the negative direction. Then the specimen was unloaded and the test was stopped, because of the bolt rupture. Figure 4.26 is a plot of typical bolt behavior on the ruptured side of the connection, and Figure 4.27 shows the specimen after bolt rupture.

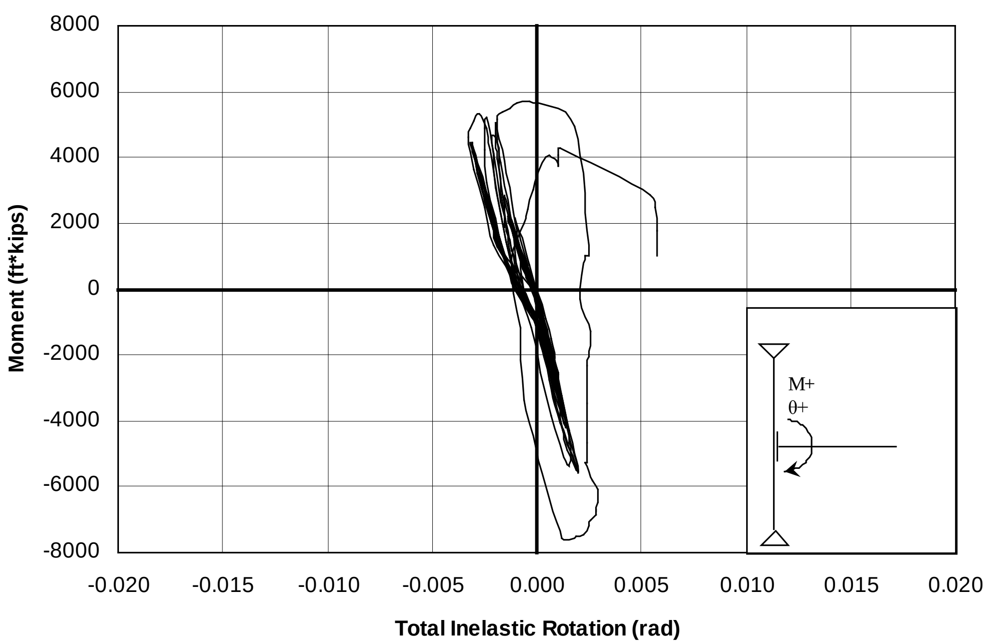


FIGURE 4.25: MRE1/3S-1.25-1.25-72 A TOTAL INELASTIC RESPONSE

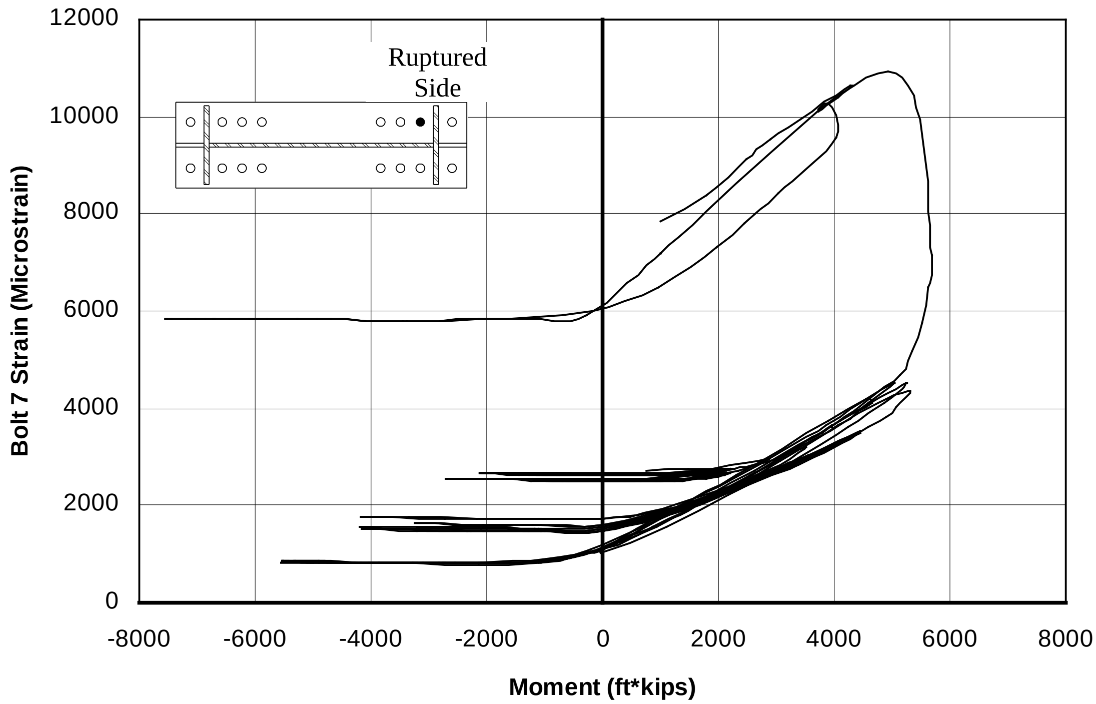
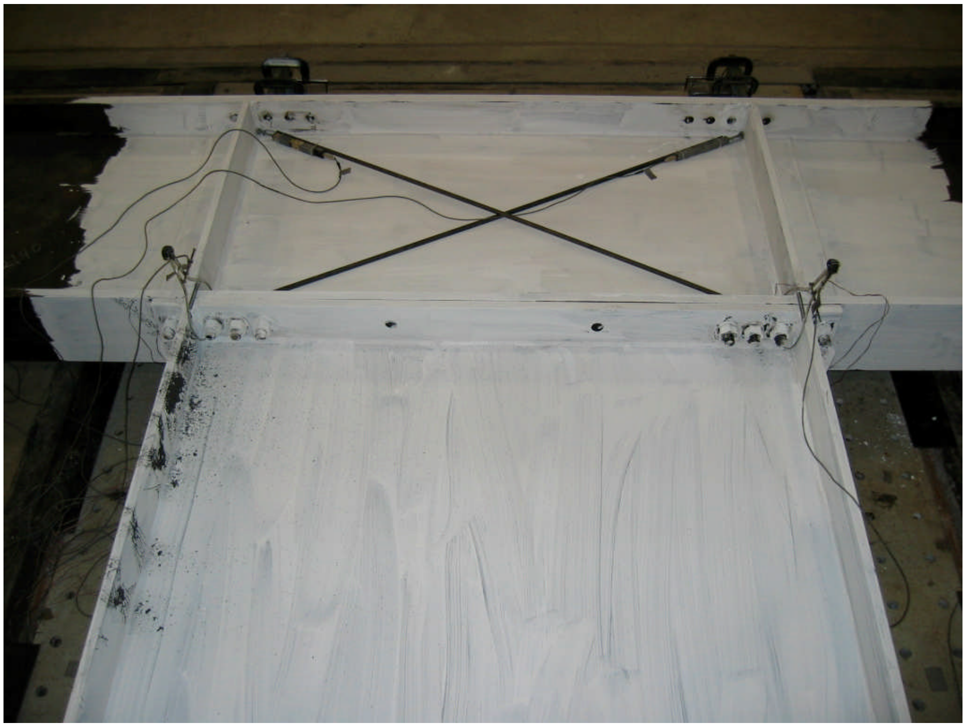
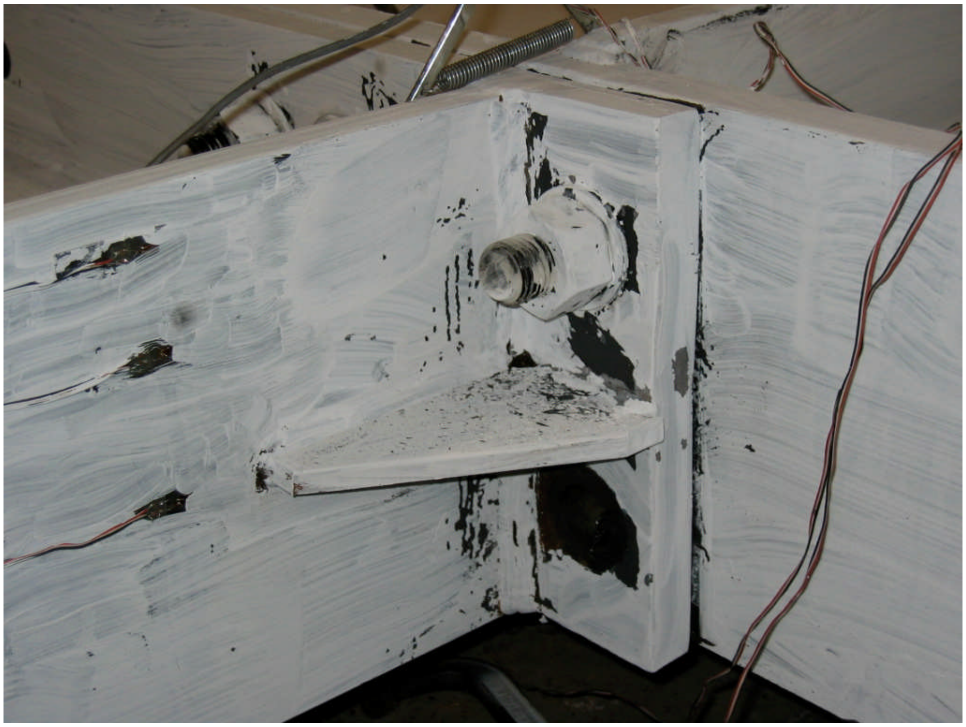


FIGURE 4.26: MRE1/3S-1.25-1.25-72 A BOLT RESPONSE



a) Overall View of Connection Region



b) Area Near Ruptured Bolt

FIGURE 4.27: MRE1/3S-1.25-1.25-72 A AFTER BOLT RUPTURE

4.2.10 MRE1/3S-1.25-1.25-72 B

The MRE1/3S-1.25-1.25-72 B specimen began to exhibit inelastic behavior in the 0.015 rad Load Step. The total inelastic behavior, shown in Figure 4.28, was negligible and comprised almost entirely of end-plate separation from the column flange.

Loss of bolt pretension occurred at and above the 0.0075 rad Load Step. By the end of the 0.01 rad Load Step, the outermost bolts had lost all of the original pretension, and the innermost bolts had lost approximately 50 percent of the original pretension. Whitewash flaking on the beam flanges became evident during the 0.0075 rad Load Step. During the first half of the first cycle of the 0.015 rad Load Step, one of the outermost bolts ruptured at an interstory drift angle of 0.013 rad. The specimen immediately exhibited a 10 percent strength decrease. At this point the specimen was unloaded, and then loaded in the opposite (negative) direction. At an interstory drift angle of 0.013 rad, the two outermost bolts were stripped of their threads by the attached nuts. The specimen immediately exhibited a 7 percent strength decrease. Then the specimen was unloaded and the test was stopped, because of the bolt rupture. Figure 4.29 is a plot of typical bolt behavior on the ruptured side of the connection, and Figure 4.30 shows each side of the connection after testing.

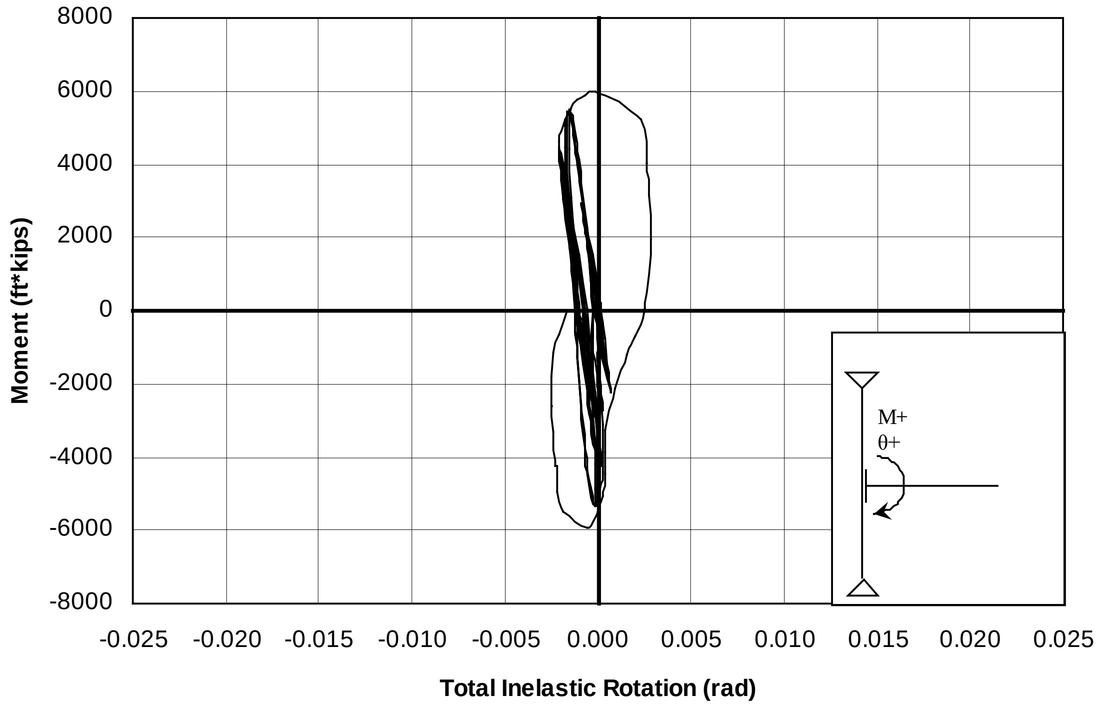


FIGURE 4.28: MRE1/3S-1.25-1.25-72 B TOTAL INELASTIC RESPONSE

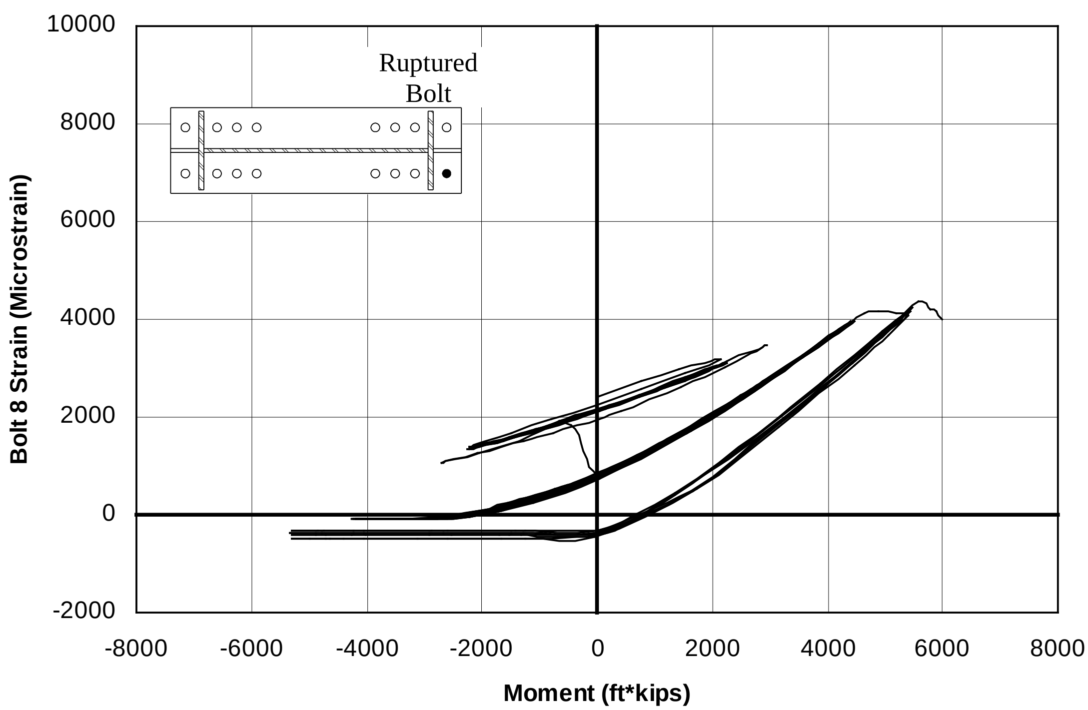
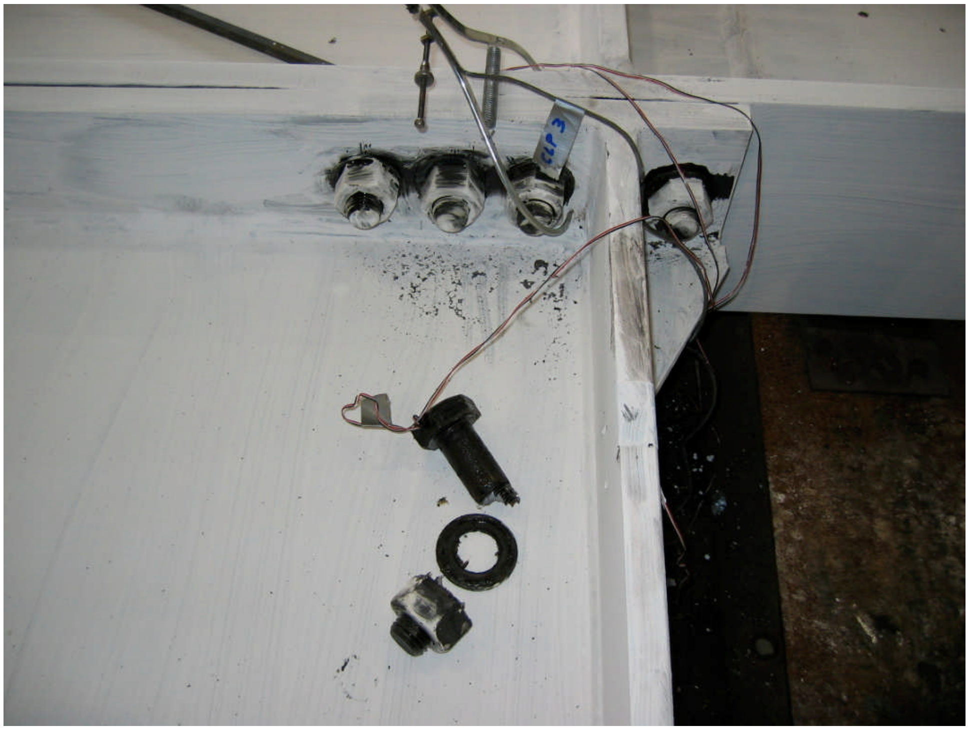
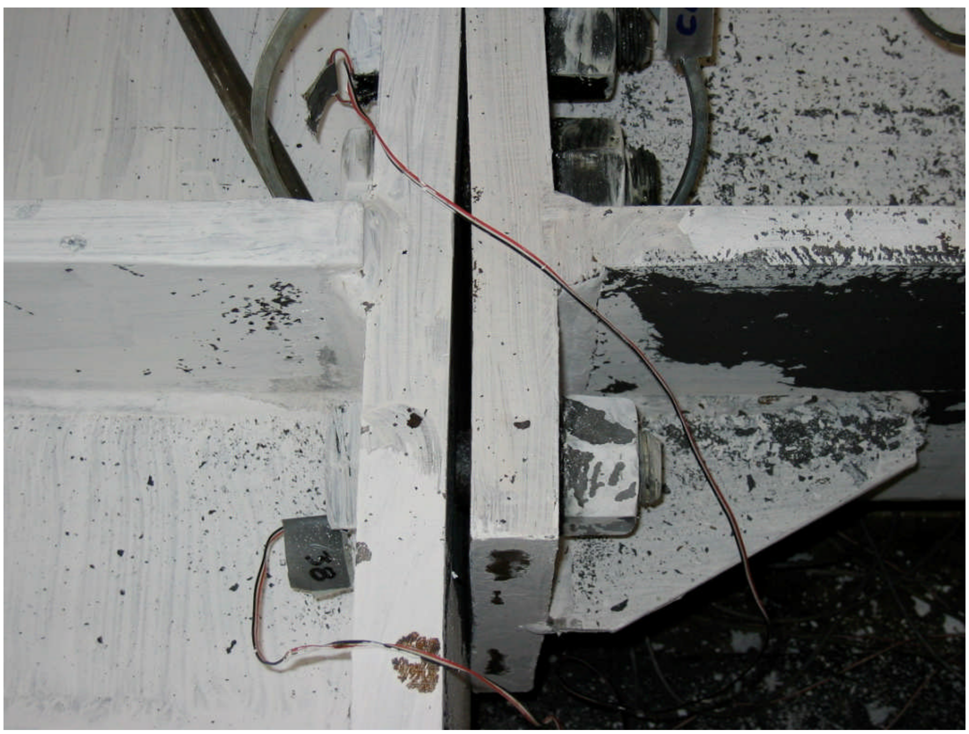


FIGURE 4.29: MRE1/3S-1.25-1.25-72 B BOLT RESPONSE



a) Bolt Rupture



b) Thread Stripping

FIGURE 4.30: MRE1/3S-1.25-1.25-72 B BRITTLE FAILURES

4.2.11 MRE1/3-1.25-1.5-72

The MRE1/3-1.25-1.5-72 specimen began to exhibit inelastic behavior in the 0.015 rad Load Step. The total inelastic behavior, shown in Figure 4.31, was comprised of end-plate separation from the column flange and small contributions from beam yielding and panel zone yielding.

Loss of bolt pretension occurred at and above the 0.0075 rad Load Step. By the end of the 0.015 rad Load Step, the outermost bolts had lost all of the original pretension, and the innermost bolts had lost approximately 50 percent of the original pretension. Whitewash flaking on the column flange adjacent to the end-plate became evident during the 0.0075 rad Load Step. Whitewash flaking on the beam flange became evident during the 0.015 rad Load Step. During the first half of the first cycle of the 0.02 rad Load Step, the four outermost bolts were stripped of their threads by the attached nuts. The specimen immediately exhibited a 20 percent strength decrease. At this point the specimen was unloaded, and the test was stopped. Figure 4.32 is a plot of typical bolt behavior for the stripped bolts, and Figure 4.33 shows the specimen after testing. Figure 4.34 shows one of the stripped bolts after the connection was tested and disassembled.

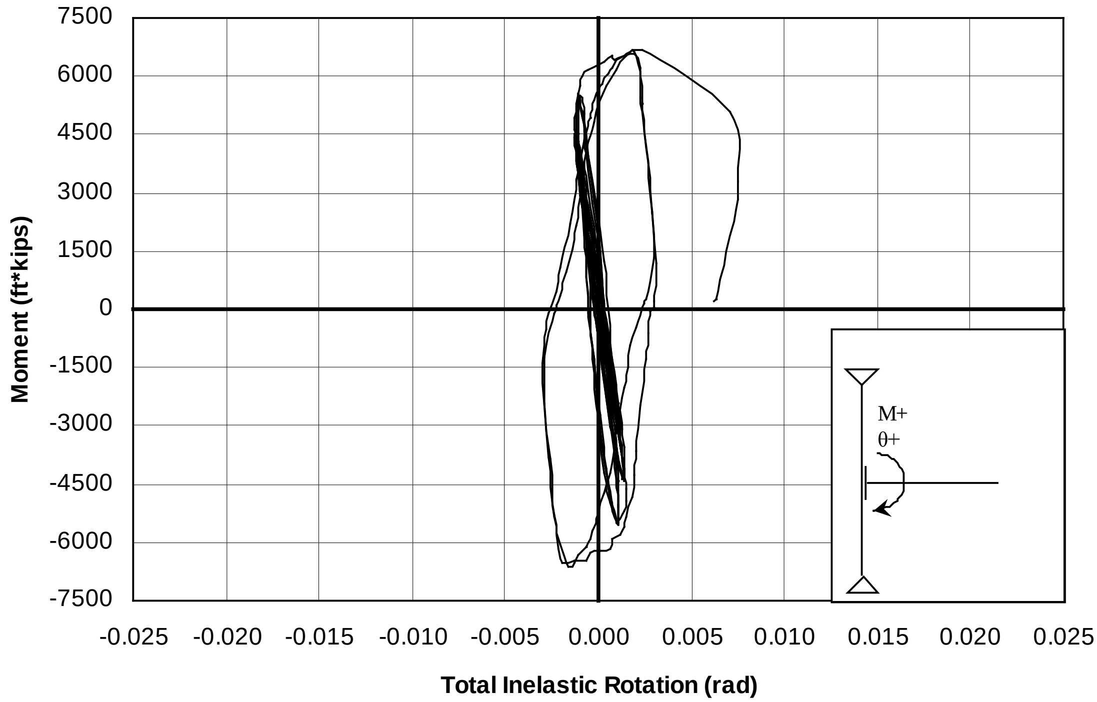


FIGURE 4.31: MRE1/3-1.25-1.5-72 TOTAL INELASTIC RESPONSE

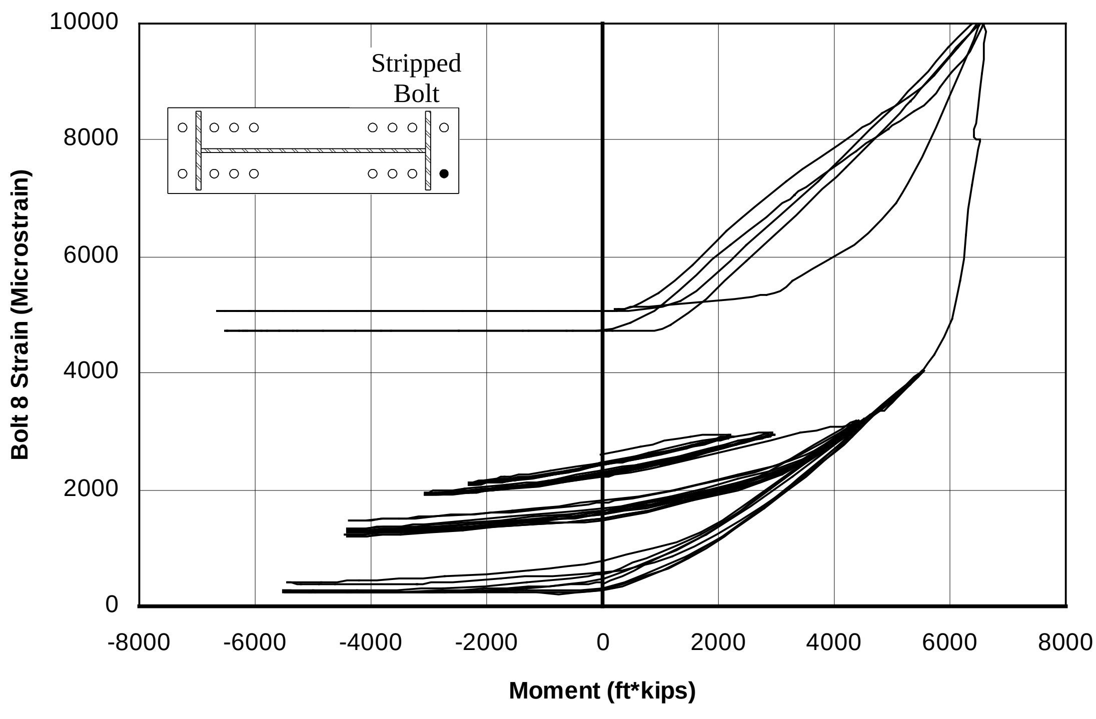
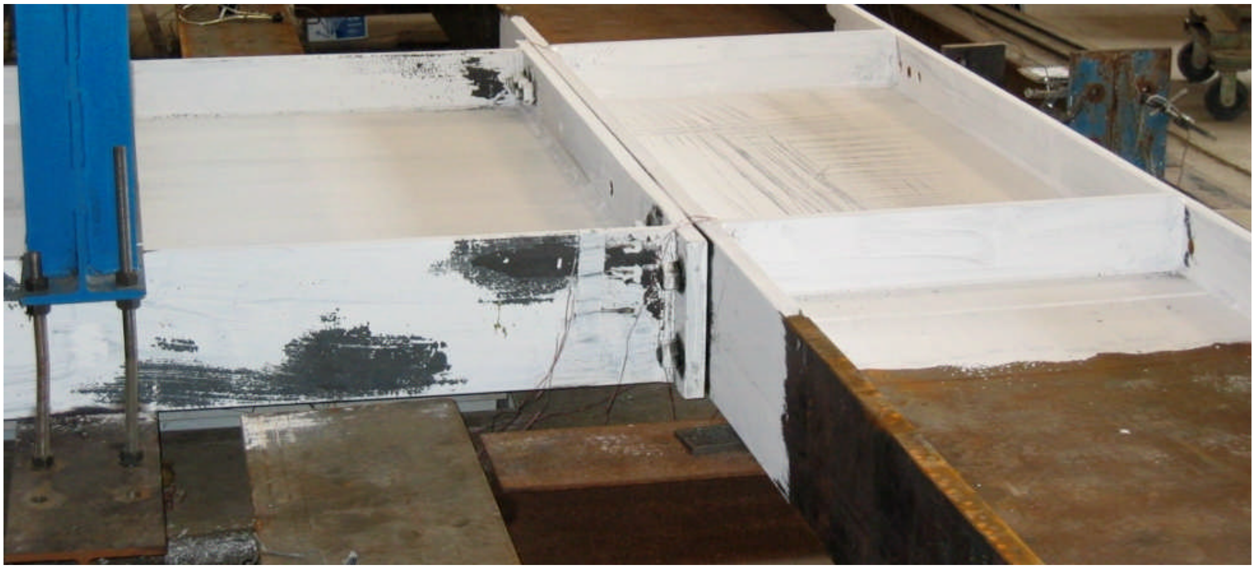
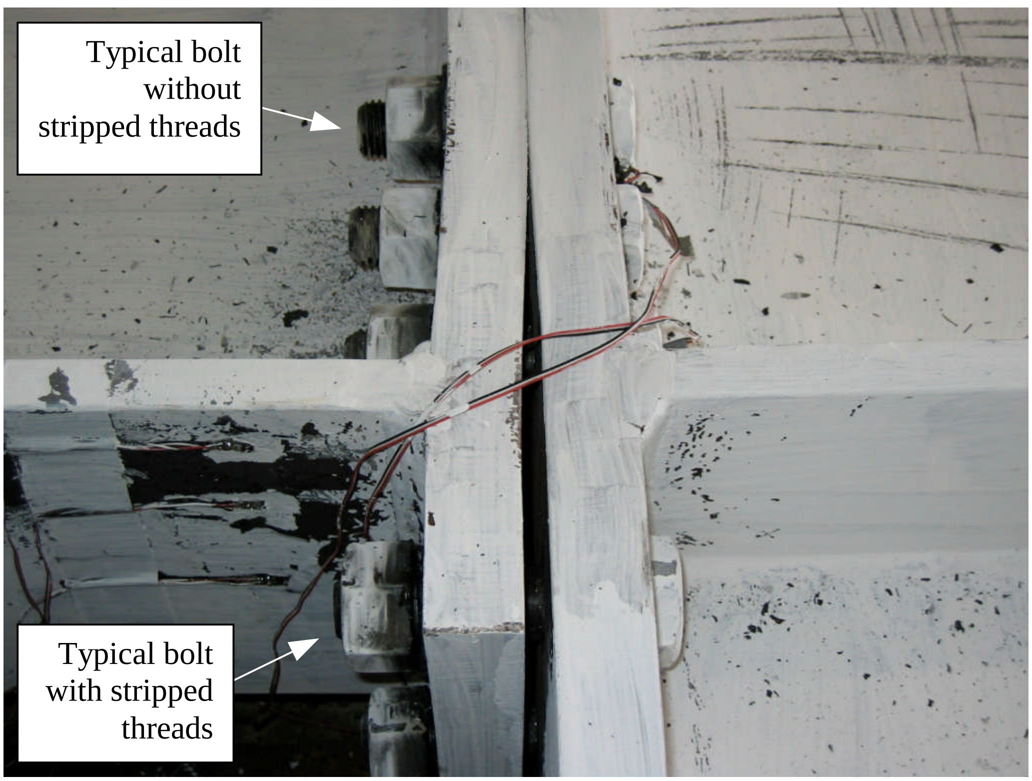


FIGURE 4.32: MRE1/3-1.25-1.5-72 BOLT RESPONSE



a) Overall View of Connection Region



b) Thread Stripping

FIGURE 4.33: MRE1/3-1.25-1.5-72 AFTER TESTING

FIGURE 4.34: MRE1/3-1.25-1.5-72 TYPICAL BOLT WITH STRIPPED THREADS

4.3 SUMMARY

The performances of the eleven test specimens are summarized in Table 4.1. The ratio of the maximum applied moment ($M_{max,fs}$) at the faying surface to the following is shown: beam moment strength (M_b), beam plastic strength (M_p), and the beam expected plastic strength (M_{pe}). Measured material properties were used to calculate M_b and M_p; the yield and tensile stresses shown in Table 2-1 of the LRFD Manual (AISC 2001) were used to calculate M_{pe}. A ratio greater than one is unconservative with respect to connection design, because the connection design moment strength was exceeded. Also shown is the maximum interstory drift angle θ_{max}, and the corresponding applied moment at the column centerline, $M_{\theta max}$. Note that θ_{max} is not the interstory drift angle for the last complete cycle, but is the largest rotation achieved by the Test Specimen.

TABLE 4.1: SUMMARY OF SPECIMEN PERFORMANCE

Specimen ID	$\frac{M_{max,fs}}{M_b}$	$\frac{M_{max,fs}}{M_p}$	$\frac{M_{max,fs}}{M_{pe}}$	θ_{max} (rad)	$M_{\theta max}$ (ft*kips)
MRE1/2 - 0.875 - 1.0 - 24	1.03	1.03	0.95	0.030	759.1
4ES - 1.0 - 1.0 - 24	1.09	1.09	1.01	0.050	722.3
MRE1/3 - 0.75 - 0.75 - 30	0.91	0.78	0.73	0.040	245.1
MRE1/3S - 0.75 - 0.625 - 30	0.92	0.79	0.73	0.040	243.6
4E - 1.25 - 1.25 - 60 A	0.97	0.72	0.69	0.040	1183.7
4E - 1.25 - 1.25 - 60 B	1.00	0.77	0.84	0.030	1342.9
MRE1/2 - 1.0 - 1.0 - 60 A	0.83	0.63	0.70	0.014	2310.6
MRE1/2 - 1.0 - 1.0 - 60 B	1.03	0.79	0.86	0.021	2174.7
MRE1/3S - 1.25 - 1.25 - 72 A	1.27	1.08	0.91	0.014	7553.5
MRE1/3S - 1.25 - 1.25 - 72 B	0.96	0.72	0.72	0.013	5975.8
MRE1/3 - 1.25 - 1.5 - 72	1.08	0.81	0.81	0.016	6688.1

CHAPTER 5 - ANALYSIS OF RESULTS

5.1 GENERAL

The results of eleven beam-to-column extended end-plate moment connections were presented in the previous chapter. In this chapter, the results are compared to the following: the predicted values using the thick plate procedure from AISC Design Guide 16, the predicted values of beam strength, and the AISC Seismic Provisions criteria for connection prequalification.

5.2 CONNECTION STRENGTH

5.2.1 Experimental and Analytical Comparisons

Comparisons of the experimental and analytical results for connection strength are shown in Table 5.1. The end-plate design ratio was calculated by dividing the end-plate strength, M_{PL} (from yield line analysis), by the maximum applied moment calculated at the connection faying surface, $M_{max,fs}$. The bolt design ratio is calculated by dividing the no-prying strength of the connection bolts, M_{NP}, by the maximum applied moment at the connection faying surface, $M_{max,fs}$. Measured material properties were used to calculate M_{PL}, and nominal bolt tensile stresses were used to calculate M_{NP}. Calculations of M_{NP} and M_{PL} are located in the Appendices. Also noted is whether or not any inelastic beam behavior was observed.

For the *smaller* of the two ratios, design ratios less than or equal to unity are conservative since the minimum predicted strength is lower than the maximum applied moment. If the smaller design ratio is greater than unity, the results are unconservative. For the *larger* of the two ratios, if the corresponding failure mode was not observed, then even if the ratio is greater than one, the conclusion is that the predicted strength was not exceeded. For all specimens, bolt strength controls the connection strength.

TABLE 5.1: CONNECTION STRENGTH COMPARISONS

Specimen ID	Design Ratios		Number of Ruptured Bolts [1]	Beam Inelastic Behavior
	$\frac{M_{PL}}{M_{max,fs}}$	$\frac{M_{NP}}{M_{max,fs}}$		
MRE1/2 - 0.875 - 1.0 - 24	1.59	1.05	6	Observed
4ES - 1.0 - 1.0 - 24	1.57	0.92	-	Observed
MRE1/3 - 0.75 - 0.75 - 30	1.51	1.42	-	Observed
MRE1/3S - 0.75 - 0.625 - 30	1.93	1.40	1	Observed
4E - 1.25 - 1.25 - 60 A	1.68	1.00	4	Observed
4E - 1.25 - 1.25 - 60 B	1.30	1.03	-	Observed
MRE1/2 - 1.0 - 1.0 - 60 A	1.42	0.93	4	Not observed
MRE1/2 - 1.0 - 1.0 - 60 B	1.16	0.94	6	Observed
MRE1/3S - 1.25 - 1.25 - 72 A	0.95	0.88	1	Not observed
MRE1/3S - 1.25 - 1.25 - 72 B	1.14	1.11	3	Not observed
MRE1/3 - 1.25 - 1.5 - 72	1.25	0.99	4	Observed

1. Includes bolts with stripped threads

5.2.2 End-Plate Deformation

Significant end-plate deformation was not observed during any of the tests. In cases where end-plate separation from the column flange was exhibited, the end-plates maintained their original geometry and did not appear to be bent or deformed significantly. The conclusion is that end-plate separation from the column flange was due mainly to bolt stretching and not end-plate bending. End-plates were selected that were thicker than the required values determined from the yield line solution (refer to Chapter 2). Also, for all connections, the bolt design ratio is less than the end-plate design ratio. Therefore the observed end-plate behavior was expected.

Because of the lack of end-plate deformation in the tests, it was not possible to determine the yield moment for the end-plates, M_y. Therefore, the predicted end-plate strength, M_{PL}, is compared to the maximum applied moment, M_u. This ratio provides a conservative analysis of the design procedure.

5.2.3 Weld Rupture

Specimen MRE1/2-1.0-1.0-60 A exhibited bolt rupture and rupture of the weld connecting the column flange to the column web. Due to fabrication misalignment, the column web was offset from the centerline of the column flange by approximately 3/8 in. It is likely that this eccentric placement of the column web induced additional stresses on the weld and caused weld failure when the bolts ruptured. Specimen MRE1/2-1.0-1.0-60 B was fabricated in a similar manner, however the column web was aligned with the center of the column flange. In addition, the "B" specimen was tested with A490 bolts, compared to A325 bolts for the "A" specimen. The welds connecting the column webs to the column flanges were identical for the two specimens. The "B" specimen was subjected to a moment 22 percent higher than the "A" specimen, yet the weld did not rupture on the "B" specimen. The conclusion is that the rupture on the "A" specimen was due to fabrication errors, not a faulty design procedure.

5.2.4 Bolt Tension Rupture

Bolt tension rupture occurred in eight of the eleven tests. Bolt thread stripping is considered tension rupture, because of the brittle nature of the failure. From Table 5.1 it is apparent that the strength of the connection bolts controlled the connection strength for all specimens. Two specimens, MRE1/3-0.75-0.75-30 and MRE1/3S-0.75-0.625-30, have bolt ratios significantly greater than unity. The nine remaining specimens have bolt ratios of 0.88 to 1.11, with an average value of 0.98. The bolt ratios for the specimens MRE1/3-0.75-0.75-30 and MRE1/3S-0.75-0.625-30 are high because the connection was designed for more than the beam moment strength (refer to Table 2.3). Although one bolt ruptured on the MRE1/3S-0.75-0.625-30 specimen, this occurred after the maximum moment had been achieved and during the inelastic load cycles. The conclusion is that the bolt rupture on this specimen was due to fatigue.

5.2.5 Summary

Although there is some variability in the predicted strength and maximum applied moment ratios, the design procedure correctly predicted the controlling limit state (bolt tension rupture) in every specimen where the connection failed. Furthermore, the design procedure predicted the strength of the controlling limit state with very good accuracy. Excluding the two specimens that were designed conservatively, the average controlling ratio (connection bolts) was 0.98 with a standard deviation of 5.9 percent.

5.3 BEAM STRENGTH

5.3.1 Experimental and Analytical Comparisons

Comparisons of the experimental and analytical results for beam strength are shown in Table 5.2. The moment strength design ratio was calculated by dividing the moment strength, M_b, by the maximum applied moment calculated at the connection faying surface, $M_{max,fs}$. The plastic strength design ratio was calculated by dividing the plastic strength, M_p, by the maximum applied moment at the connection faying surface, $M_{max,fs}$. The expected plastic strength design ratio was calculated by dividing the expected plastic strength, M_{pe}, by the maximum applied moment at the connection faying surface, $M_{max,fs}$. Measured material properties were used to calculate M_b, and M_p, and expected plastic strengths were calculated using the yield and tensile stresses shown in Table 2-1 of the LRFD Manual (AISC, 2001). Calculations of M_b, M_p, and

TABLE 5.2: BEAM STRENGTH COMPARISONS

Specimen ID	Design Ratios			Connection Failure
	$\frac{M_b}{M_{max,fs}}$	$\frac{M_p}{M_{max,fs}}$	$\frac{M_{pe}}{M_{max,fs}}$	
MRE1/2 - 0.875 - 1.0 - 24	0.97	0.97	1.05	Observed
4ES - 1.0 - 1.0 - 24	0.91	0.91	0.99	Not Observed
MRE1/3 - 0.75 - 0.75 - 30	1.10	1.28	1.38	Not Observed
MRE1/3S - 0.75 - 0.625 - 30	1.09	1.27	1.37	Observed
4E - 1.25 - 1.25 - 60 A	1.03	1.38	1.45	Observed
4E - 1.25 - 1.25 - 60 B	1.00	1.30	1.19	Not Observed
MRE1/2 - 1.0 - 1.0 - 60 A	1.21	1.58	1.44	Observed
MRE1/2 - 1.0 - 1.0 - 60 B	0.98	1.27	1.16	Observed
MRE1/3S - 1.25 - 1.25 - 72 A	0.79	0.92	1.09	Observed
MRE1/3S - 1.25 - 1.25 - 72 B	1.04	1.38	1.39	Observed
MRE1/3 - 1.25 - 1.5 - 72	0.93	1.23	1.24	Observed

M_{pe} are located in the Appendices. Also noted is whether or not the connection failed during the test. Design ratios less than or equal to unity are unconservative with respect to connection design since the predicted strength is lower than the maximum applied moment. Design ratios greater than unity are conservative with respect to connection design.

5.3.2 Interpretation of Results

For the two specimens having beams with compact webs, MRE1/2-0.875-1.0-24 and 4ES-1.0-1.0-24, the applied moment was nearly equal to the expected plastic beam strength (expected plastic strength ratios are 1.05 and 0.99, respectively). This confirms the use of the expected plastic strength in connection design, as required in the AISC Seismic Provisions.

For the two specimens with beams having non-compact webs, MRE1/3-0.75-0.75-30 and MRE1/3S-0.75-0.625-30, the applied moment was significantly less than the expected plastic moment, and approximately 10 percent less than the predicted beam moment strength (moment strength ratios are 1.10 and 1.09, respectively). It is possible that the column did not have sufficient weak axis or torsional stiffness to prevent the beam from buckling laterally near the connection. While the column on these two tests did not appear to translate or rotate out-of-plane appreciably (and not more than any other test), small movement would have had an adverse affect on these specimens more than other test specimens. The beam used in these tests had flanges with width-thickness ratios of 8, whereas the next largest ratio was 6.4 on the specimens with 60 in. beams. Therefore, the flanges used on the 30 in. beam might have been more susceptible to buckling if the beam could not be restrained by the column.

Seven specimens had beams with slender webs. The beam moment ratio for the MRE1/2-1.0-1.0-60 A specimen is 1.21. Yet because the predicted strength of the connection bolts on this specimen was less than the design beam strength (refer to Chapter 2), such a beam moment ratio is not unexpected. Note also that the beam exhibited negligible inelastic behavior on this test. The remaining six beam moment ratios are between 0.79 and 1.04, with an average value of 0.96 and a standard deviation of 9.3 percent. Noteworthy is that on the three tests utilizing 72 in. beams, little or no beam inelastic behavior was observed. Specimens 4E-1.25-1.25-60 A and MRE1/2-1.0-1.0-60 B exhibited beam inelastic behavior in one direction of load, while in the opposite direction both specimens exhibited bolt rupture. Specimen 4E-1.25-1.25-60 B was the only specimen that exhibited beam inelastic behavior in both directions, and its beam moment ratio is slightly unconservative (0.98). The conclusion is that if the connections were stronger, the beams with slender webs would have exhibited higher moment capacity.

5.4 MOMENT FRAME CLASSIFICATION

The moment versus rotation relationship of each connection was analyzed for compliance with the AISC Seismic Provisions criteria for connections in moment frames. Table 5.3 summarizes the preliminary classification data: the maximum applied moment calculated at the column centerline, M_{max}, the maximum interstory drift angle, θ_{max}, and the applied moment corresponding to θ_{max}, $M_{\theta max}$.

The Seismic Provisions require that the connection strength at the column face must equal at least 80 percent of the nominal plastic strength of the connected beam at the required Interstory Drift Angle. Because the connections in this study were designed for the nominal beam strength, the connections are classified on the basis of the interstory drift angle at which the connection strength was still greater than 80 percent of the maximum strength. Also, the connection must withstand one complete load cycle at the required interstory drift. Finally, the

TABLE 5.3: PRELIMINARY CLASSIFICATION DATA

Specimen ID	Mmax (ft*kips)	θ_{max} (rad)	$M_{\theta max}$ (ft*kips)	$\frac{M_{\theta max}}{M_{max}}$
MRE1/2 - 0.875 - 1.0 - 24	759.1	0.030	759.1	1.00
4ES - 1.0 - 1.0 - 24	805.4	0.050	722.3	0.90
MRE1/3 - 0.75 - 0.75 - 30	517.4	0.040	245.1	0.47
MRE1/3S - 0.75 - 0.625 - 30	521.5	0.040	243.6	0.47
4E - 1.25 - 1.25 - 60 A	2291.0	0.040	1183.7	0.52
4E - 1.25 - 1.25 - 60 B	2801.0	0.030	1342.9	0.48
MRE1/2 - 1.0 - 1.0 - 60 A	2310.6	0.014	2310.6	1.00
MRE1/2 - 1.0 - 1.0 - 60 B	2866.1	0.021	2174.7	0.76
MRE1/3S - 1.25 - 1.25 - 72 A	7553.5	0.014	7553.5	1.00
MRE1/3S - 1.25 - 1.25 - 72 B	5975.8	0.013	5975.8	1.00
MRE1/3 - 1.25 - 1.5 - 72	6688.1	0.016	6688.1	1.00

Seismic Provisions require that columns and beams incorporated into a Special Moment Frame must have elements that are seismically compact. The width-thickness ratio for webs must be less than the seismically compact ratio, λ_{ps}:

$$\lambda_{ps} = \frac{417}{\sqrt{F_y}} \tag{5.1}$$

None of the tested beams had web width-thickness ratios less than λ_{ps}. Consequently, the connections can be qualified only for Ordinary or Intermediate Moment Frames.

When the aforementioned requirements are applied to the test results, the connections can be classified appropriately. Table 5.4 summarizes the moment frames for which the tested connections are qualified. Included is the maximum applied moment calculated at the column centerline in one complete cycle, M_{cmax}, the last completed interstory drift angle for which the connection is qualified, θ_q, the applied moment corresponding to θ_q, $M_{\theta q}$, and the moment frame for which the connection is qualified.

Every configuration shown in Figure 1.4 meets the criteria for Intermediate Moment Frames in at least one specimen. Except for the 4E configuration on the 60 in. beam (which is qualified for use in an Intermediate Moment Frame), the specimens with beams having slender webs are qualified for use in Ordinary Moment Frames. The 4ES-1.0-1.0-24 connection meets all Special Moment Frame requirements except for the limiting width-thickness ratio for webs, and therefore is qualified only for an Intermediate Moment Frame. Four specimens achieved their largest moment resistance, M_{max}, on a Load Step where they did not complete one full cycle. For these connections, M_{cmax} is less than M_{max}.

TABLE 5.4: CONNECTION QUALIFICATION

Specimen ID	M_{cmax} (ft*kips)	θ_q (rad)	$M_{\theta q}$ (ft*kips)	$\frac{M_{\theta q}}{M_{cmax}}$	Moment Frame for which Connection is Qualified
MRE1/2 - 0.875 - 1.0 - 24	759.1	0.030	759.1	1.00	Intermediate
4ES - 1.0 - 1.0 - 24	805.4	0.040	708.7	0.88	Intermediate
MRE1/3 - 0.75 - 0.75 - 30	517.4	0.020	412.7	0.80	Intermediate
MRE1/3S - 0.75 - 0.625 - 30	521.5	0.020	477.2	0.92	Intermediate
4E - 1.25 - 1.25 - 60 A	2291.0	0.020	1952.2	0.85	Intermediate
4E - 1.25 - 1.25 - 60 B	2801.0	0.020	2465.5	0.88	Intermediate
MRE1/2 - 1.0 - 1.0 - 60 A	1958.0	0.010	1958.0	1.00	Ordinary
MRE1/2 - 1.0 - 1.0 - 60 B	2866.1	0.015	2866.1	1.00	Ordinary
MRE1/3S - 1.25 - 1.25 - 72 A	5534.7	0.010	5534.7	1.00	Ordinary
MRE1/3S - 1.25 - 1.25 - 72 B	5307.4	0.010	5307.4	1.00	Ordinary
MRE1/3 - 1.25 - 1.5 - 72	6641.6	0.015	6641.6	1.00	Ordinary

CHAPTER 6 - DISCUSSION

6.1 SUMMARY

The research objectives were to determine the suitability of the thick plate design procedure in AISC Design Guide 16 for extended end-plate moment connections subject to cyclic loading, to determine the appropriate beam strength for which to design the connections, and to generate a data set with which to qualify certain extended end-plate moment connections. To meet these objectives, eleven beam-to-column connections were tested according to the protocol defined in the AISC Seismic Provisions. Figure 1.4 shows the tested configurations.

6.2 DESIGN RECOMMENDATIONS

6.2.1 Connection Design

The experimental results indicate that the thick plate procedure correctly predicts the failure mode of the tested extended end-plate moment connections, and predicts the nominal strength of the connections with very good accuracy. Also, every configuration shown in Figure 1.4 satisfies the criteria for Intermediate Moment Frames for at least one specimen. Therefore, the thick plate procedure in Design Guide 16 is considered suitable for designing these connections to resist cyclic loading.

In cases where the connection failed prior to the development of significant inelastic behavior in the connecting beam, typically there was insufficient rotational capability within the connection elements (bolts and end-plate) to qualify the specimens for use in anything other than Ordinary Moment Frames. Therefore, it is recommended that extended end-plate moment connections be designed stronger than the connecting beam. Table 6.1 shows that very little inelastic behavior, 0.006 rad of inelastic rotation or less, was developed within the connections before bolt tension rupture occurred.

TABLE 6.1: SOURCES OF INELASTIC BEHAVIOR

Specimen ID	Moment Frame for which Connection is Qualified	Primary Failure Mode	Maximum Total Inelastic Rotation (rad) [1]	Maximum Beam Inelastic Rotation (rad) [1]	Maximum Connection Inelastic Rotation (rad) [1]
MRE1/2 - 0.875 - 1.0 - 24	Intermediate	Bolt Rupture	0.012	0.006	0.006
4ES - 1.0 - 1.0 - 24	Intermediate	Beam Yielding	0.033	0.028	0.005
MRE1/3 - 0.75 - 0.75 - 30	Intermediate	Beam Yielding	0.033	0.029	0.004
MRE1/3S - 0.75 - 0.625 - 30	Intermediate	Beam Yielding	0.033	0.029	0.004
4E - 1.25 - 1.25 - 60 A	Intermediate	Bolt Rupture	0.010	0.007	0.003
4E - 1.25 - 1.25 - 60 B	Intermediate	Beam Yielding	0.023	0.020	0.003
MRE1/2 - 1.0 - 1.0 - 60 A	Ordinary	Bolt Rupture	0.002	0.001	0.001
MRE1/2 - 1.0 - 1.0 - 60 B	Ordinary	Bolt Rupture	0.009	0.006	0.003
MRE1/3S - 1.25 - 1.25 - 72 A	Ordinary	Bolt Rupture	0.006	0.003	0.003
MRE1/3S - 1.25 - 1.25 - 72 B	Ordinary	Bolt Rupture	0.003	0.001	0.002
MRE1/3 - 1.25 - 1.5 - 72	Ordinary	Bolt Rupture	0.008	0.002	0.006

1. If bolt rupture occurred, values are calculated prior to bolt rupture.

6.2.2 Design Beam Strength

The experimental results indicate that extended end-plate moment connections should be designed for a moment greater than the nominal beam strength, irrespective of the width-thickness ratio of the beam web. Six of the nine specimens with beams having either non-compact or slender webs failed primarily due to bolt tension rupture. Of those six specimens, only two beams exhibited significant inelastic behavior in the connecting beam; furthermore, such behavior occurred in only one direction of loading. The conclusion is that if the connections had been stronger, moments larger than the beam moment strength, M_b, would have been required to cause hinging.

As summarized by Salmon and Johnson (1996), no inelastic behavior in plate girders (i.e., beams with slender webs) is considered possible for design purposes. However, the results from the eleven tests conducted during this study suggest that at locations near extended end-plate moment connections, inelastic behavior is likely to occur. Furthermore, because a strong column, strong connection, weak beam philosophy is the required design approach, if such behavior will occur, it must be considered in seismic design.

The magnitude of the design moment cannot be determined precisely from the experimental results, because the strength of the beams was not reached on many of the tests; the connections failed on a majority of the specimens before hinges developed. However, the strength calculated using Equation 2.1 is an upper bound on the design moment for two reasons. First, good results were obtained with tests MRE1/2-0.875-1.0-24 and 4ES-1.0-1.0-24, which utilized a beam with a compact web. Also, as shown in Table 5.2, the predicted plastic strength ratios are 0.99 or greater for all tests, with an average of 1.25 and a standard deviation of 16.3 percent, indicating that the predicted plastic strength was nearly equal to or greater than the maximum applied moment at the faying surface. Because insufficient data is available to accurately predict the strength of beams having non-compact or slender webs, Equation 2.1 is recommended for determining beam strength with which to design the connections.

Figure 6.1 illustrates a suggested model of beam bending strength in the vicinity of an extended end-plate moment connection. First, it is assumed that the stiffening effect of the end-plate is adequate to develop the expected plastic strength, M_{pe}, of the beam at a distance d/2 from the end-plate. Second, it is assumed that the strength decreases linearly to the nominal strength, M_n, a distance d/2 away from the plastic hinge.

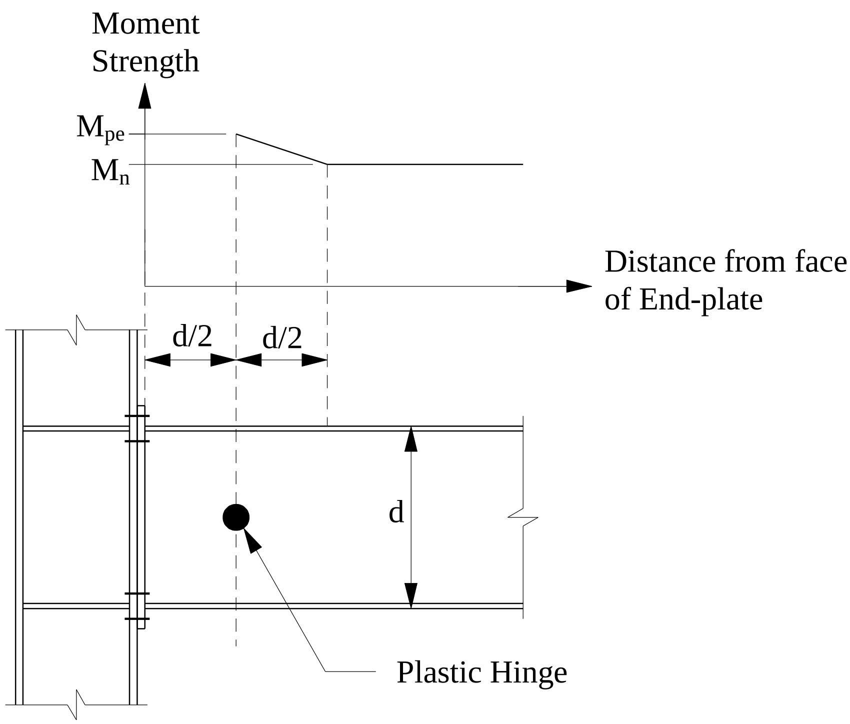


FIGURE 6.1: SUGGESTED BEAM STRENGTH MODEL IN VICINITY OF EXTENDED END-PLATE

Table 6.2 shows the ratios of M_{pe} to M_n for the tested beams. Nominal yield stress plus 5 ksi was used to calculate M_n, and the yield and tensile stresses shown in Table 2-1 of the LRFD Manual (AISC 2001) were used to calculate M_{pe}. Also shown is the width-thickness ratio for the beam web. The ratio of M_{pe} to M_n increases as the web width-thickness ratio increases.

In general, the experimental results correlate reasonably well with the suggested model. As the width-thickness ratios of the tested beams increase, the experimental results tend toward failure due to bolt tension rupture and little to no development of beam inelastic behavior. Additional experimental testing is required to determine the validity of the suggested model. If accurate, the model indicates that the design moment for connection strength is considerably larger, nearly 50 percent in the cases of specimens utilizing the 60 in. beam, than the nominal strength of the beam.

TABLE 6.2: EXPECTED PLASTIC TO NOMINAL STRENGTH RATIOS

Specimen ID	$\frac{h}{t_w}$	Expected Plastic Strength Mpe (k*ft)	Nominal Strength Mn (k*ft)	$\frac{M_{pe}}{M_n}$
MRE1/2 - 0.875 - 1.0 - 24	60.7	763	691	1.10
4ES - 1.0 - 1.0 - 24	60.7	763	691	1.10
MRE1/3 - 0.75 - 0.75 - 30	117.0	680	522	1.30
MRE1/3S - 0.75 - 0.625 - 30	117.0	680	522	1.30
4E - 1.25 - 1.25 - 60 A	156.7	3161	2210	1.43
4E - 1.25 - 1.25 - 60 B	156.7	3161	2210	1.43
MRE1/2 - 1.0 - 1.0 - 60 A	156.7	3161	2210	1.43
MRE1/2 - 1.0 - 1.0 - 60 B	156.7	3161	2210	1.43
MRE1/3S - 1.25 - 1.25 - 72 A	140.0	7865	6038	1.30
MRE1/3S - 1.25 - 1.25 - 72 B	140.0	7865	6038	1.30
MRE1/3 - 1.25 - 1.5 - 72	140.0	7865	6038	1.30

6.3 FUTURE RESEARCH NEEDS

Additional experimental testing is required to verify the accuracy of the suggested model shown in Figure 6.1. The design connection strength for future tests should be based on the expected plastic strength of the beam; the suggested model should be modified as necessary based on future test results.

Additional experimental testing is required to establish a data set for qualifying other configurations in Design Guide 16 that were not tested during this study.

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APPENDIX A

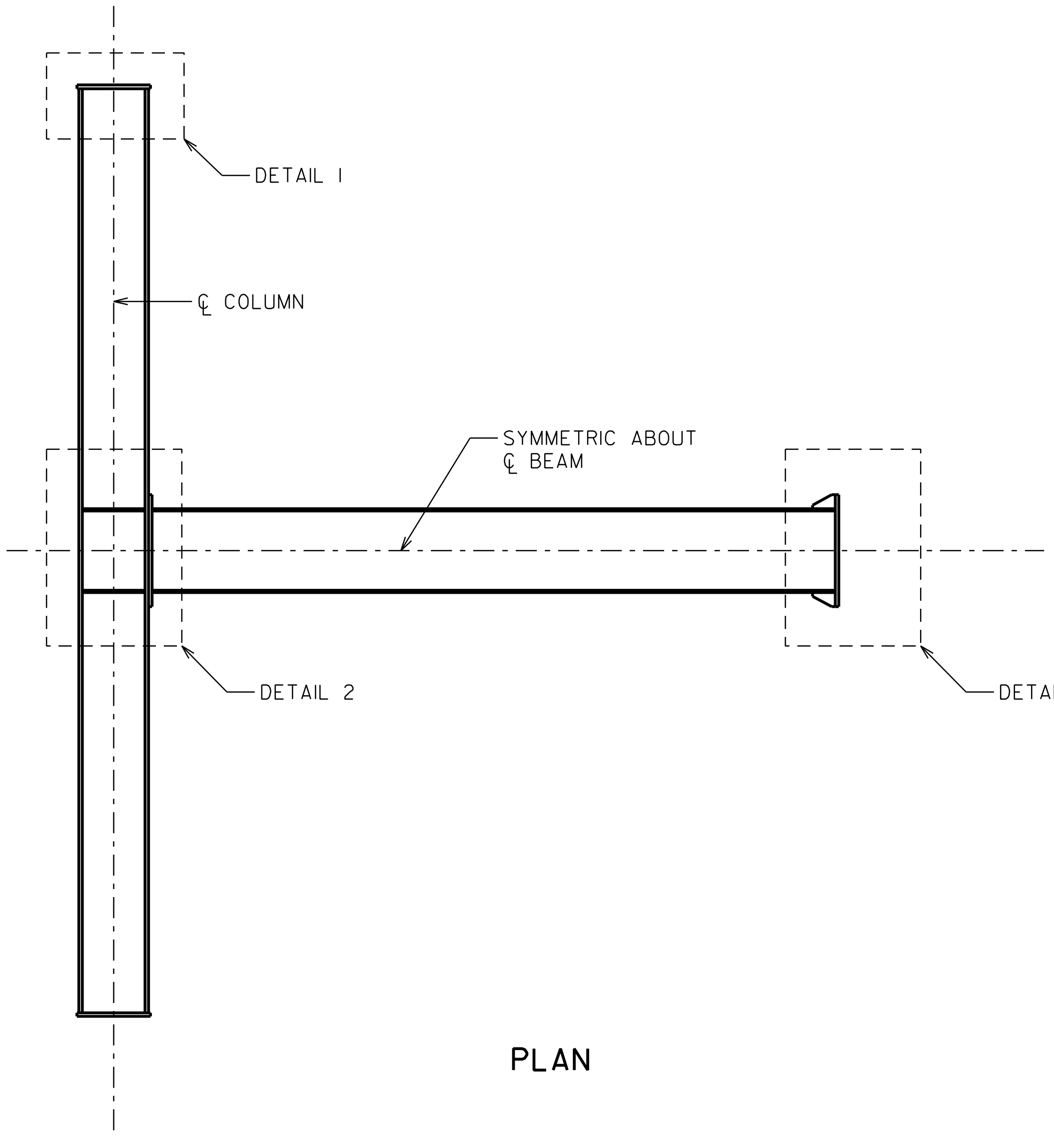
FABRICATION DRAWINGS

Fabrication Drawings

for

Test MRE1/2-0.875-1.0-24

Test 4ES-1.0-1.0-24



PLAN

BEAM

WEB = 3/8×22 3/4
FLANGE = 5/8×7
LENGTH = 197 1/2 (INCLUDES ENDPLATES)

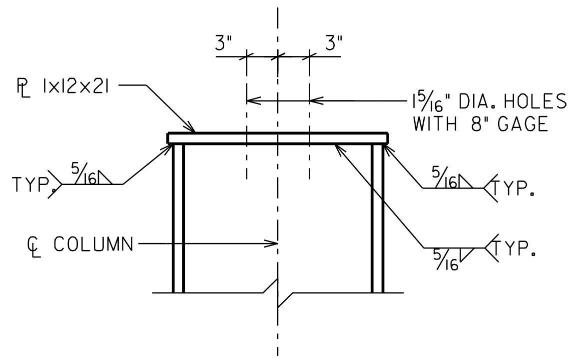
COLUMN

WEB = 3/8×18 1/8
PANEL ZONE WEB = 5/8×18 1/8
FLANGES = 1x8
LENGTH = 266 1/2 (INCLUDES ENDPLATES)

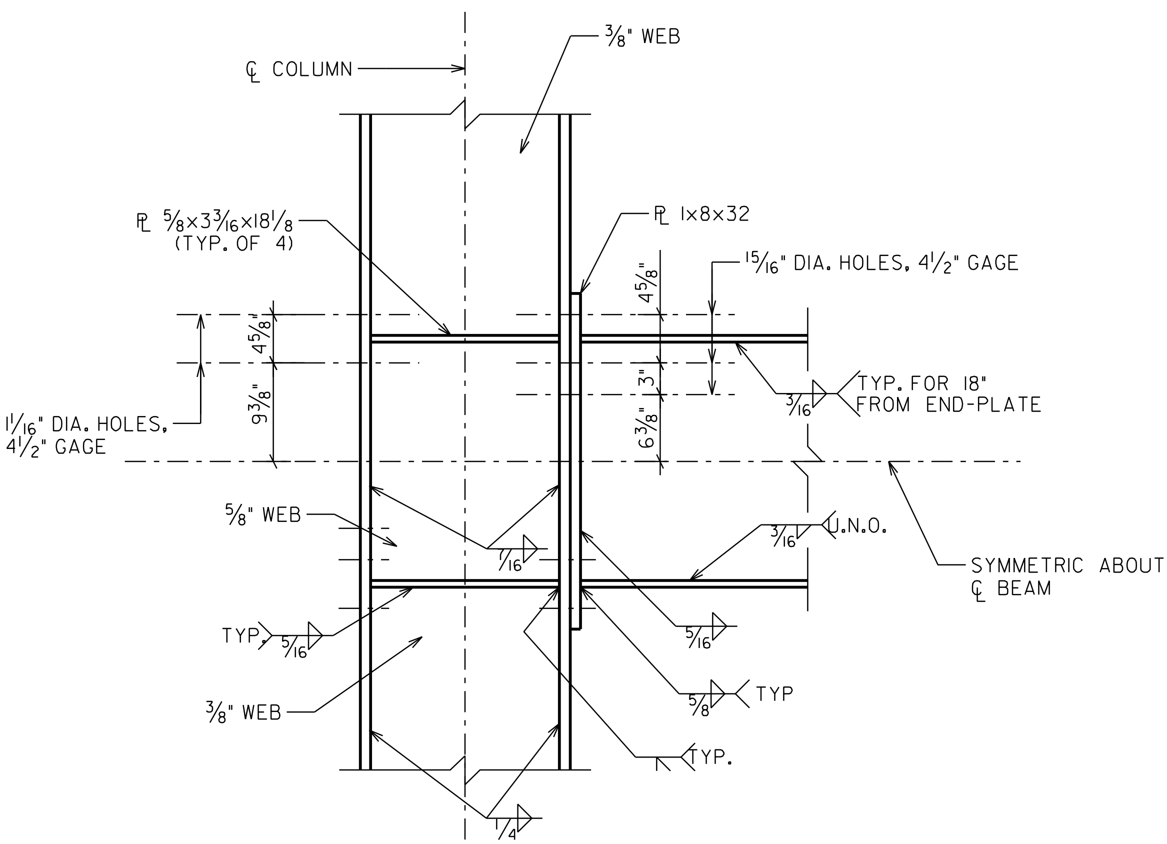
NOTES

1. ALL STEEL, ASTM A572, GRADE 50.
2. ALL BOLTS, A490.
3. ALL WELDING, AWS D1.1, E70XX ELECTRODES.

SPECIMENS: MRE1/2-0.875-1.0-24 & 4ES-1.0-1.0-24	DRAWN BY: SEB

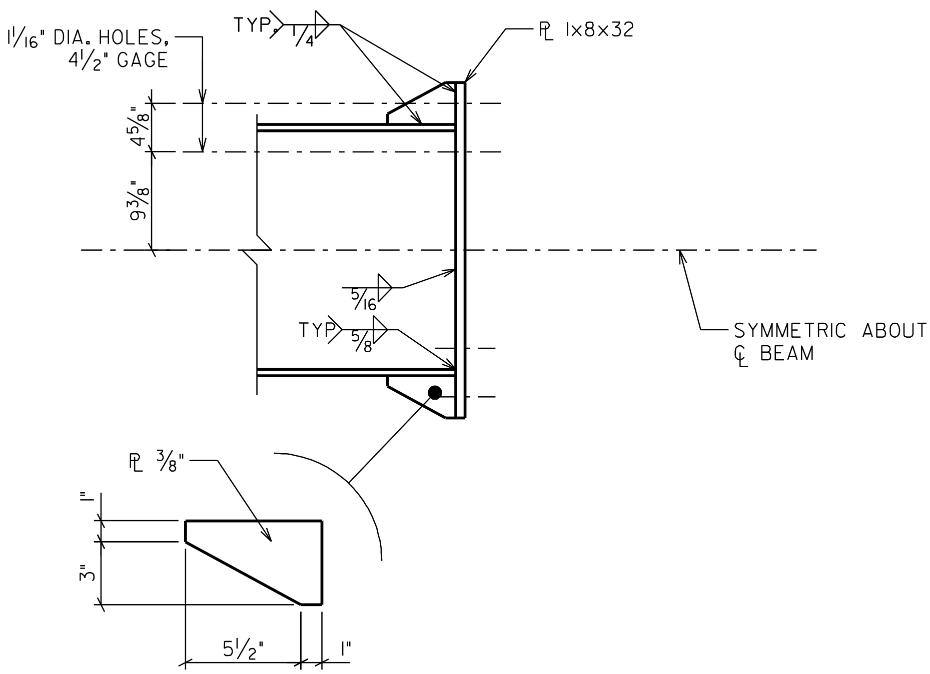


DETAIL 1



DETAIL 2

SPECIMENS: MREI/2-0.875-1.0-24 & 4ES-1.0-1.0-24	DRAWN BY: SEB



DETAIL 3

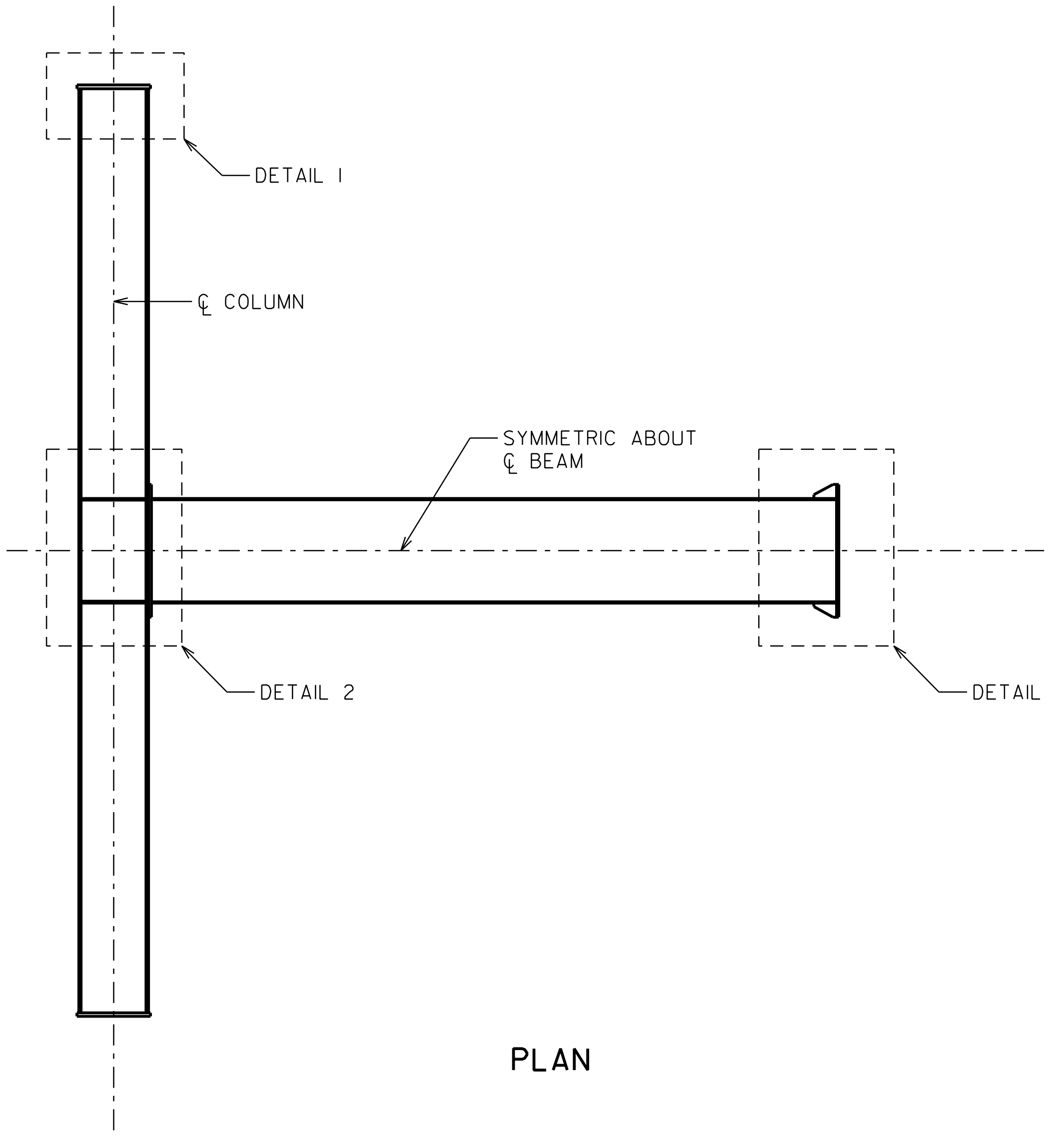
SPECIMENS: MREI/2-0.875-1.0-24 & 4ES-1.0-1.0-24 | DRAWN BY: SEB

Fabrication Drawings

for

Test MRE1/3-0.75-0.75-30

Test MRE1/3S-0.75-0.625-30



PLAN

BEAM

WEB = 1/4×29 1/4
FLANGE = 3/8×6
LENGTH = 197 1/2 (INCLUDES ENDPLATES)

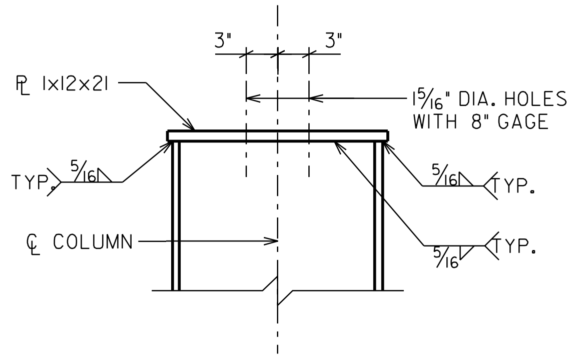
COLUMN

WEB = 1/2×18 5/8
FLANGES = 5/8×8 & 3/4×8
LENGTH = 266 1/2 (INCLUDES ENDPLATES)

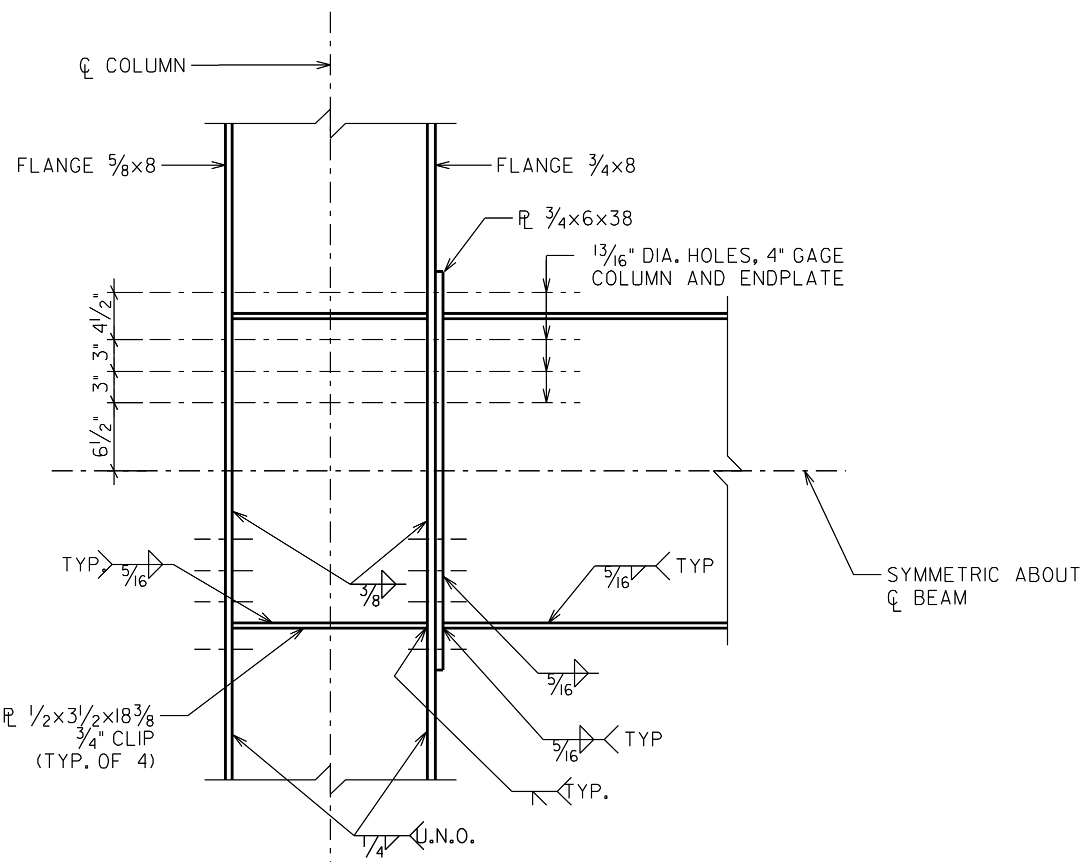
NOTES

1. ALL STEEL, ASTM A572, Grade 55.
2. ALL BOLTS, A325.
3. ALL WELDING, AWS D1.1, E70XX ELECTRODES.

SPECIMENS: MREI/3-0.75-0.75-30 & MREI/3S-0.75-0.625-30	DRAWN BY: SEB

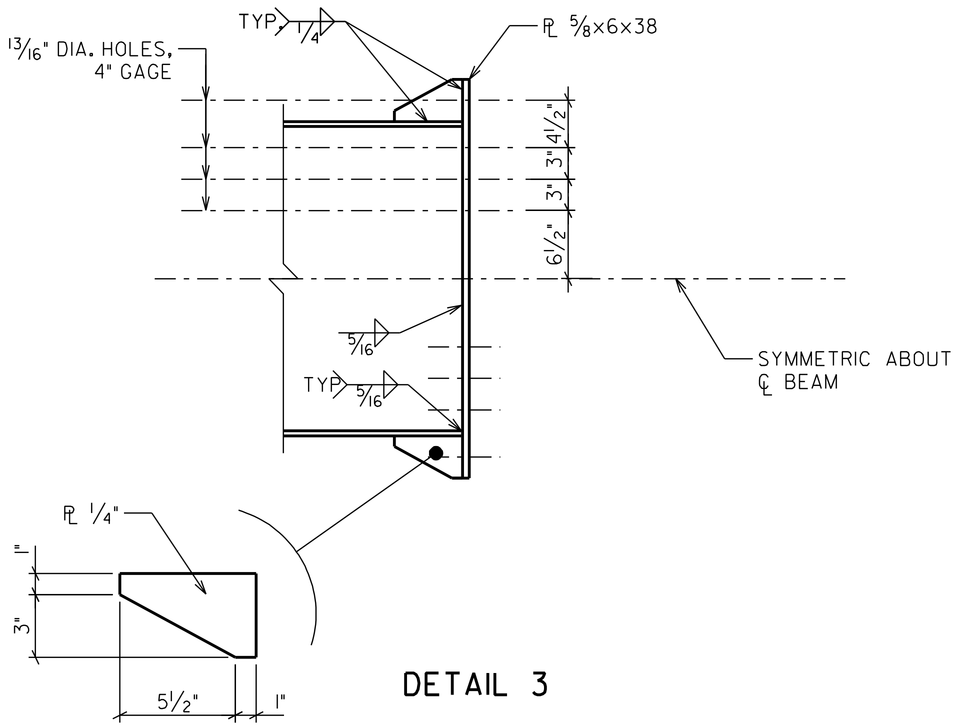


DETAIL 1



DETAIL 2

SPECIMENS: MREI/3-0.75-0.75-30 & MREI/3S-0.75-0.625-30	DRAWN BY: SEB



DETAIL 3

SPECIMENS: MREI/3-0.75-0.75-30 & MREI/3S-0.75-0.625-30	DRAWN BY: SEB

Fabrication Drawings

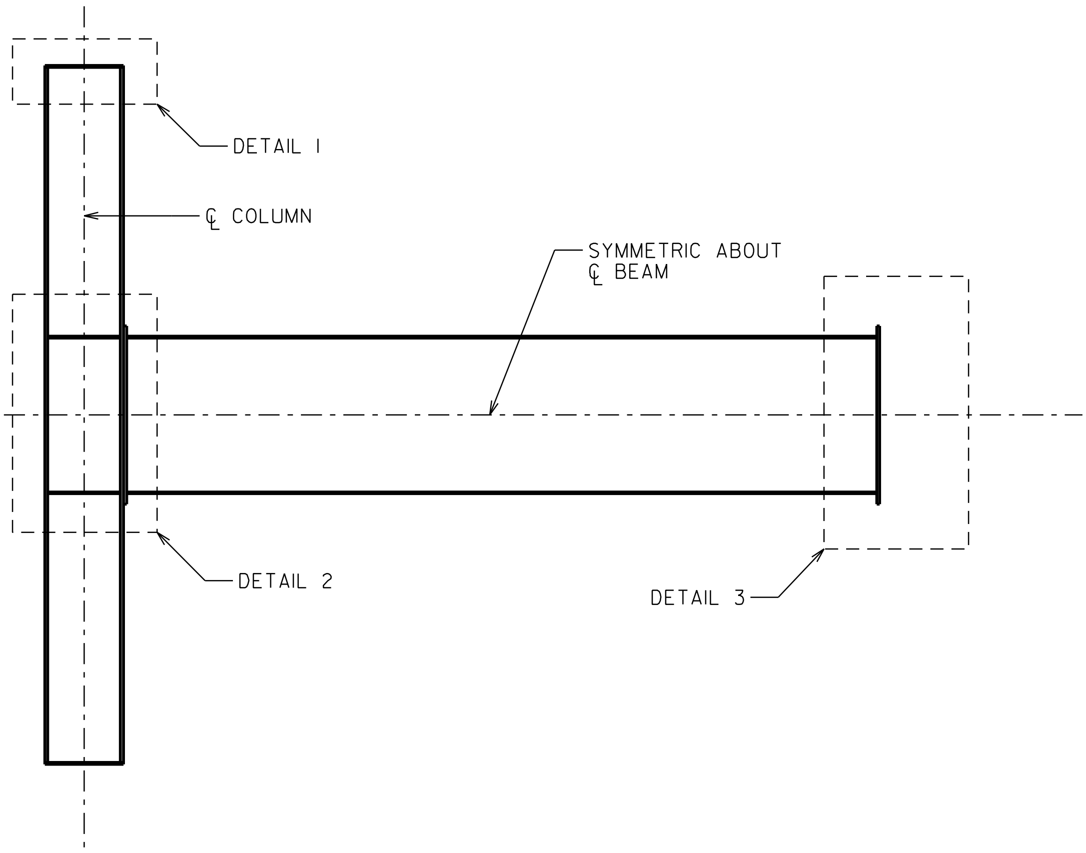
for

Test 4E-1.25-1.25-60 A

Test 4E-1.25-1.25-60 B

Test MRE1/2-1.0-1.0-60 A

Test MRE1/2-1.0-1.0-60 B



PLAN

BEAM

WEB = $\frac{3}{8}\times58\frac{3}{4}$
FLANGE = $\frac{5}{8}\times8$
LENGTH = $288\frac{1}{2}$ (INCLUDES ENDPLATES)

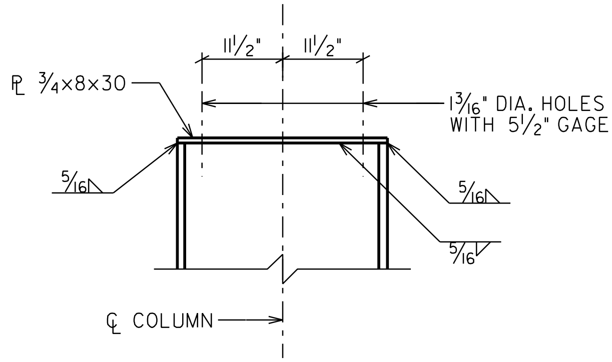
COLUMN

WEB = $\frac{1}{2}\times27\frac{3}{4}$
FLANGES = $1\frac{1}{4}\times8$ & 1×8
LENGTH = $266\frac{1}{2}$ (INCLUDES ENDPLATES)

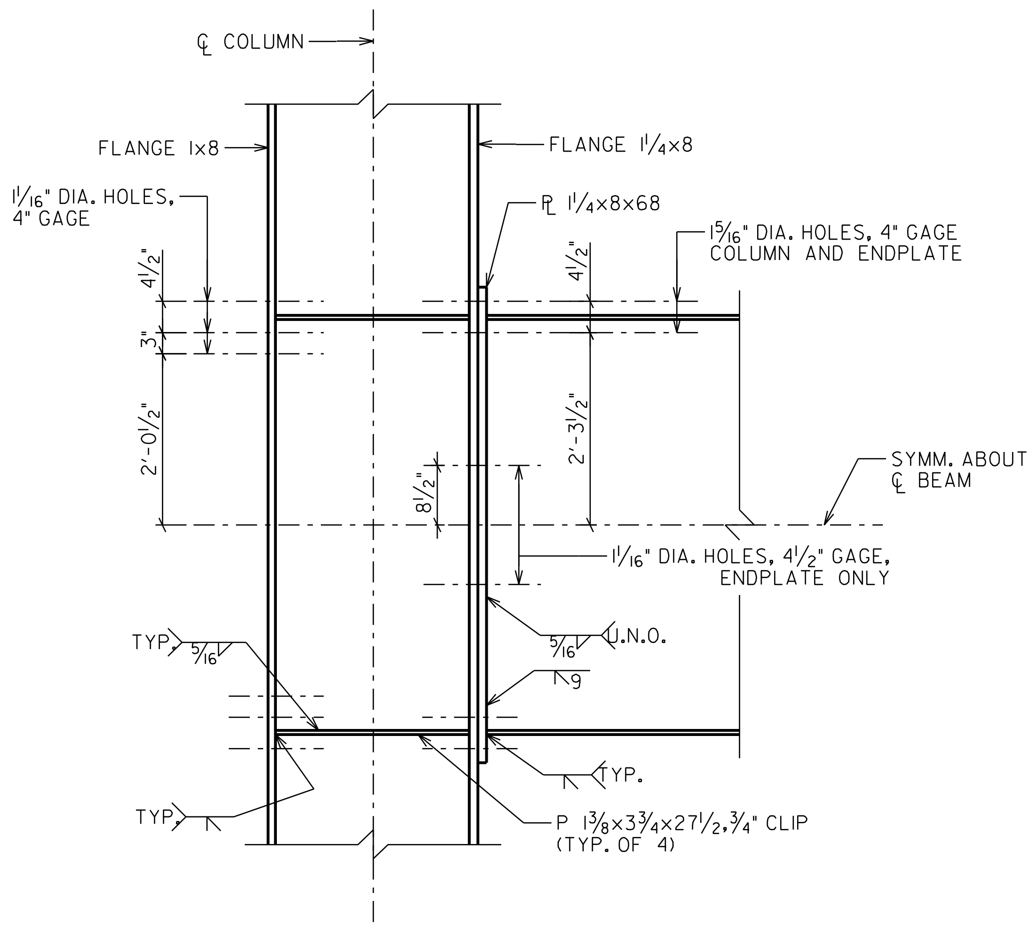
NOTES

1. ALL STEEL, ASTM A572 GRADE 50.
2. SPECIMEN A BOLTS, A325. SPECIMEN B BOLTS, A490.
3. ALL WELDING, AWS D1.1, E70XX ELECTRODES.

SPECIMENS: 4E-1.25-1.25-60 (A & B) & MRE1/2-1.0-1.0-60 (A & B)	DRAWN BY: SEB

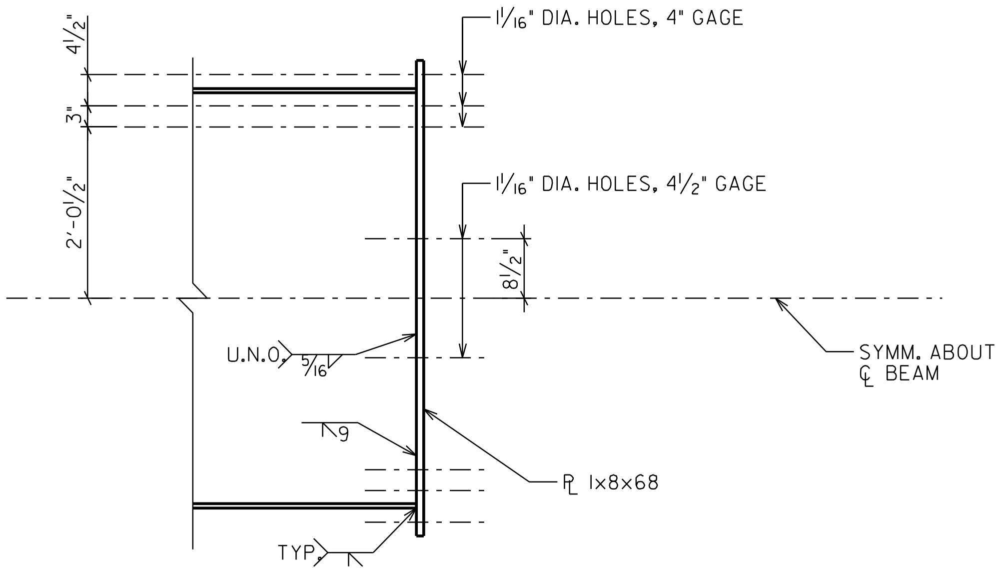


DETAIL 1



DETAIL 2

SPECIMENS: 4E-1.25-1.25-60 (A & B) & MRE1/2-1.0-1.0-60 (A & B)	DRAWN BY: SEB



DETAIL 3

SPECIMENS: 4E-1.25-1.25-60 (A & B) & MRE1/2-1.0-1.0-60 (A & B) | DRAWN BY: SEB

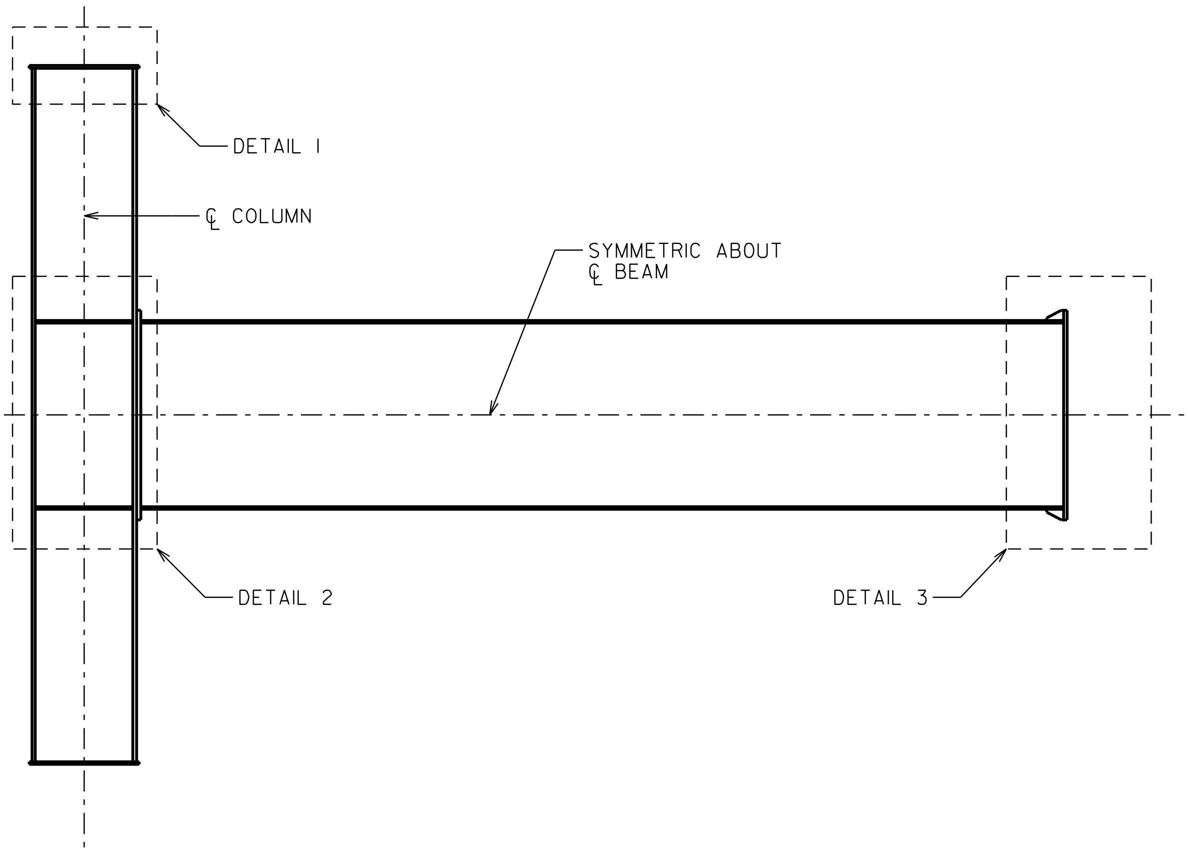
Fabrication Drawings

for

Test MRE1/3S-1.25-1.25-72 A

Test MRE1/3S-1.25-1.25-72 B

Test MRE1/3-1.25-1.5-72



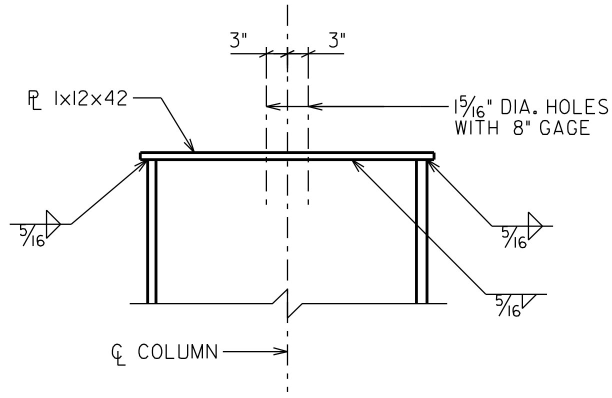
PLAN

BEAM
WEB = $\frac{1}{2}\times70$
FLANGE = 1x12
LENGTH = 355 (INCLUDES ENDPLATES)

COLUMN
WEB = $\frac{1}{2}\times37$ 1/4
PANEL ZONE WEB = $\frac{3}{4}\times37$ 1/4
FLANGES = $1\frac{1}{4}\times12$ & $1\frac{1}{2}\times12$
LENGTH = $266\frac{1}{2}$ (INCLUDES ENDPLATES)

NOTES
1. ALL STEEL, ASTM A572. COLUMN AND BEAM WEBS SHALL BE GRADE 50, ALL ELSE SHALL BE GRADE 55.
2. ALL BOLTS, A490.
3. ALL WELDING, AWS D1.1, E70XX ELECTRODES.

SPECIMENS: MREI/3S-1.25-1.25-72 (A & B) & MREI/3-1.25-1.5-72	DRAWN BY: SEB



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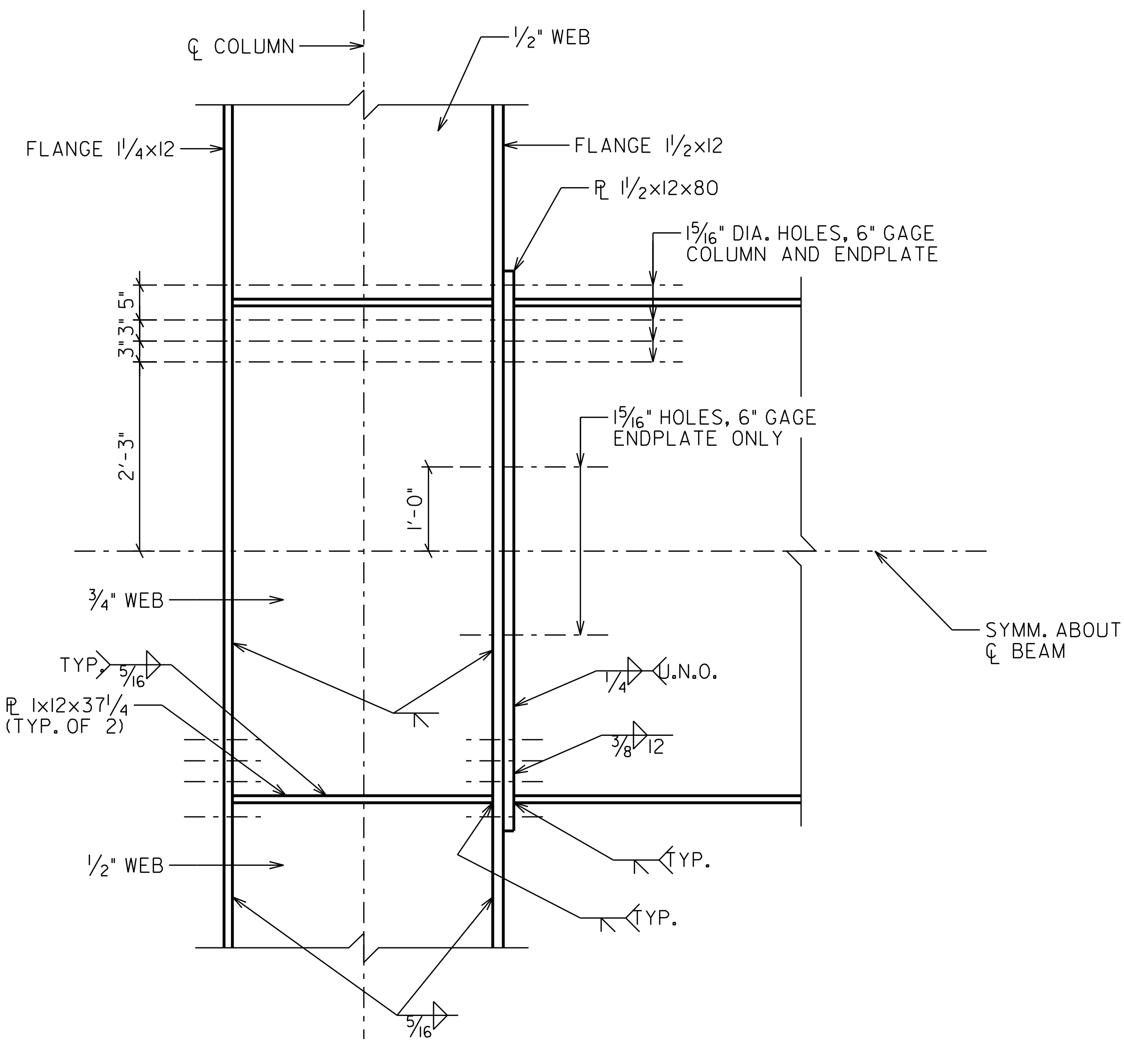
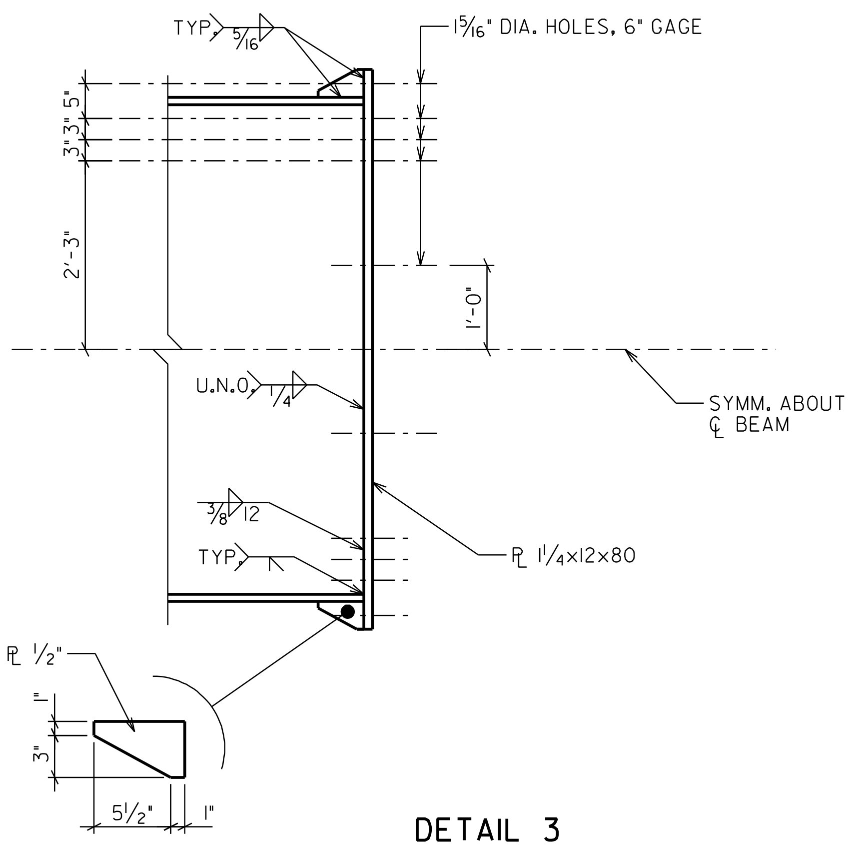


DETAIL 2

SPECIMENS: MREI/3S-1.25-1.25-72 (A & B) & MREI/3-1.25-1.5-72 | DRAWN BY: SEB



DETAIL 3

SPECIMENS: MREI/3S-1.25-1.25-72 (A & B) & MREI/3-1.25-1.5-72	DRAWN BY: SEB

APPENDIX B

MRE 1/2 – 0.875 – 1.0 – 24

RESULTS AND TEST DATA

SPECIMEN PROPERTIES & TEST SUMMARY

TEST NAME: MRE1/2-0.875-1.0-24
TEST DATE: October 24, 2003

BEAM DATA

DEPTH, d:	24.00	in
FLANGE WIDTH, b_f:	6.94	in
FLANGE THICKNESS, t_f:	0.63	in
WEB THICKNESS, t_w:	0.38	in
FLANGE YIELD STRESS, F_{yf}:	56.0	ksi
FLANGE ULTIMATE STRESS, F_{uf}:	78.2	ksi
WEB YIELD STRESS, F_{yw}:	54.9	ksi
WEB ULTIMATE STRESS, F_{uw}:	69.1	ksi

END-PLATE DATA

END-PLATE THICKNESS, t_p:	1.00	in
END-PLATE WIDTH, b_p:	8.00	in
END-PLATE LENGTH, L_p:	32.00	in
END-PLATE EXTENSION OUTSIDE FLANGE, p_{ext}:	4.00	in
OUTER PITCH, BOLT TO FLANGE, p_{fo}:	2.07	in
INNER PITCH, BOLT TO FLANGE, p_{fi}:	1.96	in
PITCH, BOLT TO BOLT, p_b:	3.00	in
GAGE, g:	4.47	in
END-PLATE YIELD STRESS, F_{yp}:	63.3	ksi
END-PLATE ULTIMATE STRESS, F_{up}:	85.1	ksi

BOLT DATA

BOLT DIAMETER, d_b:	0.875	in
BOLT LENGTH, L_b:	3.25	in
BOLT TYPE:	A490	
NOMINAL BOLT TENSILE STRESS (AISC J3.6), F_t:	113	ksi
NOMINAL BOLT TENSILE STRENGTH, P_t:	70	kips
BOLT PRETENSION, T_b:	49	kips

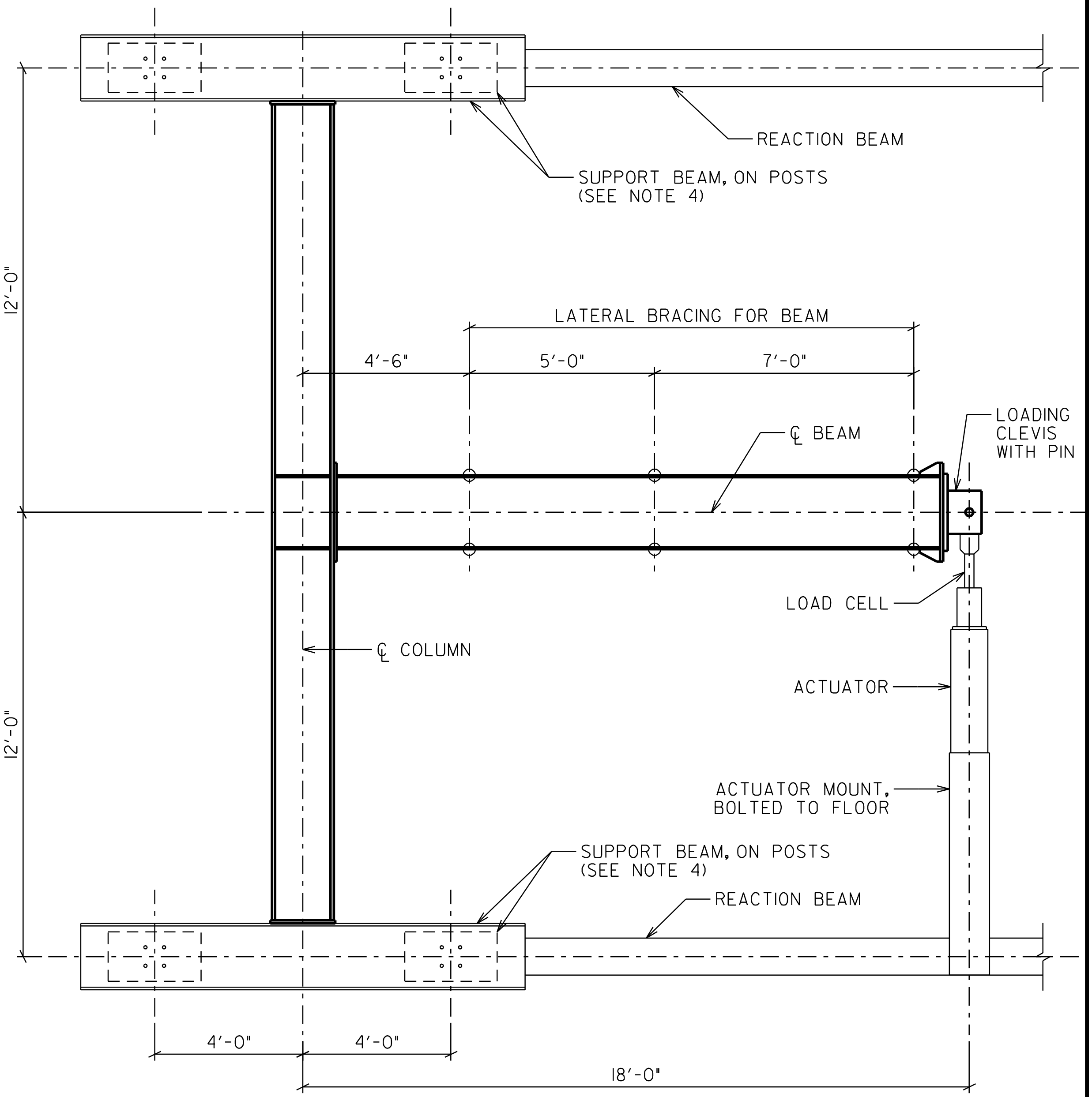
EXPERIMENTAL RESULTS

FAILURE MODE:	Bolt Tension Rupture	
MAXIMUM MOMENT AT COLUMN CENTERLINE, $M_{max,cc}$:	759.1	ft-kips
MAXIMUM MOMENT AT FAYING SURFACE, $M_{max,fs}$:	724.0	ft-kips
MAXIMUM INTERSTORY DRIFT ANGLE, θ_{max}:	0.030	rad
MAXIMUM INTERSTORY DRIFT ANGLE AT 80% 0F $M_{max,cc}$, θ_{preq}:	0.030	rad

CALCULATED STRENGTHS

BEAM MOMENT STRENGTH [1], M_b:	701.6	ft-kips
BEAM PLASTIC STRENGTH [1], M_p:	701.6	ft-kips
BEAM EXPECTED PLASTIC STRENGTH, M_{pe}:	762.8	ft-kips
END-PLATE STRENGTH [1], M_{PL}:	1002.6	ft-kips
BOLT TENSION RUPTURE (w/o Prying, using F_t), M_{NP}:	757.7	ft-kips

1. Measured material properties used.



MREI/2-0.875-1.0-24 TEST SUBASSEMBLAGE PLAN

BEAM

WEB = 3/8×22 3/4
FLANGE = 5/8×7
LENGTH = 197 1/2 (INCLUDES ENDPLATES)

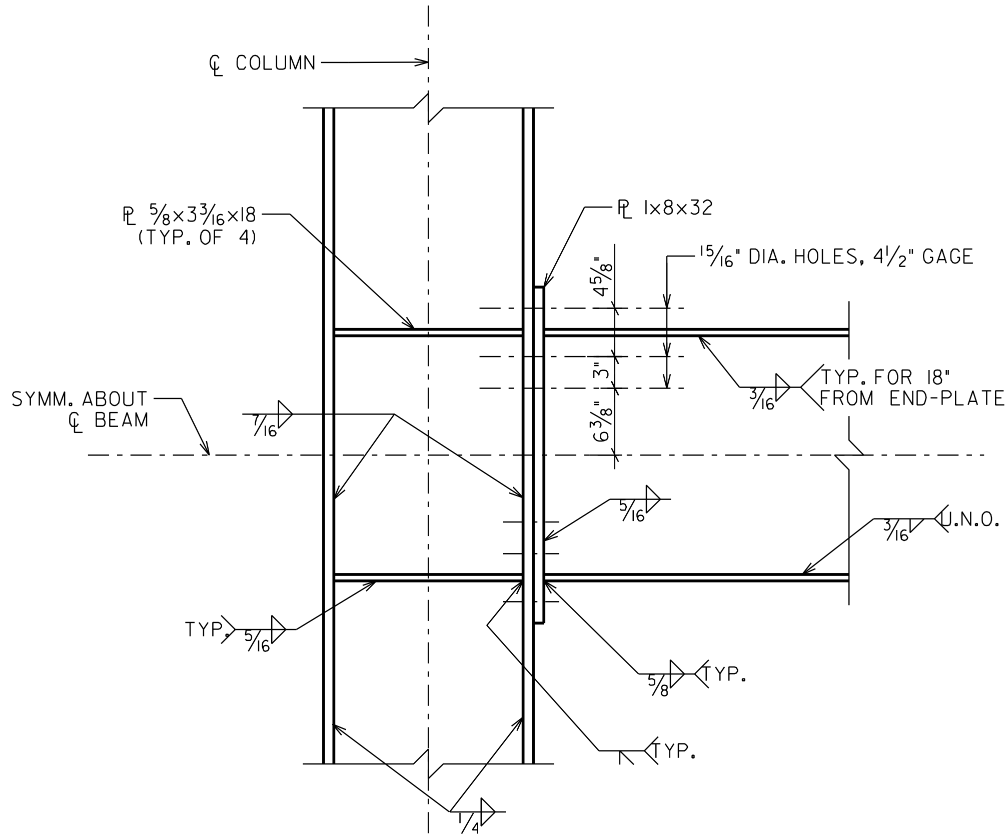
COLUMN

WEB = 3/8×18 1/8
PANEL ZONE WEB = 5/8×18 1/8
FLANGE = 1×8
LENGTH = 266 1/2 (INCLUDES ENDPLATES)

NOTES

1. COLUMN, BEAM, AND CONNECTION STEEL, A572 Gr50.
2. ALL BOLTS, A490.
3. ALL WELDING, AWS D1.1, E70XX ELECTRODES.
4. COLUMN BOLTED TO SUPPORT BEAM. SUPPORT BEAM BOLTED POSTS. POSTS BOLTED TO REACTION BEAM.

SPECIMEN: MREI/2-0.875-1.0-24	DRAWN BY: SEB



MREI/2-0.875-1.0-24
CONNECTION DETAIL

SPECIMEN: MREI/2-0.875-1.0-24	DRAWN BY: SEB

SAC PROTOCOL LOADING HISTORY

Test Name: MRE1/2-0.875-1.0-24 Test By: SEB Date: 10/24/2003

Load Step	Cycle	Max. Pos. Load (kips)	Max. Disp. (in)	Max. Neg. Load (kips)	Max. Disp. (in)	Comments
I (0.00375 rad)	1	9.1	0.803	-9.6	-0.805	
δ=0.81"	2	9.7	0.867	-9.8	-0.817	
	3	9.2	0.814	-9.8	-0.815	
	4	9.2	0.817	-9.6	-0.814	
	5	9.2	0.819	-9.6	-0.805	
	6	9.4	0.829	-9.7	-0.821	
		Permanent Set =	-0.002"			
II (0.005 rad)	1	12.1	1.079	-12.6	-1.083	
δ=1.08"	2	12.3	1.086	-12.8	-1.093	
	3	12.1	1.078	-12.6	-1.077	
	4	12.3	1.084	-12.6	-1.079	
	5	12.3	1.087	-12.7	-1.092	
	6	12.4	1.099	-12.6	-1.077	
		Permanent Set =	-0.005"			
III (0.0075 rad)	1	18.3	1.657	-18.8	-1.642	
δ=1.62"	2	18.5	1.663	-19.0	-1.651	
	3	18.2	1.640	-19.6	-1.713	
	4	18.1	1.639	-19.1	-1.643	
	5	18.1	1.632	-19.1	-1.637	
	6	18.3	1.626	-19.3	-1.662	
		Permanent Set =	-0.040"			
IV (0.01 rad)	1	23.4	2.169	-24.5	-2.163	
δ=2.16"	2	23.6	2.155	-24.4	-2.154	
	3	23.5	2.160	-24.6	-2.161	
	4	23.6	2.159	-24.4	-2.153	
		Permanent Set =	-0.041"			
V (0.015 rad)	1	32.8	3.249	-34.5	-3.237	Slight Comp. Flange Flaking
δ=3.24"	2	33.2	3.245	-34.6	-3.243	
		Permanent Set =	-0.121"			
VI (0.02 rad)	1	37.7	4.326	-39.3	-4.313	Extensive Flange Flaking
δ=4.32"	2	38.1	4.334	-39.3	-4.319	
		Permanent Set =	-0.525"			
VII (0.03 rad)	1	40.7	6.482	-42.3	-6.486	Slight Comp. Flange Buckling
δ=6.48"	2	40.9	6.487	~ -42	~ -6	Bolt Rupture
		Permanent Set =				
VIII (0.04 rad)	1					
	2					
		Permanent Set =				
IX (0.05 rad)	1					
	2					
		Permanent Set =				
X (0.06 rad)	1					
	2					
		Permanent Set =				

CALCULATED STRENGTHS

TEST NAME: MRE1/2-0.875-1.0-24
TEST DATE: October 24, 2003

Specified Grade for all Steel = ASTM A572, Grade 50

MEASURED PROPERTIES

$d := 24 \cdot in$ $F_{yf} := 56.0 \cdot ksi$ $t_p := 1.00 \cdot in$ $p_{fo} := 2.07 \cdot in$ $p_b := 3.00 \cdot in$

$b_f := 6.94 \cdot in$ $F_{uf} := 78.2 \cdot ksi$ $b_p := 8.00 \cdot in$ $p_{fi} := 1.96 \cdot in$

$t_f := 0.63 \cdot in$ $F_{yw} := 54.9 \cdot ksi$ $L_p := 32.00 \cdot in$ $g := 4.47 \cdot in$

$t_w := 0.38 \cdot in$ $F_{uw} := 69.1 \cdot ksi$ $p_{ext} := 4.00 \cdot in$ $F_{yp} := 63.3 \cdot ksi$

SPECIFIED PROPERTIES

$R_y := 1.1$ $P_t := 70 \cdot k$

BEAM MOMENT STRENGTH, Mb

$\frac{b_f}{2 \cdot t_f} = 5.5$ $\frac{65}{\sqrt{50}} = 9.2$ Flange is compact

$h := d - 2 \cdot t_f$

$\frac{h}{t_w} = 59.8$ $\frac{640}{\sqrt{50}} = 90.5$ Web is compact

Therefore, Mb= Mp

$$M_b := 2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right) \cdot F_{yf} + 2 \cdot t_w \cdot \frac{h}{2} \cdot \frac{h}{4} \cdot F_{yw} \qquad M_b = 701.6 k \cdot ft$$

BEAM PLASTIC STRENGTH, Mp

$$M_p := M_b \qquad M_p = 701.6 k \cdot ft$$

BEAM EXPECTED PLASTIC STRENGTH, Mpe

$$M_{pe} := 1.1 \cdot R_y \cdot \left[2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right) \cdot 50 \cdot ksi + 2 \cdot t_w \cdot \frac{h}{2} \cdot \frac{h}{4} \cdot 50 \cdot ksi\right] \qquad M_{pe} = 762.8 k \cdot ft$$

END-PLATE STRENGTH, Mpl

$d_0 := d - \frac{t_f}{2} + p_{fo}$ $\quad d_0 = 25.8in$ $\quad h_0 := d_0 + \frac{t_f}{2}$ $\quad h_0 = 26.1in$

$d_1 := d - \frac{t_f}{2} - t_f - p_{fi}$ $\quad d_1 = 21.1in$ $\quad h_1 := d_1 + \frac{t_f}{2}$ $\quad h_1 = 21.4in$

$d_2 := d_1 - p_b$ $\quad d_2 = 18.1in$ $\quad h_2 := d_2 + \frac{t_f}{2}$ $\quad h_2 = 18.4in$

$s := \frac{1}{2} \cdot \sqrt{b_p \cdot g}$ $\quad s = 2.99in$

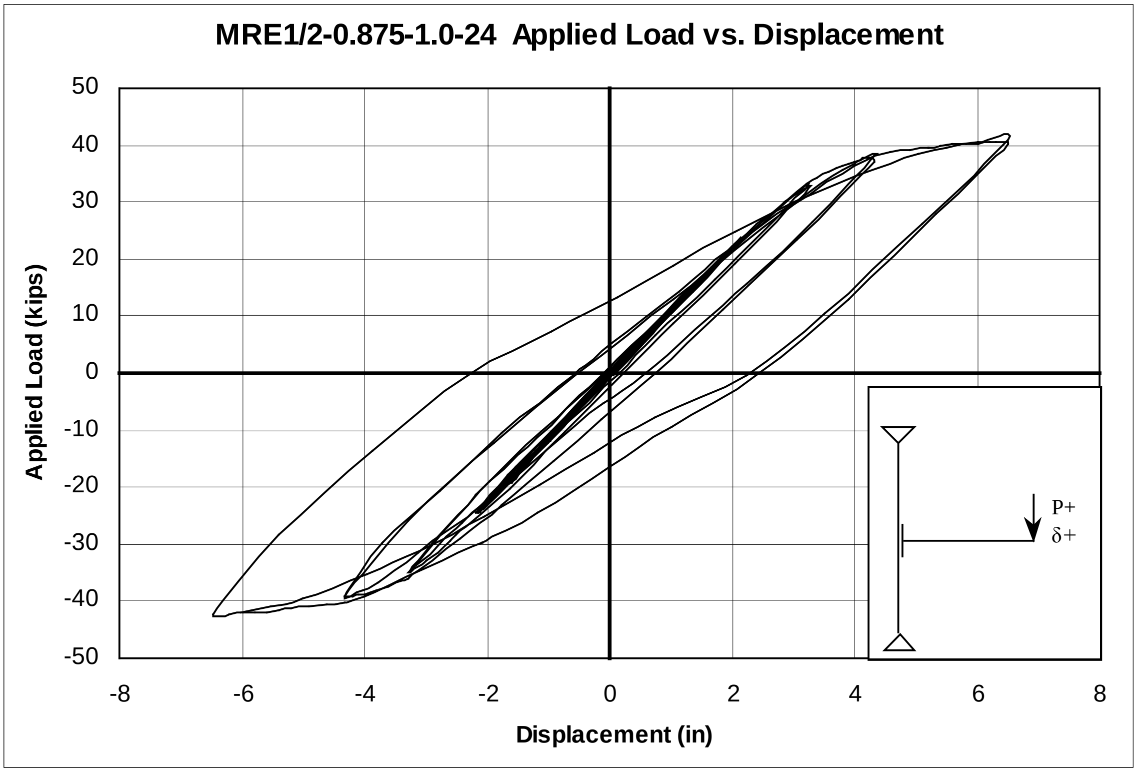
$$Y := \frac{b_p}{2} \cdot \left(\frac{h_1}{p_{fi}} + \frac{h_2}{s} + \frac{h_0}{p_{fo}} - \frac{1}{2} \right) + \frac{2}{g} \cdot \left[h_1 \cdot \left(p_{fi} + 0.75 \cdot p_b \right) + h_2 \cdot \left(s + 0.25 \cdot p_b \right) \right] + \frac{g}{2}$$

$Y = 190.1in$

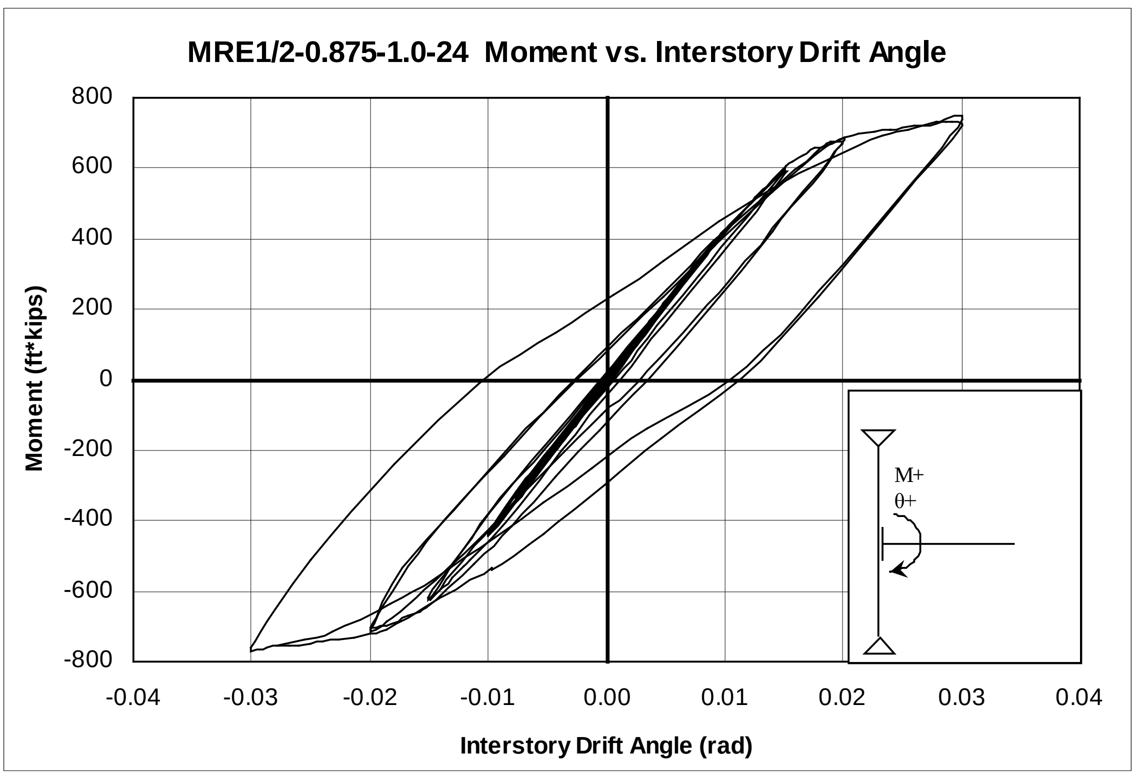
$M_{PL} := t_p{}^2 \cdot F_{yp} \cdot Y$ $\quad M_{PL} = 1002.6k \cdot ft$

BOLT STRENGTH WITHOUT PRYING, Mnp

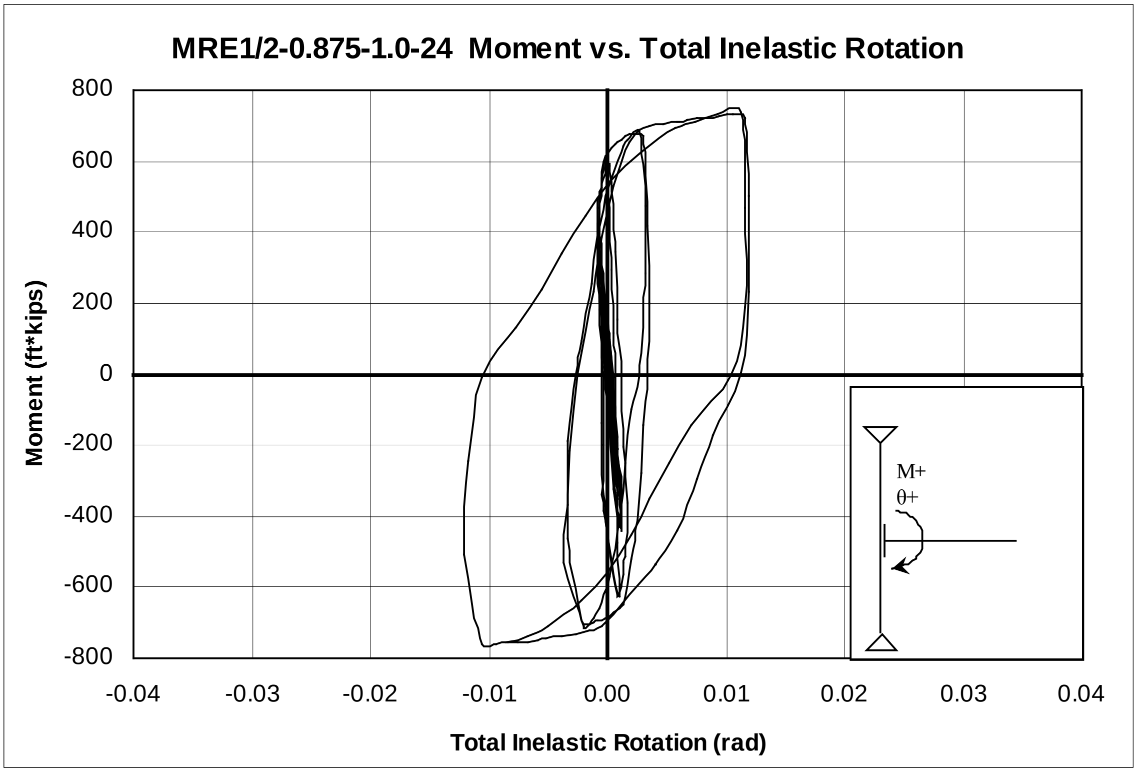
$M_{NP} := 2 \cdot P_t \cdot \left(d_0 + d_1 + d_2 \right)$ $\quad M_{NP} = 757.7k \cdot ft$

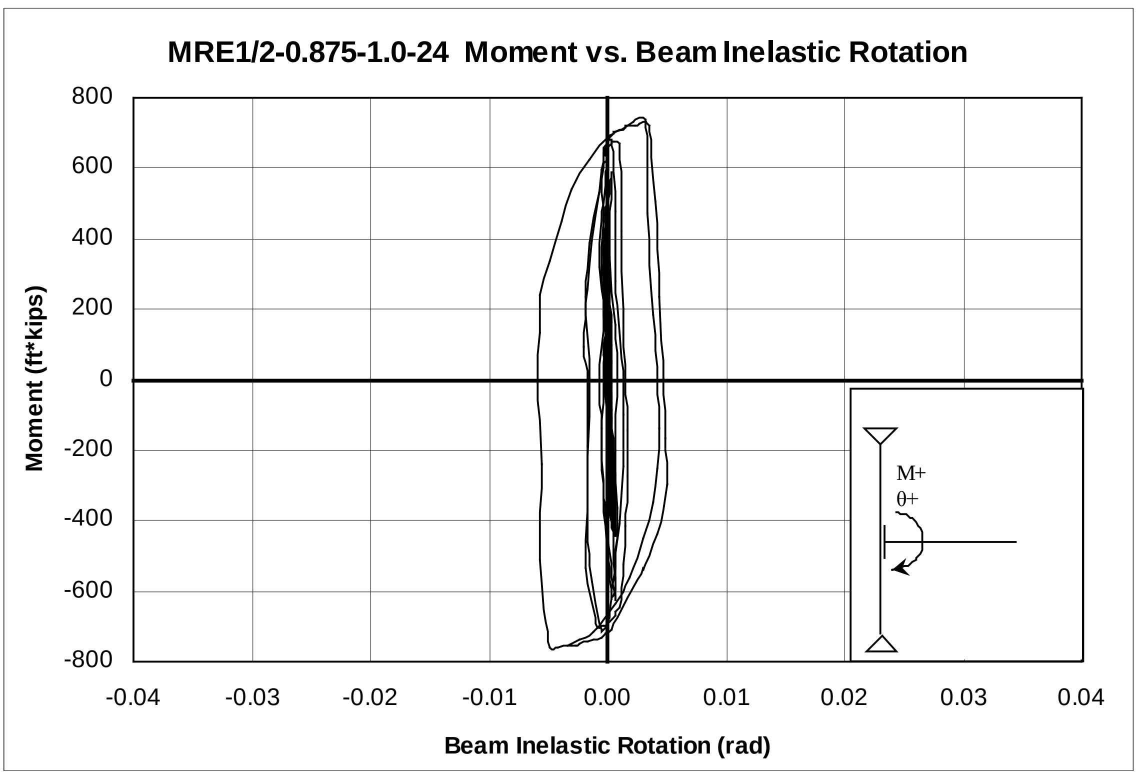


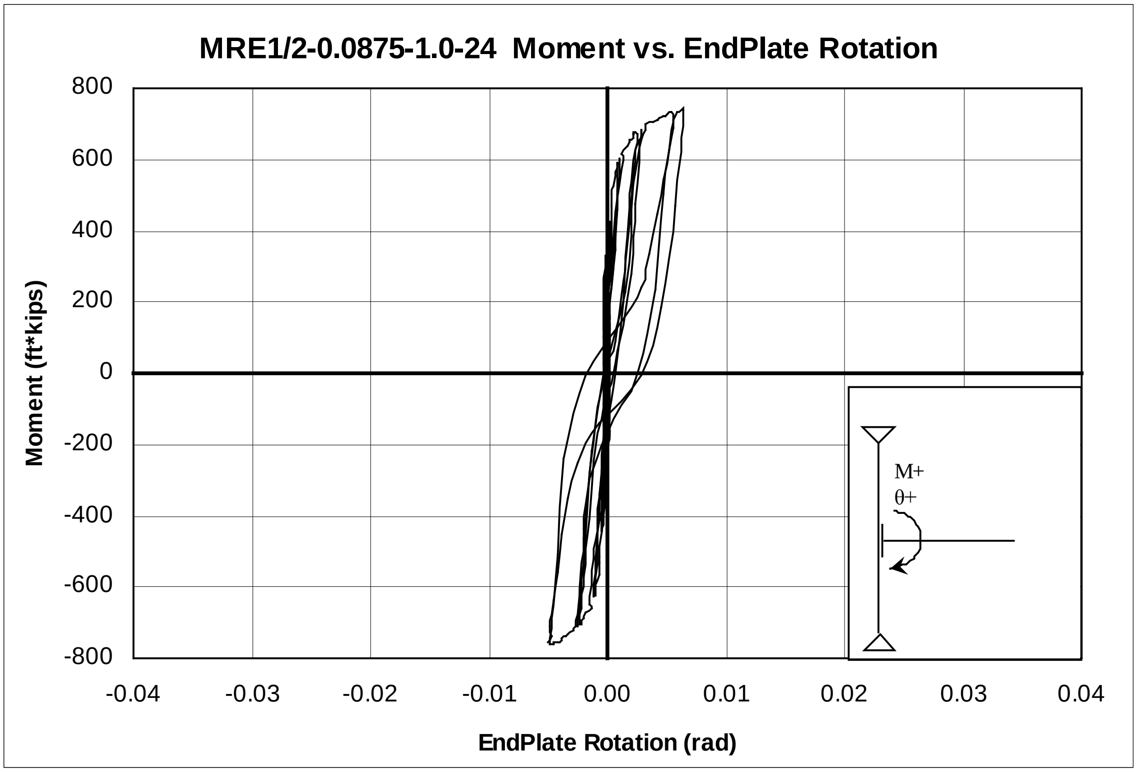
MRE1/2-0.875-1.0-24 Applied Load vs. Displacement
Applied Load (kips)
50
40
30
20
10
0
-10
-20
-30
-40
-50
-8
-6
-4
-2
0
2
4
6
8
Displacement (in)
P+
δ+

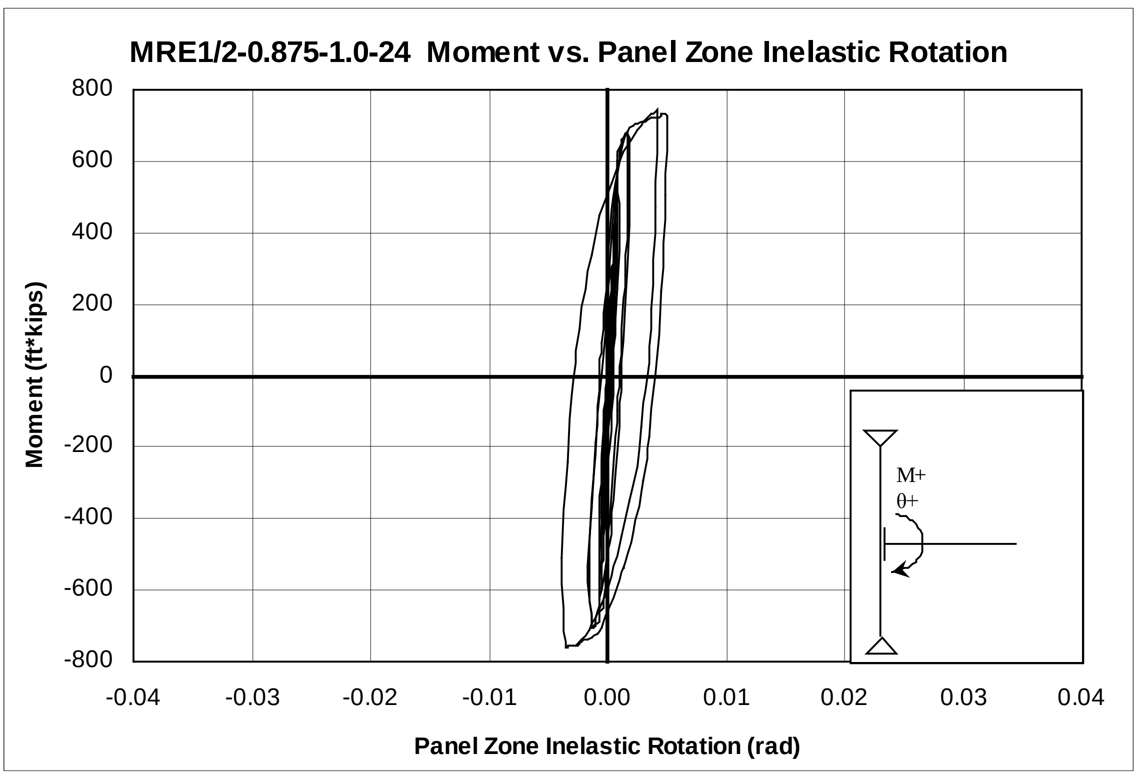


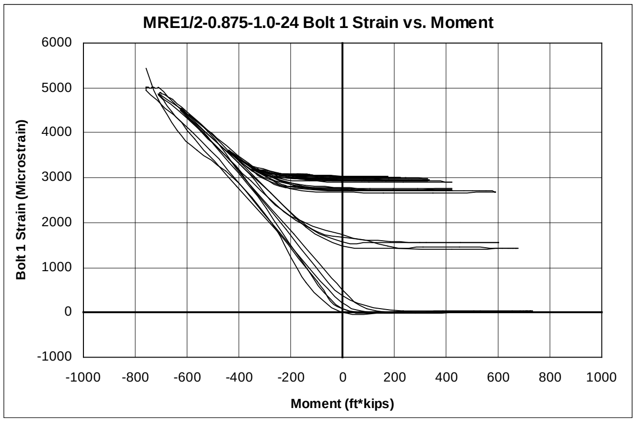
MRE1/2-0.875-1.0-24 Moment vs. Interstory Drift Angle
Moment (ft*kips)
800
600
400
200
0
-200
-400
-600
-800
-0.04
-0.03
-0.02
-0.01
0.00
0.01
0.02
0.03
0.04
Interstory Drift Angle (rad)
M+
θ+

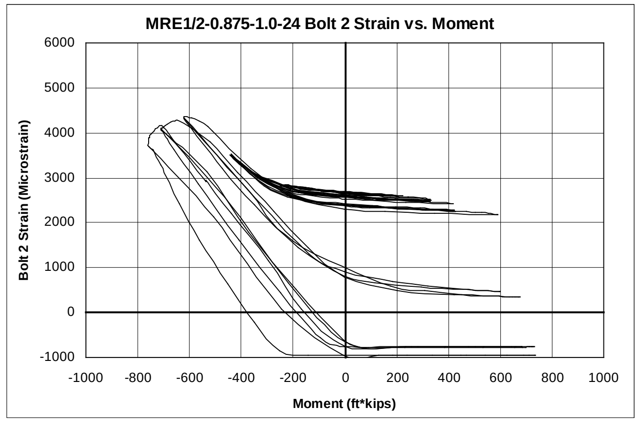

MRE1/2-0.875-1.0-24 Moment vs. Total Inelastic Rotation
Moment (ft*kips)
800
600
400
200
0
-200
-400
-600
-800
-0.04
-0.03
-0.02
-0.01
0.00
0.01
0.02
0.03
0.04
Total Inelastic Rotation (rad)
M+
θ+

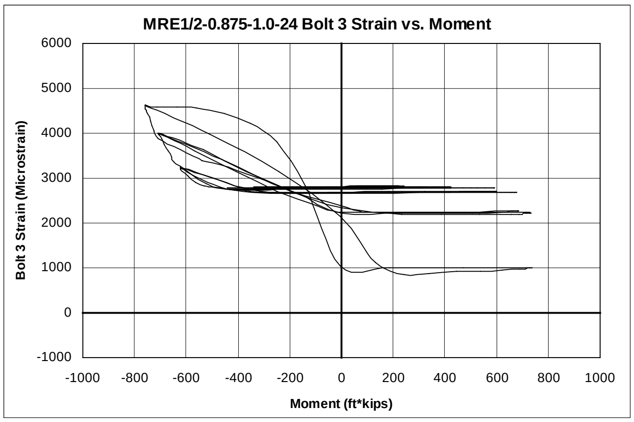

MRE1/2-0.875-1.0-24 Moment vs. Beam Inelastic Rotation
Moment (ft*kips)
800
600
400
200
0
-200
-400
-600
-800
-0.04
-0.03
-0.02
-0.01
0.00
0.01
0.02
0.03
0.04
Beam Inelastic Rotation (rad)
M+
θ+

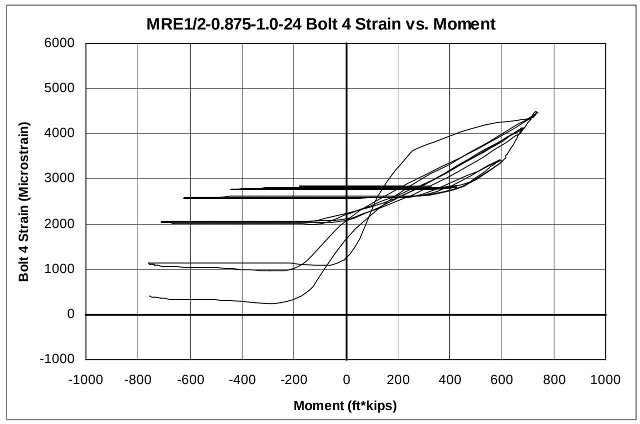

MRE1/2-0.0875-1.0-24 Moment vs. EndPlate Rotation
800
600
400
200
0
-200
-400
-600
-800
Moment (ft*kips)
-0.04
-0.03
-0.02
-0.01
0.00
0.01
0.02
0.03
0.04
EndPlate Rotation (rad)
M+
θ+

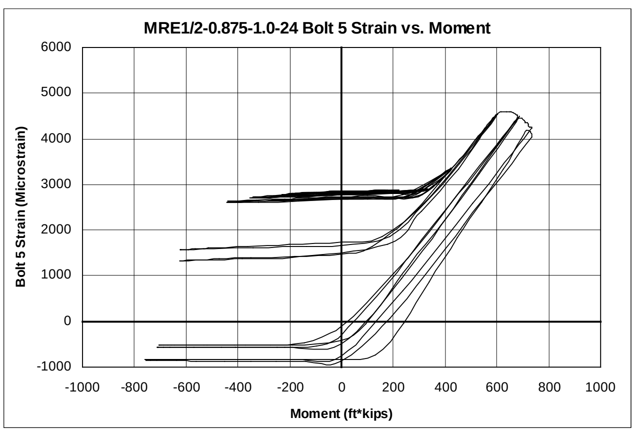

MRE1/2-0.875-1.0-24 Moment vs. Panel Zone Inelastic Rotation
800
600
400
200
0
-200
-400
-600
-800
Moment (ft*kips)
-0.04
-0.03
-0.02
-0.01
0.00
0.01
0.02
0.03
0.04
Panel Zone Inelastic Rotation (rad)
M+
θ+

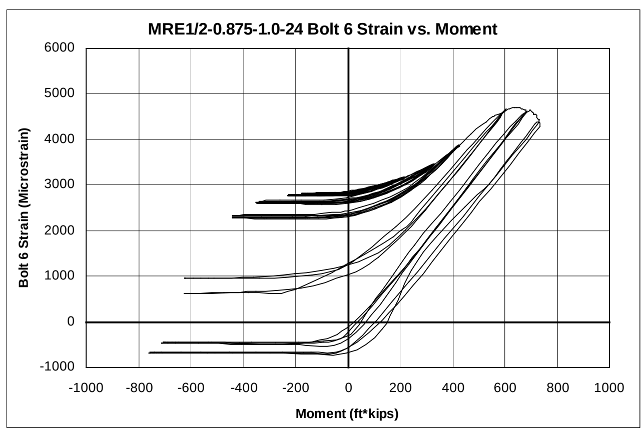

MRE1/2-0.875-1.0-24 Bolt 1 Strain vs. Moment
Bolt 1 Strain (Microstrain)
Moment (ft*kips)

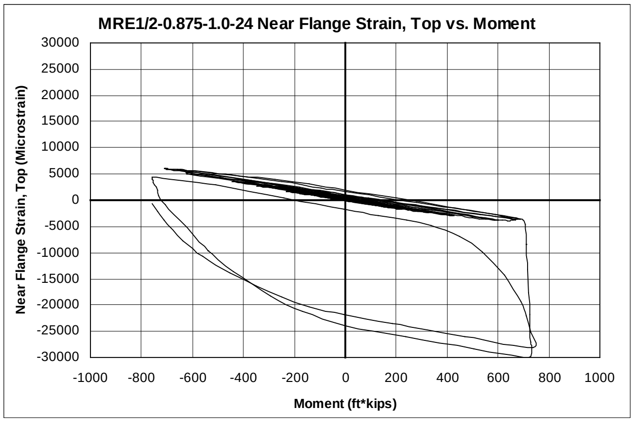

MRE1/2-0.875-1.0-24 Bolt 2 Strain vs. Moment
Bolt 2 Strain (Microstrain)
Moment (ft*kips)

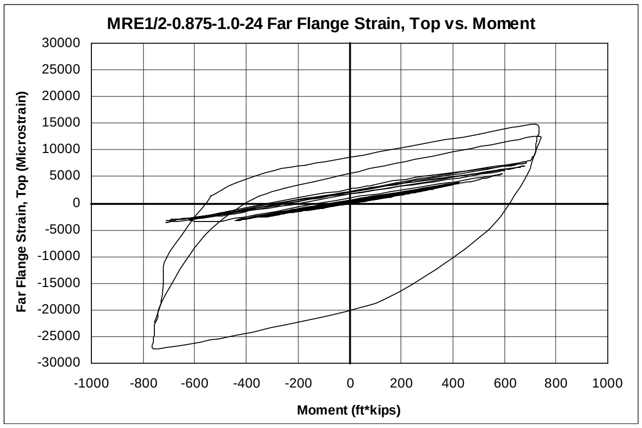

MRE1/2-0.875-1.0-24 Bolt 3 Strain vs. Moment
Bolt 3 Strain (Microstrain)
Moment (ft*kips)

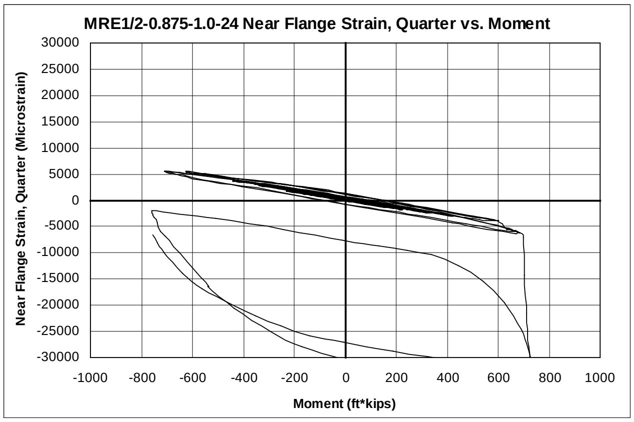

MRE1/2-0.875-1.0-24 Bolt 4 Strain vs. Moment
Bolt 4 Strain (Microstrain)
Moment (ft*kips)

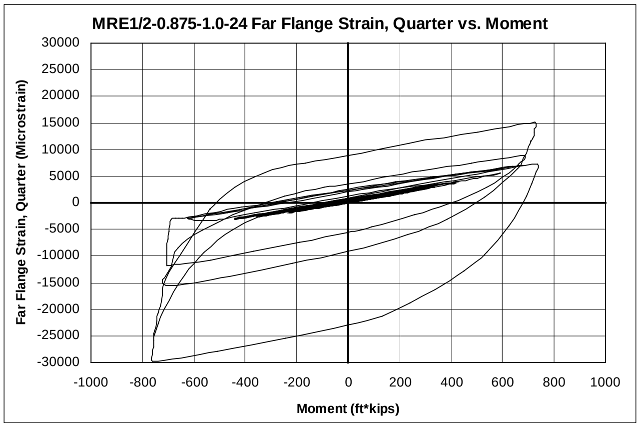

MRE1/2-0.875-1.0-24 Bolt 5 Strain vs. Moment
Bolt 5 Strain (Microstrain)
Moment (ft*kips)

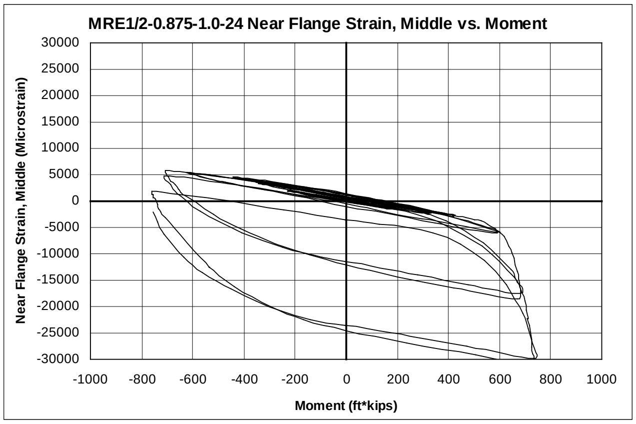

MRE1/2-0.875-1.0-24 Bolt 6 Strain vs. Moment
Bolt 6 Strain (Microstrain)
Moment (ft*kips)

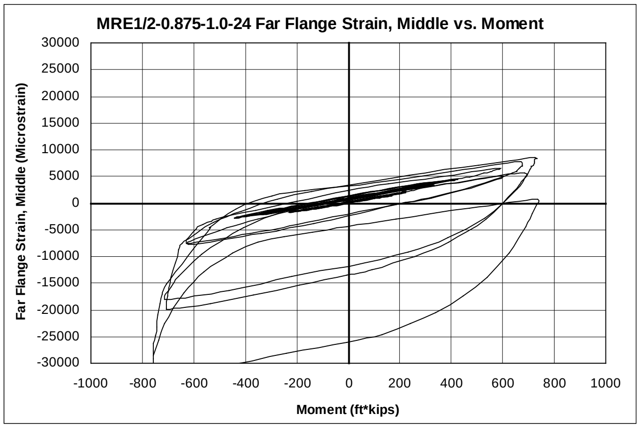

MRE1/2-0.875-1.0-24 Near Flange Strain, Top vs. Moment
Near Flange Strain, Top (Microstrain)
Moment (ft*kips)

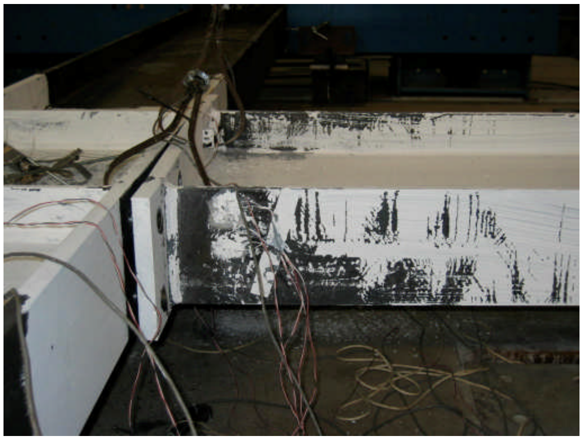

MRE1/2-0.875-1.0-24 Far Flange Strain, Top vs. Moment
Far Flange Strain, Top (Microstrain)
Moment (ft*kips)


MRE1/2-0.875-1.0-24 Near Flange Strain, Quarter vs. Moment
Near Flange Strain, Quarter (Microstrain)
Moment (ft*kips)

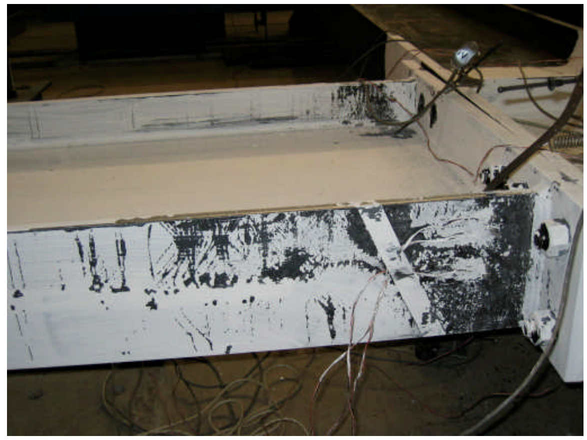

MRE1/2-0.875-1.0-24 Far Flange Strain, Quarter vs. Moment
Far Flange Strain, Quarter (Microstrain)
Moment (ft*kips)


MRE1/2-0.875-1.0-24 Near Flange Strain, Middle vs. Moment
Near Flange Strain, Middle (Microstrain)
Moment (ft*kips)

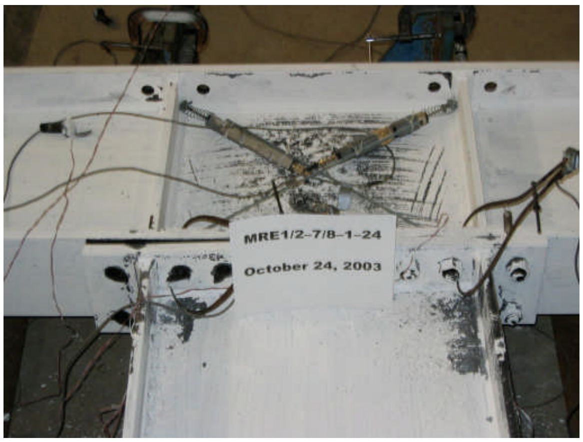

MRE1/2-0.875-1.0-24 Far Flange Strain, Middle vs. Moment
Far Flange Strain, Middle (Microstrain)
Moment (ft*kips)



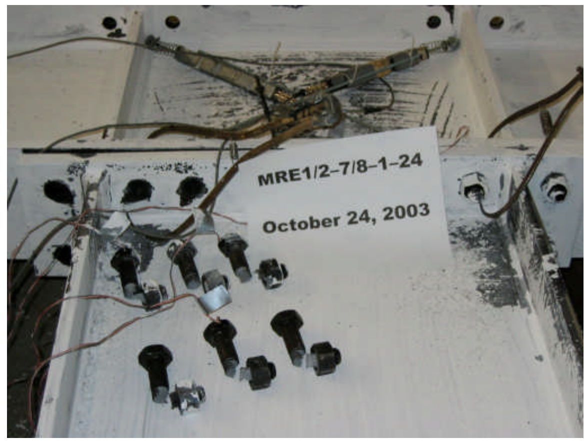
Near Flange



Far Flange



Top View



Top View with Bolts

MRE1/2-0.875-1.0-24 AFTER TESTING

APPENDIX C

4ES – 1.0 – 1.0 – 24

RESULTS AND TEST DATA

SPECIMEN PROPERTIES & TEST SUMMARY

TEST NAME: 4ES-1.0-1.0-24
TEST DATE: November 14, 2003

BEAM DATA

DEPTH, d:	24.00	in
FLANGE WIDTH, b_f:	6.94	in
FLANGE THICKNESS, t_f:	0.63	in
WEB THICKNESS, t_w:	0.38	in
FLANGE YIELD STRESS, F_{yf}:	56.0	ksi
FLANGE ULTIMATE STRESS, F_{uf}:	78.2	ksi
WEB YIELD STRESS, F_{yw}:	54.9	ksi
WEB ULTIMATE STRESS, F_{uw}:	69.1	ksi

END-PLATE DATA

END-PLATE THICKNESS, t_p:	1.01	in
END-PLATE WIDTH, b_p:	8.00	in
END-PLATE LENGTH, L_p:	32.00	in
END-PLATE EXTENSION OUTSIDE FLANGE, p_{ext}:	4.00	in
OUTER PITCH, BOLT TO FLANGE, p_{fo}:	2.01	in
INNER PITCH, BOLT TO FLANGE, p_{fi}:	2.09	in
PITCH, BOLT TO BOLT, p_b:	na	
GAGE, g:	4.50	in
END-PLATE YIELD STRESS, F_{yp}:	60.7	ksi
END-PLATE ULTIMATE STRESS, F_{up}:	85.3	ksi

BOLT DATA

BOLT DIAMETER, d_b:	1.00	in
BOLT LENGTH, L_b:	3.25	in
BOLT TYPE:	A490	
NOMINAL BOLT TENSILE STRESS (AISC J3.6), F_t:	113	ksi
NOMINAL BOLT TENSILE STRENGTH, P_t:	91	kips
BOLT PRETENSION, T_b:	64	kips

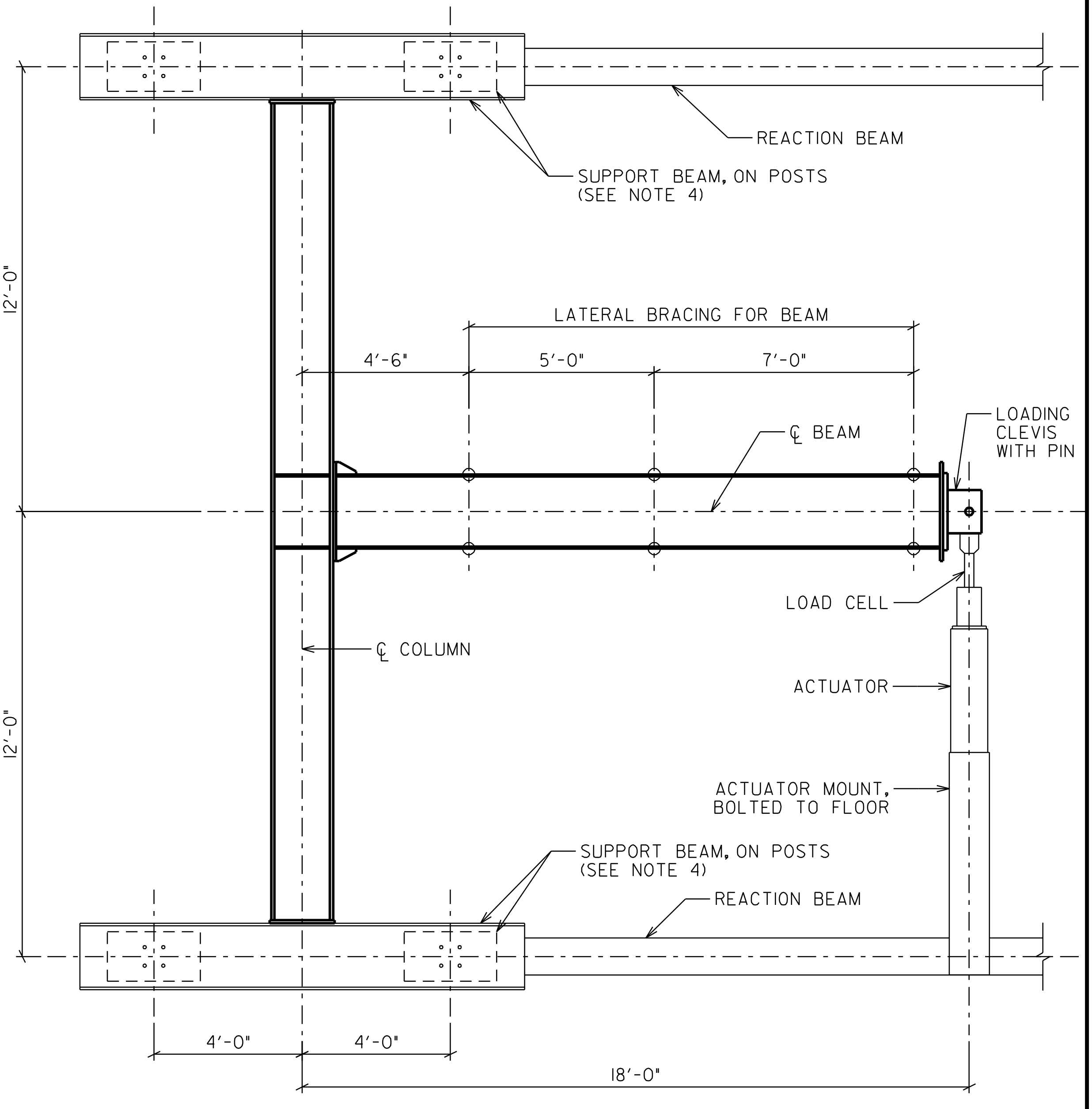
EXPERIMENTAL RESULTS

FAILURE MODE:	Flange Local Buckling	
MAXIMUM MOMENT AT COLUMN CENTERLINE, $M_{max,cc}$:	805.4	ft-kips
MAXIMUM MOMENT AT FAYING SURFACE, $M_{max,fs}$:	768.1	ft-kips
MAXIMUM INTERSTORY DRIFT ANGLE, θ_{max}:	0.050	rad
MAXIMUM INTERSTORY DRIFT ANGLE AT 80% 0F $M_{max,cc}$, θ_{preq}:	0.040	rad

CALCULATED STRENGTHS

BEAM MOMENT STRENGTH [1], M_b:	701.6	ft-kips
BEAM PLASTIC STRENGTH [1], M_p:	701.6	ft-kips
BEAM EXPECTED PLASTIC STRENGTH, M_{pe}:	762.8	ft-kips
END-PLATE STRENGTH [1], M_{PL}:	1200.1	ft-kips
BOLT TENSION RUPTURE (w/o Prying, using F_t), M_{NP}:	707.7	ft-kips

1. Measured material properties used.



4ES-1.0-1.0-24 TEST SUBASSEMBLAGE PLAN

BEAM

WEB = 3/8×22 3/4
FLANGE = 5/8×7
LENGTH = 197 1/2 (INCLUDES ENDPLATES)

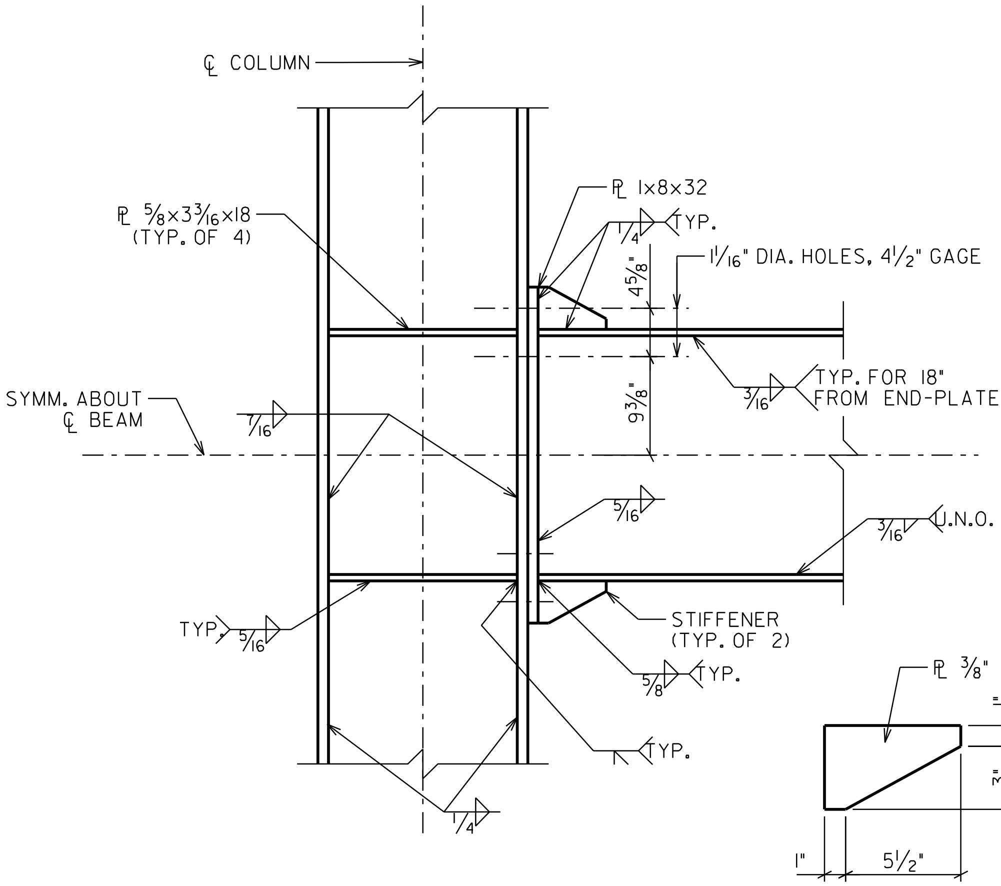
COLUMN

WEB = 3/8×18 1/8
PANEL ZONE WEB = 5/8×18 1/8
FLANGE = 1×8
LENGTH = 266 1/2 (INCLUDES ENDPLATES)

NOTES

1. COLUMN, BEAM, AND CONNECTION STEEL, A572 Gr50.
2. ALL BOLTS, A490.
3. ALL WELDING, AWS D1.1, E70XX ELECTRODES.
4. COLUMN BOLTED TO SUPPORT BEAM. SUPPORT BEAM BOLTED POSTS. POSTS BOLTED TO REACTION BEAM.

SPECIMEN: 4ES-0.75-0.75-24	DRAWN BY: SEB



4ES-1.0-1.0-24
CONNECTION DETAIL

STIFFENER

SPECIMEN: 4ES-1.0-1.0-24	DRAWN BY: SEB

SAC PROTOCOL LOADING HISTORY

Test Name: 4ES-1.0-1.0-24 Test By: SEB Date: 11/14/2003

Load Step	Cycle	Max. Pos. Load (kips)	Max. Disp. (in)	Max. Neg. Load (kips)	Max. Disp. (in)	Comments
I (0.00375 rad)	1	9.5	0.816	-9.0	-0.810	
δ=0.81"	2	9.2	0.814	-9.3	-0.808	
	3	9.3	0.828	-9.1	-0.811	
	4	9.2	0.814	-9.3	-0.825	
	5	9.1	0.814	-9.2	-0.811	
	6	9.2	0.814	-9.4	-0.818	
		Permanent Set =	-0.008"			
II (0.005 rad)	1	11.9	1.083	-12.6	-1.104	
δ=1.08"	2	12.0	1.087	-12.6	-1.088	
	3	12.1	1.105	-12.6	-1.092	
	4	11.8	1.082	-12.6	-1.095	
	5	12.2	1.113	-12.7	-1.101	
	6	11.9	1.085	-12.6	-1.098	
		Permanent Set =	-0.007"			
III (0.0075 rad)	1	17.3	1.640	-18.3	-1.623	
δ=1.62"	2	17.3	1.636	-18.5	-1.635	
	3	17.3	1.638	-18.3	-1.620	
	4	17.3	1.634	-18.4	-1.623	
	5	17.3	1.636	-18.4	-1.622	
	6	17.3	1.633	-18.4	-1.622	
		Permanent Set =	-0.013"			
IV (0.01 rad)	1	22.2	2.165	-24.0	-2.164	
δ=2.16"	2	22.3	2.162	-24.0	-2.160	
	3	22.3	2.163	-24.1	-2.168	
	4	22.3	2.163	-24.1	-2.175	
		Permanent Set =	-0.015"			
V (0.015 rad)	1	31.8	3.254	-34.8	-3.247	
δ=3.24"	2	32.0	3.240	-34.9	-3.248	
		Permanent Set =	-0.071"			
VI (0.02 rad)	1	38.1	4.327	-41.6	-4.343	Slight whitewash flaking
δ=4.32"	2	38.8	4.355	-41.4	-4.338	
		Permanent Set =	-0.561"			
VII (0.03 rad)	1	42.3	6.487	-44.2	-6.488	Extensive whitewash flaking
δ=6.48"	2	42.6	6.492	-44.3	-6.471	
		Permanent Set =	-2.221"			
VIII (0.04 rad)	1	44.2	8.659	-43.0	-8.656	Flange local buckling
δ=8.64"	2	43.5	8.650	-39.4	-8.643	
		Permanent Set =	-4.125"			
IX (0.05 rad)	1	40.2	10.803	~ -34	~ -9	Test Stopped @ Bracing Limits
δ=10.8"	2					
		Permanent Set =				

CALCULATED STRENGTHS

TEST NAME: 4ES-1.0-1.0-24
TEST DATE: November 14, 2003

Specified Grade for all Steel = ASTM A572, Grade 50

MEASURED PROPERTIES

$d := 24 \cdot in$ $\quad F_{yf} := 56.0 \cdot ksi$ $\quad t_p := 1.01 \cdot in$ $\quad p_{fo} := 2.01 \cdot in$

$b_f := 6.94 \cdot in$ $\quad F_{uf} := 78.2 \cdot ksi$ $\quad b_p := 8.00 \cdot in$ $\quad p_{fi} := 2.09 \cdot in$

$t_f := 0.63 \cdot in$ $\quad F_{yw} := 54.9 \cdot ksi$ $\quad L_p := 32.00 \cdot in$ $\quad g := 4.5 \cdot in$

$t_w := 0.38 \cdot in$ $\quad F_{uw} := 69.1 \cdot ksi$ $\quad p_{ext} := 4.00 \cdot in$ $\quad F_{yp} := 60.7 \cdot ksi$

SPECIFIED PROPERTIES

$R_y := 1.1$ $\quad P_t := 91 \cdot k$

BEAM MOMENT STRENGTH, Mb

$\frac{b_f}{2 \cdot t_f} = 5.5$ $\quad \frac{65}{\sqrt{50}} = 9.2$ $\quad$ Flange is compact

$h := d - 2 \cdot t_f$

$\frac{h}{t_w} = 59.8$ $\quad \frac{640}{\sqrt{50}} = 90.5$ $\quad$ Web is compact

Therefore, Mb= Mp

$$M_b := 2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2} \right) \cdot F_{yf} + 2 \cdot t_w \cdot \frac{h}{2} \cdot \frac{h}{4} \cdot F_{yw} \qquad M_b = 701.6 k \cdot ft$$

BEAM PLASTIC STRENGTH, Mp

$$M_p := M_b \qquad M_p = 701.6 k \cdot ft$$

BEAM EXPECTED PLASTIC STRENGTH, Mpe

$$M_{pe} := 1.1 \cdot R_y \cdot \left[2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2} \right) \cdot 50 \cdot ksi + 2 \cdot t_w \cdot \frac{h}{2} \cdot \frac{h}{4} \cdot 50 \cdot ksi \right] \qquad M_{pe} = 762.8 k \cdot ft$$

END-PLATE STRENGTH, Mpl

$d_0 := d - \frac{t_f}{2} + p_{fo}$ $\quad d_0 = 25.7\text{in}$ $\qquad h_0 := d_0 + \frac{t_f}{2}$ $\quad h_0 = 26\text{in}$

$d_1 := d - \frac{t_f}{2} - t_f - p_{fi}$ $\quad d_1 = 21\text{in}$ $\qquad h_1 := d_1 + \frac{t_f}{2}$ $\quad h_1 = 21.3\text{in}$

$s := \frac{1}{2} \cdot \sqrt{b_p \cdot g}$ $\quad s = 3\text{in}$ $\qquad d_e := p_{ext} - p_{fo}$ $\quad d_e = 1.99\text{in}$

s > de, therefore Case 2 controls

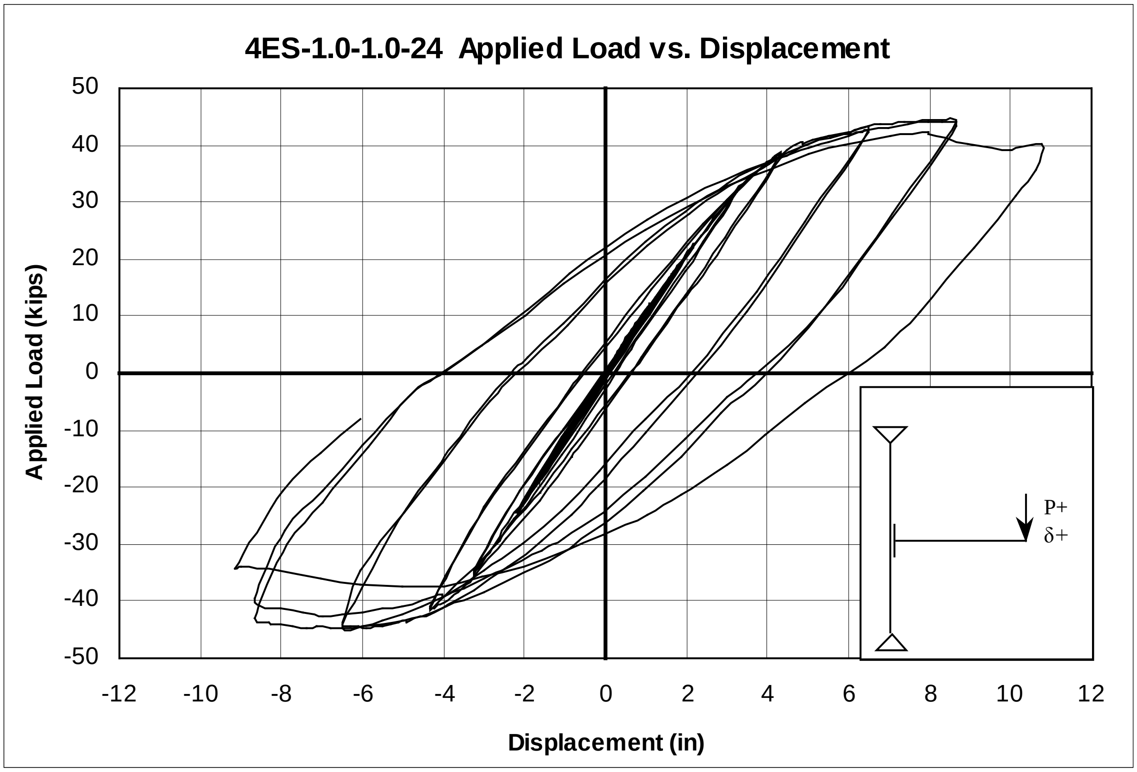
$$Y := \frac{b_p}{2} \cdot \left[h_1 \cdot \left(\frac{1}{p_{fi}} + \frac{1}{s} \right) + h_0 \cdot \left(\frac{1}{p_{fo}} + \frac{1}{2 \cdot s} \right) \right] + \frac{2}{g} \cdot \left[h_1 \cdot \left(p_{fi} + s \right) + h_0 \cdot \left(d_e + p_{fo} \right) \right]$$

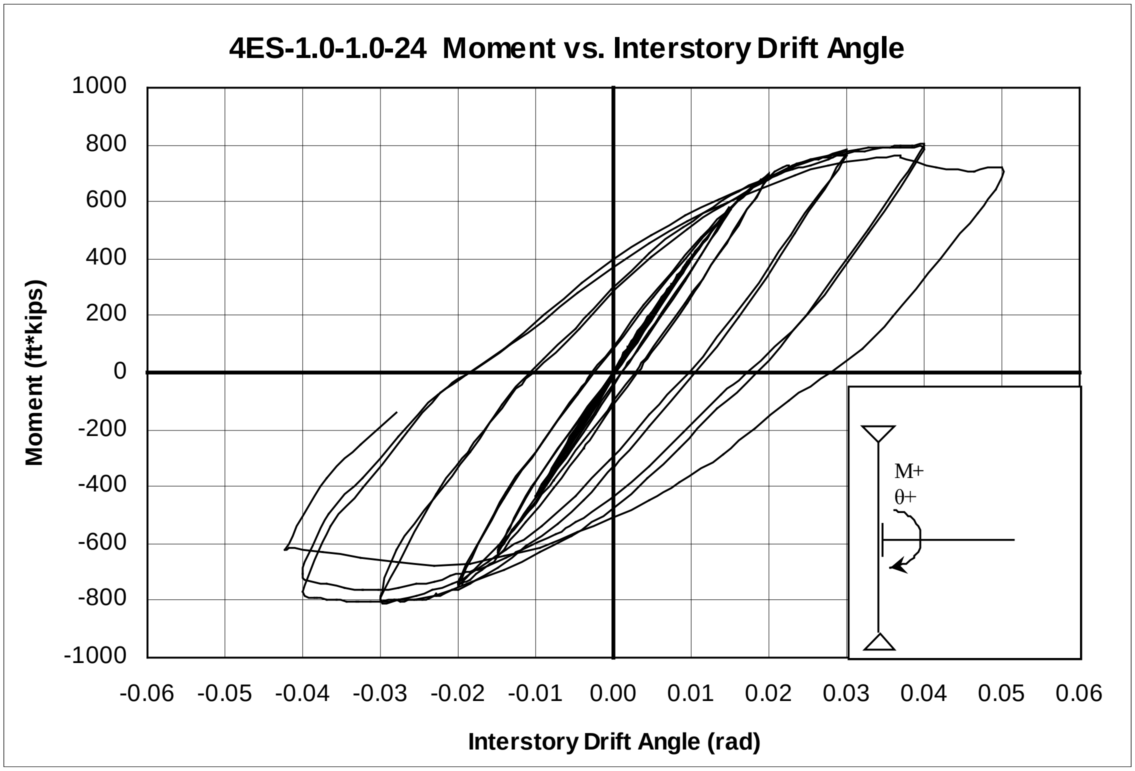
$Y = 232.6\text{in}$

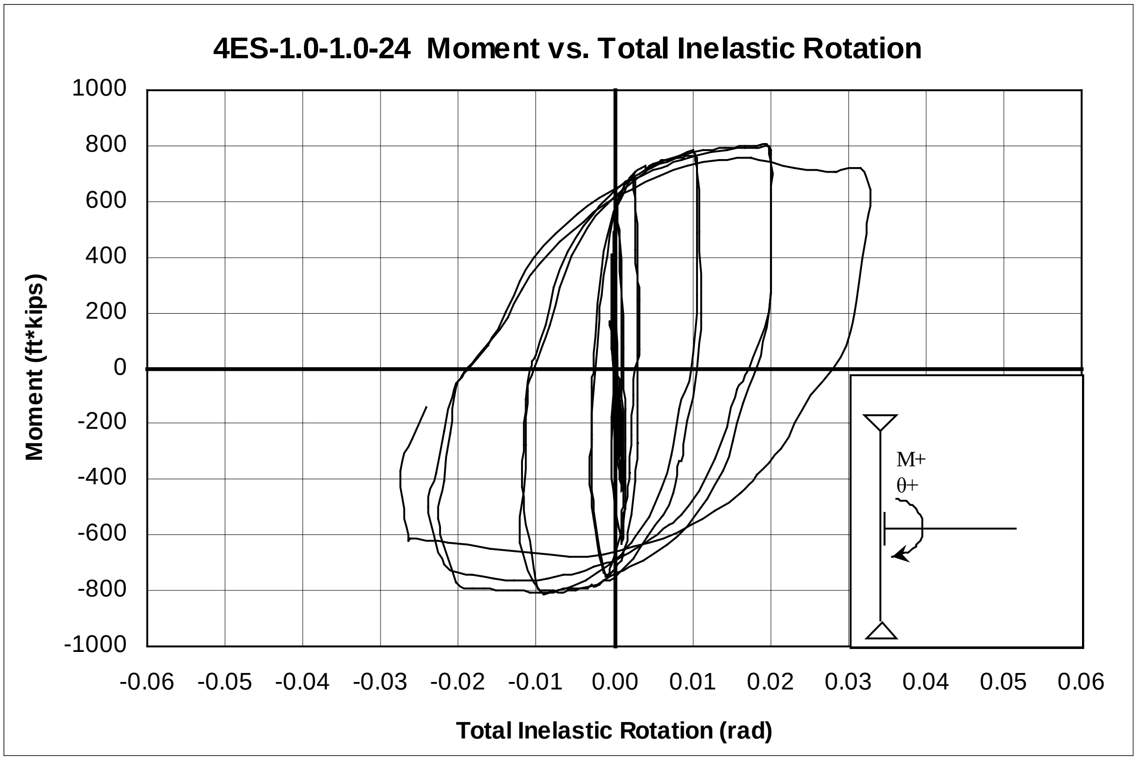
$M_{PL} := t_p^{\,2} \cdot F_{yp} \cdot Y$ $\qquad M_{PL} = 1200.1\text{k} \cdot \text{ft}$

BOLT STRENGTH WITHOUT PRYING, Mnp

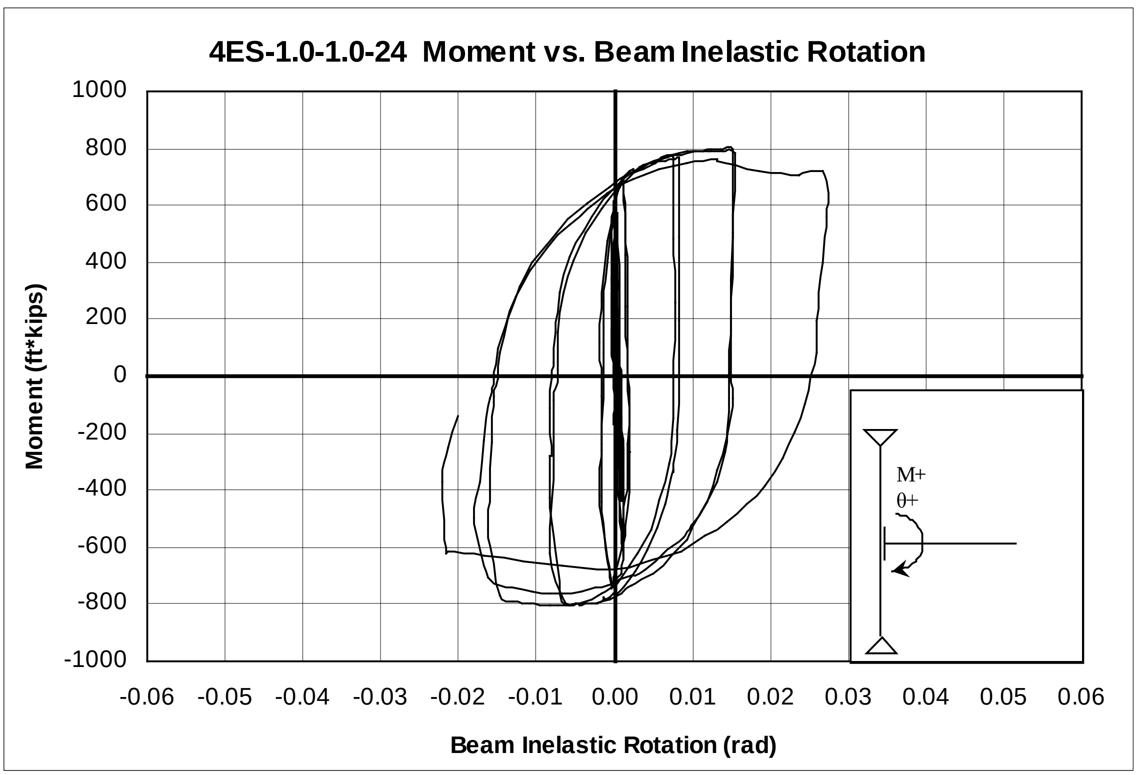
$M_{NP} := 2 \cdot P_t \cdot \left(d_0 + d_1 \right)$ $\qquad M_{NP} = 707.7\text{k} \cdot \text{ft}$


4ES-1.0-1.0-24 Applied Load vs. Displacement
Applied Load (kips)
50
40
30
20
10
0
-10
-20
-30
-40
-50
-12
-10
-8
-6
-4
-2
0
2
4
6
8
10
12
Displacement (in)
P+
δ+

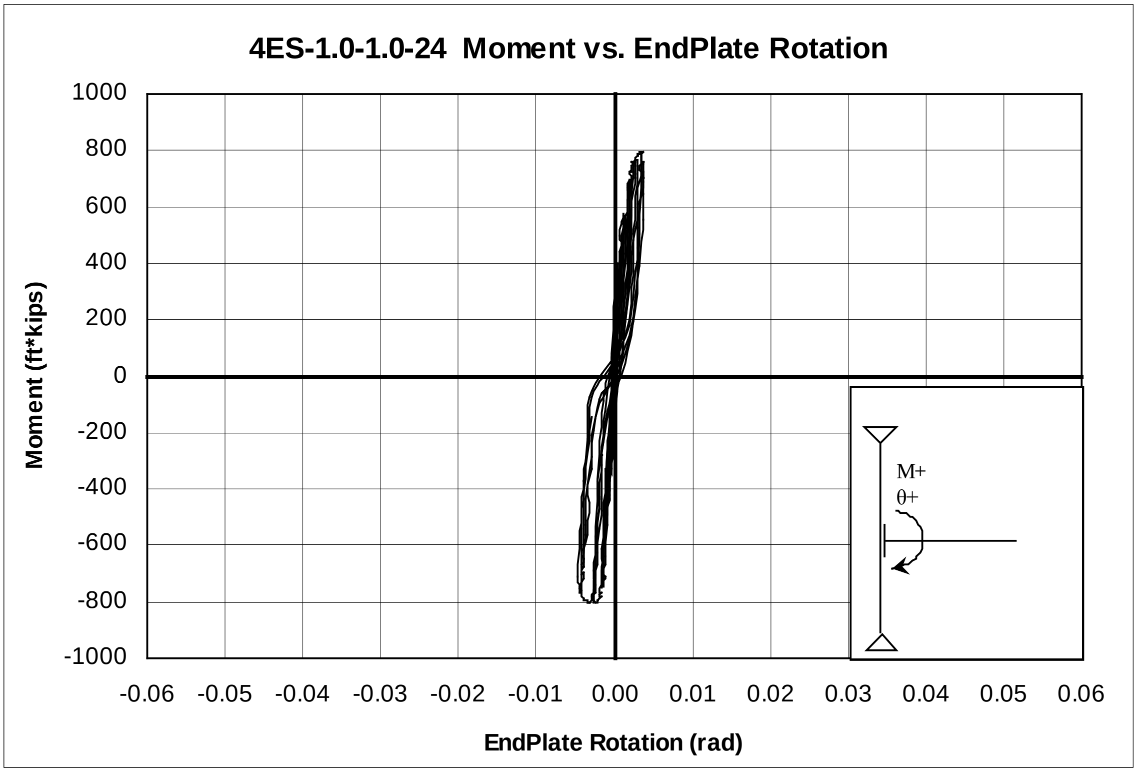

4ES-1.0-1.0-24 Moment vs. Interstory Drift Angle
Moment (ft*kips)
1000
800
600
400
200
0
-200
-400
-600
-800
-1000
-0.06
-0.05
-0.04
-0.03
-0.02
-0.01
0.00
0.01
0.02
0.03
0.04
0.05
0.06
Interstory Drift Angle (rad)
M+
θ+

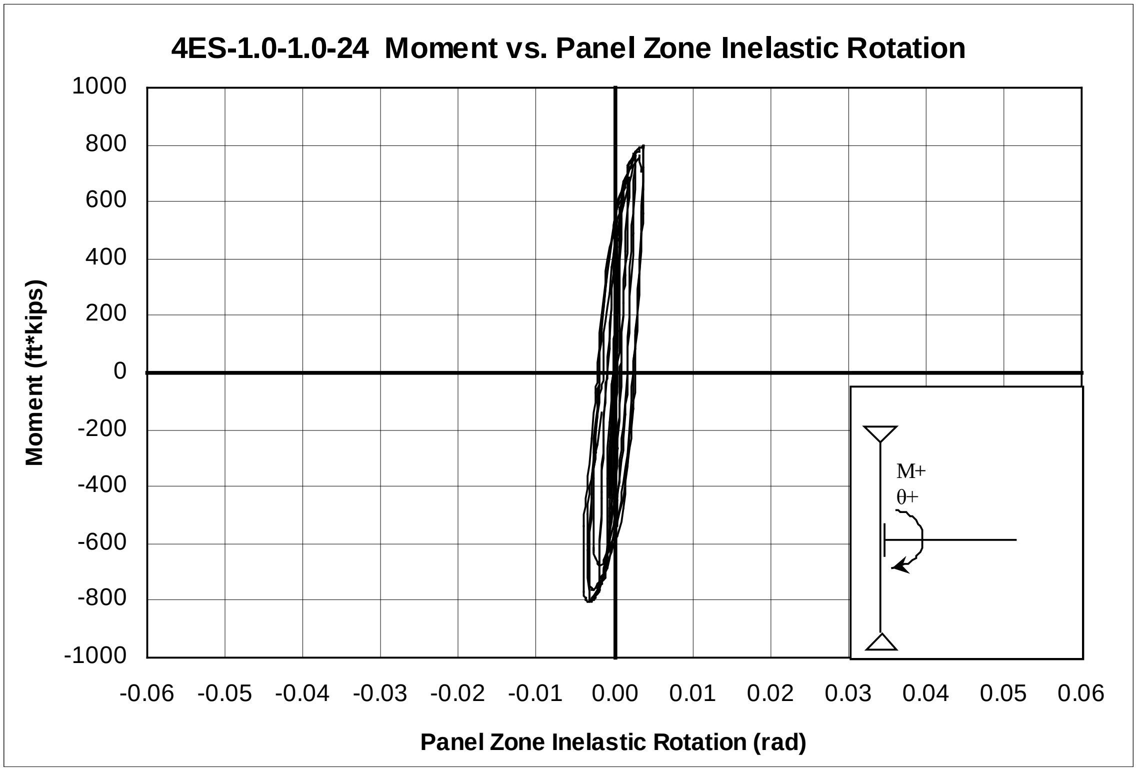


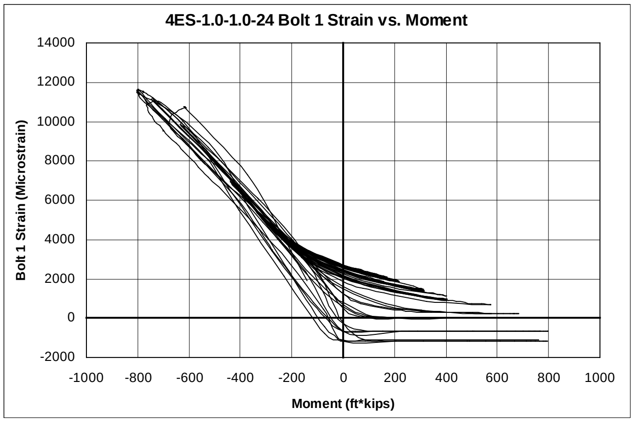
4ES-1.0-1.0-24 Moment vs. Total Inelastic Rotation
Moment (ft*kips)
1000
800
600
400
200
0
-200
-400
-600
-800
-1000
-0.06 -0.05 -0.04 -0.03 -0.02 -0.01 0.00 0.01 0.02 0.03 0.04 0.05 0.06
Total Inelastic Rotation (rad)
M+
θ+

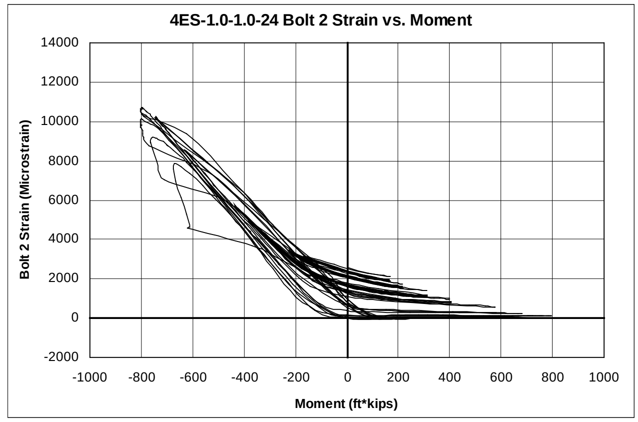


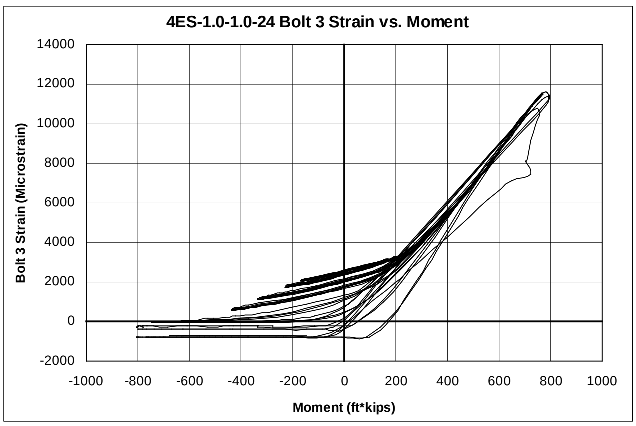
4ES-1.0-1.0-24 Moment vs. Beam Inelastic Rotation
Moment (ft*kips)
1000
800
600
400
200
0
-200
-400
-600
-800
-1000
-0.06 -0.05 -0.04 -0.03 -0.02 -0.01 0.00 0.01 0.02 0.03 0.04 0.05 0.06
Beam Inelastic Rotation (rad)
M+
θ+

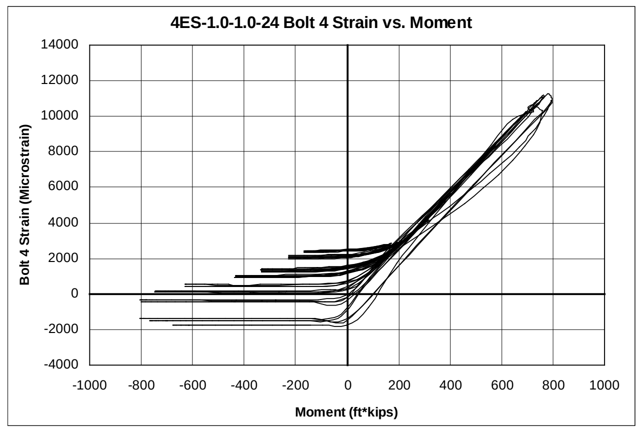

4ES-1.0-1.0-24 Moment vs. EndPlate Rotation
1000
800
600
400
200
0
-200
-400
-600
-800
-1000
Moment (ft*kips)
-0.06
-0.05
-0.04
-0.03
-0.02
-0.01
0.00
0.01
0.02
0.03
0.04
0.05
0.06
EndPlate Rotation (rad)
M+
θ+

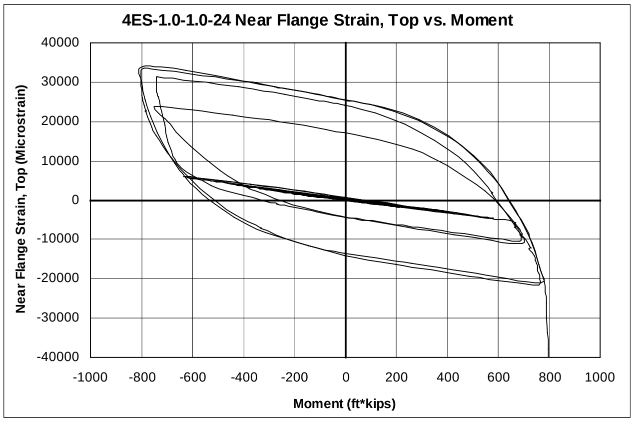

4ES-1.0-1.0-24 Moment vs. Panel Zone Inelastic Rotation
1000
800
600
400
200
0
-200
-400
-600
-800
-1000
Moment (ft*kips)
-0.06
-0.05
-0.04
-0.03
-0.02
-0.01
0.00
0.01
0.02
0.03
0.04
0.05
0.06
Panel Zone Inelastic Rotation (rad)
M+
θ+


4ES-1.0-1.0-24 Bolt 1 Strain vs. Moment
Bolt 1 Strain (Microstrain)
Moment (ft*kips)
14000
12000
10000
8000
6000
4000
2000
0
-2000
-1000
-800
-600
-400
-200
0
200
400
600
800
1000

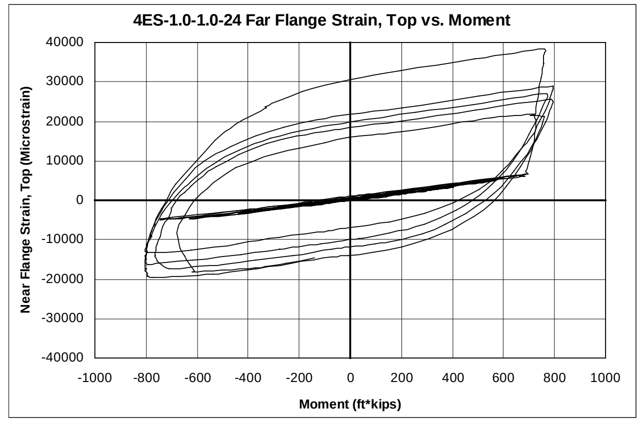

4ES-1.0-1.0-24 Bolt 2 Strain vs. Moment
Bolt 2 Strain (Microstrain)
Moment (ft*kips)
14000
12000
10000
8000
6000
4000
2000
0
-2000
-1000
-800
-600
-400
-200
0
200
400
600
800
1000


4ES-1.0-1.0-24 Bolt 3 Strain vs. Moment
Bolt 3 Strain (Microstrain)
Moment (ft*kips)
14000
12000
10000
8000
6000
4000
2000
0
-2000
-1000
-800
-600
-400
-200
0
200
400
600
800
1000

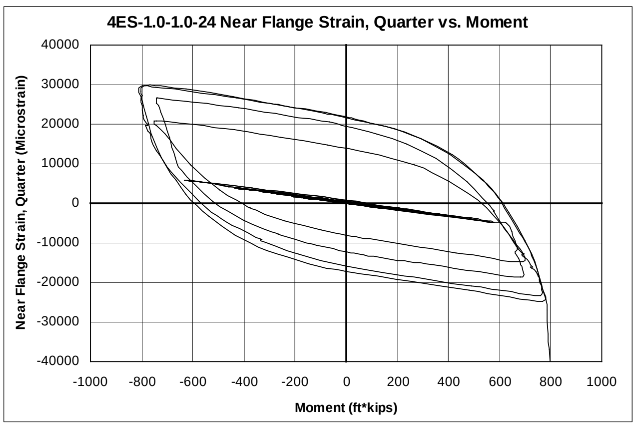

4ES-1.0-1.0-24 Bolt 4 Strain vs. Moment
Bolt 4 Strain (Microstrain)
Moment (ft*kips)
14000
12000
10000
8000
6000
4000
2000
0
-2000
-4000
-1000
-800
-600
-400
-200
0
200
400
600
800
1000



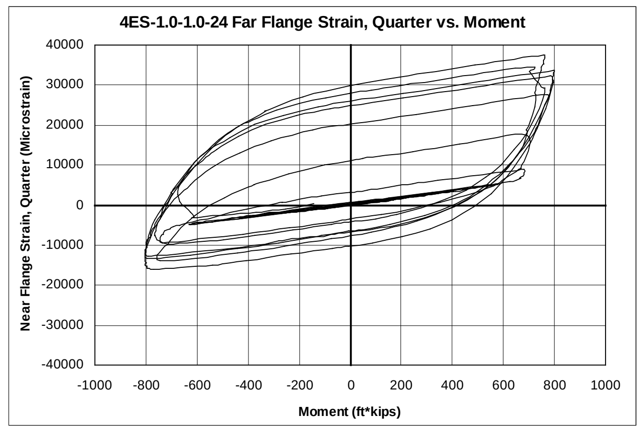
4ES-1.0-1.0-24 Near Flange Strain, Top vs. Moment
Near Flange Strain, Top (Microstrain)
Moment (ft*kips)



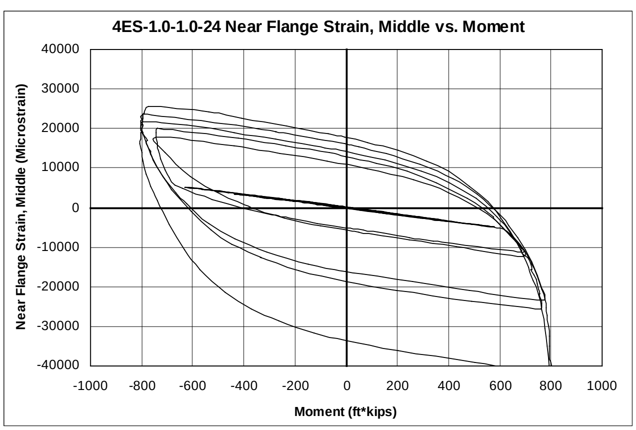
4ES-1.0-1.0-24 Far Flange Strain, Top vs. Moment
Near Flange Strain, Top (Microstrain)
Moment (ft*kips)



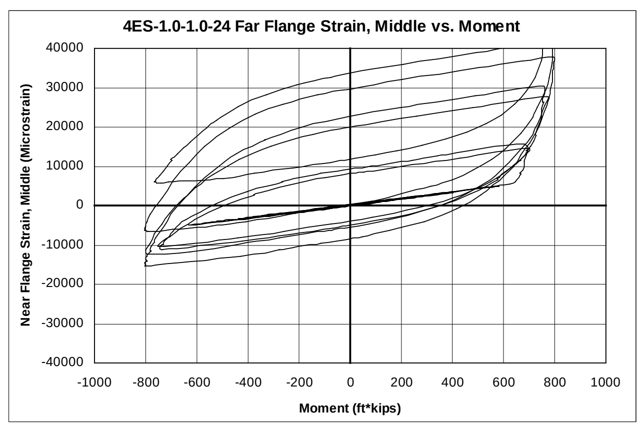
4ES-1.0-1.0-24 Near Flange Strain, Quarter vs. Moment
Near Flange Strain, Quarter (Microstrain)
Moment (ft*kips)



4ES-1.0-1.0-24 Far Flange Strain, Quarter vs. Moment
Near Flange Strain, Quarter (Microstrain)
Moment (ft*kips)

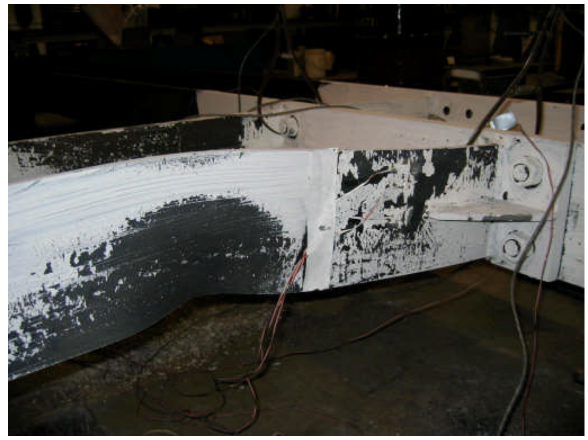


4ES-1.0-1.0-24 Near Flange Strain, Middle vs. Moment
Near Flange Strain, Middle (Microstrain)
Moment (ft*kips)



4ES-1.0-1.0-24 Far Flange Strain, Middle vs. Moment
Near Flange Strain, Middle (Microstrain)
Moment (ft*kips)

Near Flange



Far Flange

Top View

4ES-1.0-1.0-24 AFTER TESTING

APPENDIX D

MRE1/3 – 0.75 – 0.75 – 30

RESULTS AND TEST DATA

SPECIMEN PROPERTIES & TEST SUMMARY

TEST NAME: MRE1/3-0.75-0.75-30
TEST DATE: December 1, 2003

BEAM DATA

DEPTH, d:	30.00	in
FLANGE WIDTH, b_f:	6.00	in
FLANGE THICKNESS, t_f:	0.39	in
WEB THICKNESS, t_w:	0.25	in
FLANGE YIELD STRESS, F_{yf}:	58.5	ksi
FLANGE ULTIMATE STRESS, F_{uf}:	88.1	ksi
WEB YIELD STRESS, F_{yw}:	66.0	ksi
WEB ULTIMATE STRESS, F_{uw}:	81.2	ksi

END-PLATE DATA

END-PLATE THICKNESS, t_p:	0.75	in
END-PLATE WIDTH, b_p:	6.01	in
END-PLATE LENGTH, L_p:	38.00	in
END-PLATE EXTENSION OUTSIDE FLANGE, p_{ext}:	4.00	in
OUTER PITCH, BOLT TO FLANGE, p_{fo}:	2.01	in
INNER PITCH, BOLT TO FLANGE, p_{fi}:	2.05	in
PITCH, BOLT TO BOLT, p_b:	3.00	in
GAGE, g:	2.99	in
END-PLATE YIELD STRESS, F_{yp}:	53.6	ksi
END-PLATE ULTIMATE STRESS, F_{up}:	80.7	ksi

BOLT DATA

BOLT DIAMETER, d_b:	0.75	in
BOLT LENGTH, L_b:	2.5	in
BOLT TYPE:	A325	
NOMINAL BOLT TENSILE STRESS (AISC J3.6), F_t:	90	ksi
NOMINAL BOLT TENSILE STRENGTH, P_t:	40	kips
BOLT PRETENSION, T_b:	28	kips

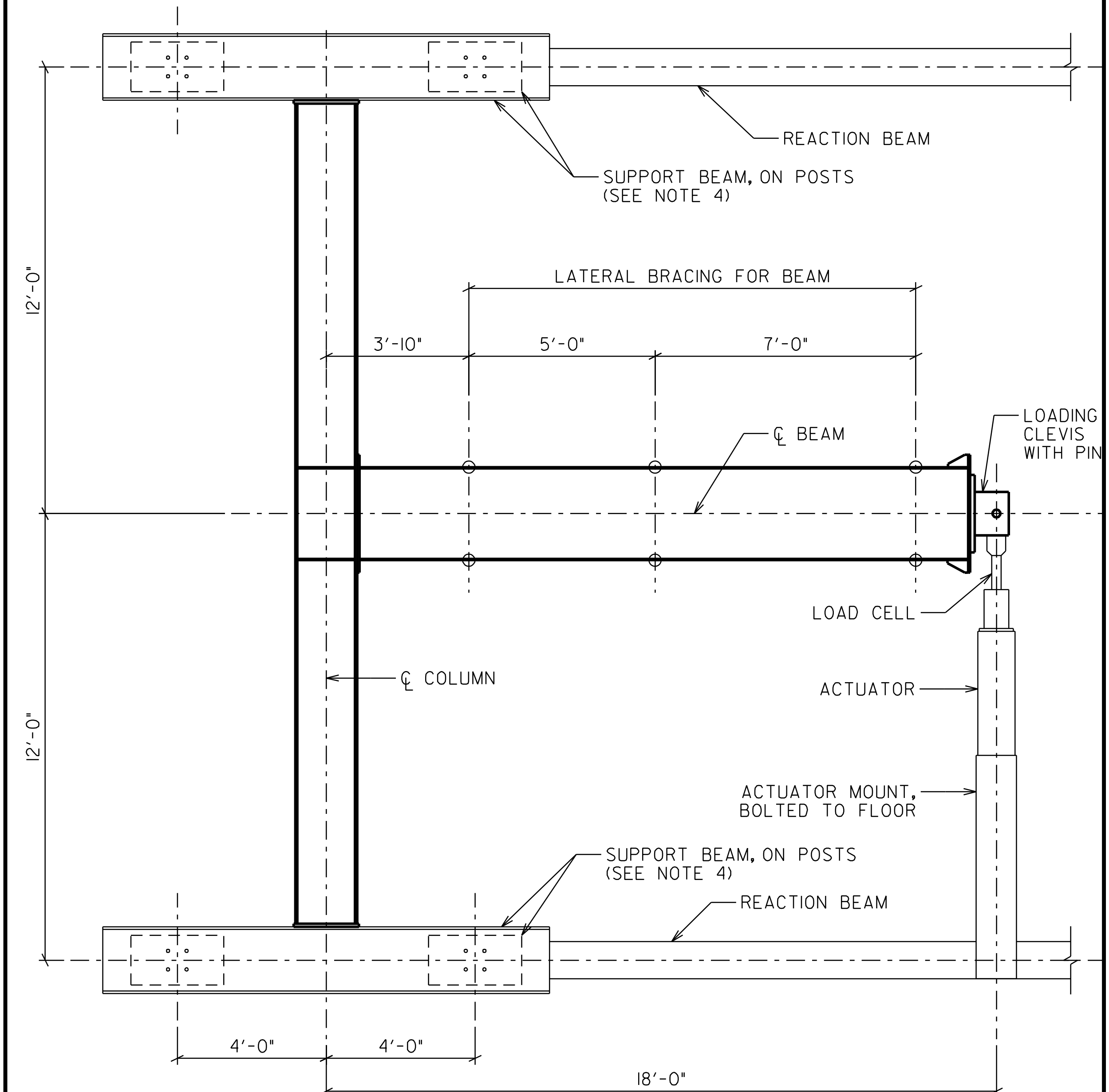
EXPERIMENTAL RESULTS

FAILURE MODE:	Flange Local Buckling	
MAXIMUM MOMENT AT COLUMN CENTERLINE, $M_{max,cc}$:	517.4	ft-kips
MAXIMUM MOMENT AT FAYING SURFACE, $M_{max,fs}$:	493.4	ft-kips
MAXIMUM INTERSTORY DRIFT ANGLE, θ_{max}:	0.040	rad
MAXIMUM INTERSTORY DRIFT ANGLE AT 80% 0F $M_{max,cc}$, θ_{preq}:	0.028	rad

CALCULATED STRENGTHS

BEAM MOMENT STRENGTH [1], M_b:	542.6	ft-kips
BEAM PLASTIC STRENGTH [1], M_p:	631.3	ft-kips
BEAM EXPECTED PLASTIC STRENGTH, M_{pe}:	680.2	ft-kips
END-PLATE STRENGTH [1], M_{PL}:	733.0	ft-kips
BOLT TENSION RUPTURE (w/o Prying, using F_t), M_{NP}:	699.4	ft-kips

1. Measured material properties used.



MREI/3-0.75-0.75-30 TEST SUBASSEMBLAGE PLAN

BEAM

WEB = 1/4×29 1/4
FLANGE = 3/8×6
LENGTH = 197 1/2 (INCLUDES ENDPLATES)

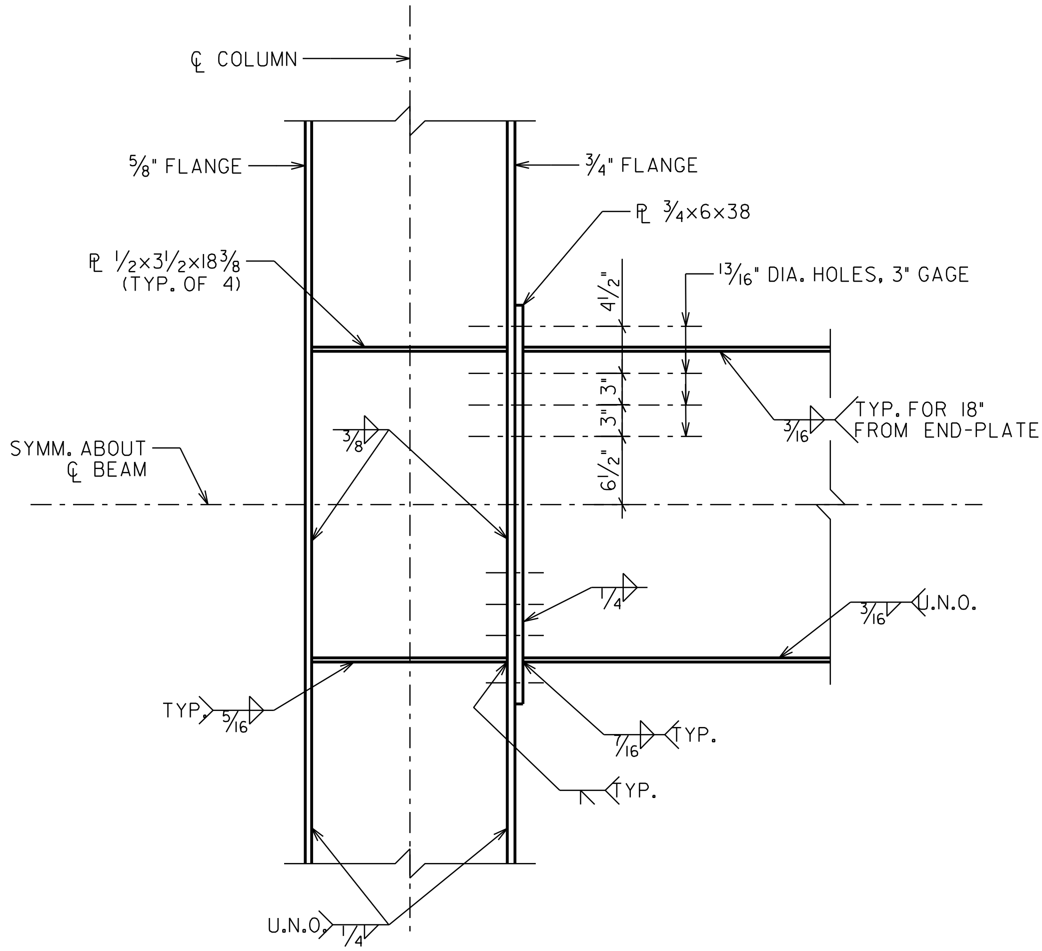
COLUMN

WEB = 1/2×18 5/8
FLANGES = 3/4×8 & 5/8×8
LENGTH = 266 1/2 (INCLUDES ENDPLATES)

NOTES

1. COLUMN, BEAM, AND CONNECTION STEEL, A572 Gr55.
2. ALL BOLTS, A325.
3. ALL WELDING, AWS D1.1, E70XX ELECTRODES.
4. COLUMN BOLTED TO SUPPORT BEAM. SUPPORT BEAM BOLTED POSTS. POSTS BOLTED TO REACTION BEAM.

SPECIMEN: MREI/3-0.75-0.75-30	DRAWN BY: SEB



MREI/3-0.75-0.75-30
CONNECTION DETAIL

SPECIMEN: MREI/3-0.75-0.75-30	DRAWN BY: SEB

SAC PROTOCOL LOADING HISTORY

Test Name: MRE1/3-0.75-0.75-30 Test By: SEB Date: 12/1/2003

Load Step	Cycle	Max. Pos. Load (kips)	Max. Disp. (in)	Max. Neg. Load (kips)	Max. Disp. (in)	Comments
I (0.00375 rad)	1	7.9	0.835	-7.6	-0.807	
δ=0.81"	2	7.9	0.813	-7.6	-0.816	
	3	7.8	0.802	-7.6	-0.810	
	4	8.1	0.815	-7.6	-0.810	
	5	8.0	0.816	-7.7	-0.819	
	6	8.0	0.813	-7.6	-0.810	
		Permanent Set =	-0.012"			
II (0.005 rad)	1	10.5	1.089	-10.2	-1.083	
δ=1.08"	2	10.7	1.089	-10.1	-1.082	
	3	10.5	1.076	-10.1	-1.085	
	4	10.7	1.098	-10.1	-1.086	
	5	10.6	1.087	-10.3	-1.105	
	6	10.7	1.091	-10.1	-1.086	
		Permanent Set =	-0.020"			
III (0.0075 rad)	1	15.3	1.636	-15.6	-1.692	
δ=1.62"	2	15.5	1.621	-15.1	-1.634	
	3	15.5	1.621	-15.8	-1.718	
	4	15.6	1.613	-14.9	-1.619	
	5	15.6	1.615	-15.0	-1.620	
	6	15.7	1.627	-15.0	-1.622	
		Permanent Set =	-0.054"			
IV (0.01 rad)	1	20.0	2.161	-19.9	-2.179	
δ=2.16"	2	20.4	2.167	-20.0	-2.177	
	3	20.3	2.157	-20.0	-2.178	
	4	20.3	2.155	-19.9	-2.157	
		Permanent Set =	-0.068"			
V (0.015 rad)	1	27.3	3.240	-26.8	-3.277	Slight whitewash flaking on flanges
δ=3.24"	2	27.6	3.255	-26.5	-3.276	
		Permanent Set =	-0.364"			
VI (0.02 rad)	1	26.9	4.321	-23.7	-4.362	Flange local buckling
δ=4.32"	2	25.5	4.337	-22.9	-4.345	
		Permanent Set =	-1.421"			
VII (0.03 rad)	1	22.6	6.479	-19.7	-6.485	Flange and web local buckling
δ=6.48"	2	21.6	6.470	-18.8	-6.477	
		Permanent Set =	-3.182"			
VIII (0.04 rad)	1	17.6	8.682	-14.8	-8.640	Test stopped due to
δ=8.64"	2	16.7	8.704	-13.7	-8.670	50% reduction in capacity
		Permanent Set =	-4.401"			
IX (0.05 rad)	1					
	2					
		Permanent Set =				

CALCULATED STRENGTHS

TEST NAME: MRE1/3-0.75-0.75-30
TEST DATE: December 1, 2003

Specified Grade for all Steel = ASTM A572, Grade 55

MEASURED PROPERTIES

$d := 30 \cdot in$	$F_{yf} := 58.5 \cdot ksi$	$t_p := 0.75 \cdot in$	$p_{fo} := 2.01 \cdot in$	$p_b := 3.00 \cdot in$
$b_f := 6.00 \cdot in$	$F_{uf} := 88.1 \cdot ksi$	$b_p := 6.01 \cdot in$	$p_{fi} := 2.05 \cdot in$	
$t_f := 0.39 \cdot in$	$F_{yw} := 66.0 \cdot ksi$	$L_p := 38.00 \cdot in$	$g := 2.99 \cdot in$	
$t_w := 0.25 \cdot in$	$F_{uw} := 81.2 \cdot ksi$	$p_{ext} := 4.00 \cdot in$	$F_{yp} := 53.6 \cdot ksi$	$h := d - 2 \cdot t_f$

SPECIFIED PROPERTIES

$R_y := 1.1$ $\quad P_t := 40 \cdot k$

BEAM PLASTIC STRENGTH, Mp

$$M_p := 2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right) \cdot F_{yf} + 2 \cdot t_w \cdot \frac{h}{2} \cdot \frac{h}{4} \cdot F_{yw} \qquad M_p = 631.3 k \cdot ft$$

BEAM EXPECTED PLASTIC STRENGTH, Mpe

$$M_{pe} := 1.1 \cdot R_y \cdot \left[2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right) \cdot 55 \cdot ksi + 2 \cdot t_w \cdot \frac{h}{2} \cdot \frac{h}{4} \cdot 55 \cdot ksi\right] \qquad M_{pe} = 680.2 k \cdot ft$$

BEAM MOMENT STRENGTH, Mb

$\frac{b_f}{2 \cdot t_f} = 7.7$ $\quad \frac{65}{\sqrt{55}} = 8.8$ $\quad$ Flange is compact

$\frac{640}{\sqrt{55}} = 86.3$ $\quad \frac{h}{t_w} = 116.9$ $\quad \frac{970}{\sqrt{55}} = 130.8$ $\quad$ Web is non-compact

Therefore, Mb < Mp

$$I_x := 2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right)^2 + \frac{1}{12} \cdot t_w \cdot h^3 \qquad I_x = 1546 in^4 \qquad S_x := \frac{I_x}{0.5 \cdot d} \qquad S_x = 103 in^3$$

$$a_r := \frac{h \cdot t_w}{b_f \cdot t_f} \qquad m := \frac{55}{55} \qquad R_e := \frac{12 + a_r \cdot \left(3 \cdot m - m^3\right)}{12 + 2 \cdot a_r} \qquad R_e = 1$$

BEAM MOMENT STRENGTH, Mb (CONT'D)

$M_r := R_e \cdot S_x \cdot F_{yf}$ $\quad M_r = 502.3 k \cdot ft$

$\lambda_p := \frac{640}{\sqrt{55}}$ $\quad \lambda := \frac{h}{t_w}$ $\quad \lambda_r := \frac{970}{\sqrt{55}}$

$M_b := M_p - (M_p - M_r) \cdot \left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right)$ $\quad M_b = 542.6 k \cdot ft$

END-PLATE STRENGTH, Mpl

$d_0 := d - \frac{t_f}{2} + p_{fo}$ $\quad d_0 = 31.8 in$ $\quad h_0 := d_0 + \frac{t_f}{2}$ $\quad h_0 = 32 in$

$d_1 := d - \frac{t_f}{2} - t_f - p_{fi}$ $\quad d_1 = 27.4 in$ $\quad h_1 := d_1 + \frac{t_f}{2}$ $\quad h_1 = 27.6 in$

$d_2 := d_1 - p_b$ $\quad d_2 = 24.4 in$ $\quad h_2 := d_2 + \frac{t_f}{2}$ $\quad h_2 = 24.6 in$

$d_3 := d_2 - p_b$ $\quad d_3 = 21.4 in$ $\quad h_3 := d_3 + \frac{t_f}{2}$ $\quad h_3 = 21.6 in$

$s := \frac{1}{2} \cdot \sqrt{b_p \cdot g}$ $\quad s = 2.12 in$

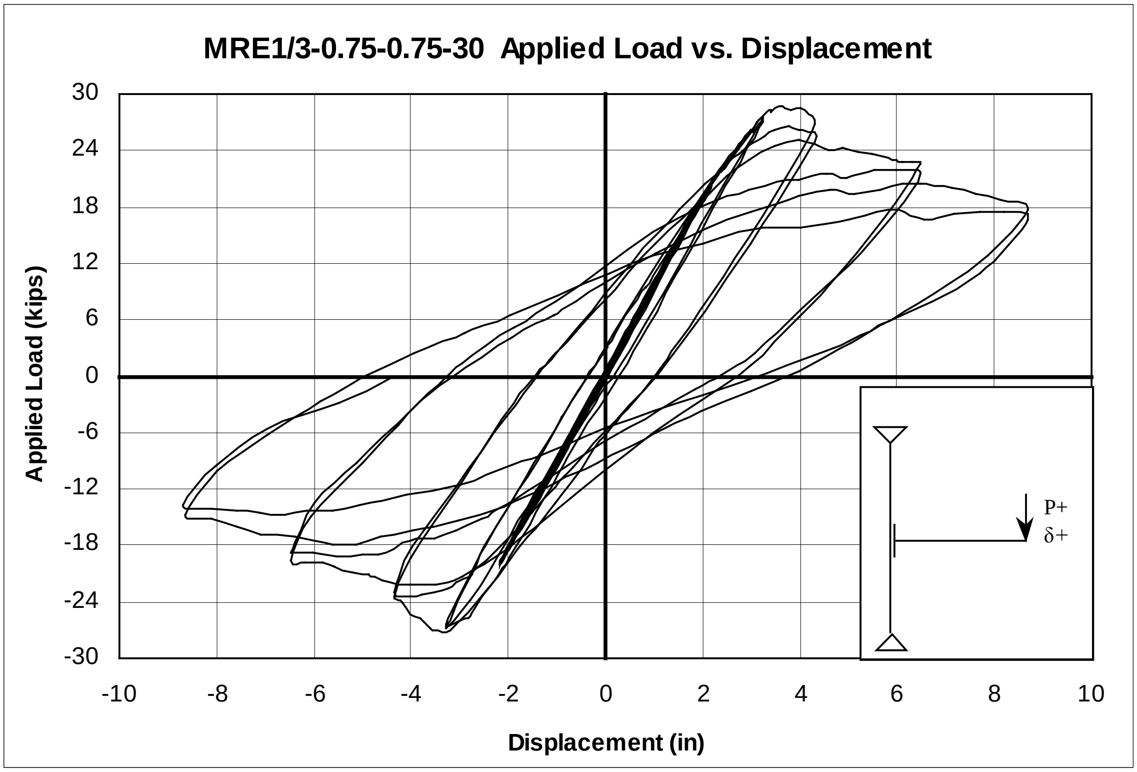
$$Y := \frac{b_p}{2} \cdot \left(\frac{h_1}{p_{fi}} + \frac{h_3}{s} + \frac{h_0}{p_{fo}} - \frac{1}{2} \right) + \frac{2}{g} \cdot \left[h_1 \cdot (p_{fi} + 1.5 \cdot p_b) + h_3 \cdot (s + 0.5 \cdot p_b) \right] + \frac{g}{2}$$

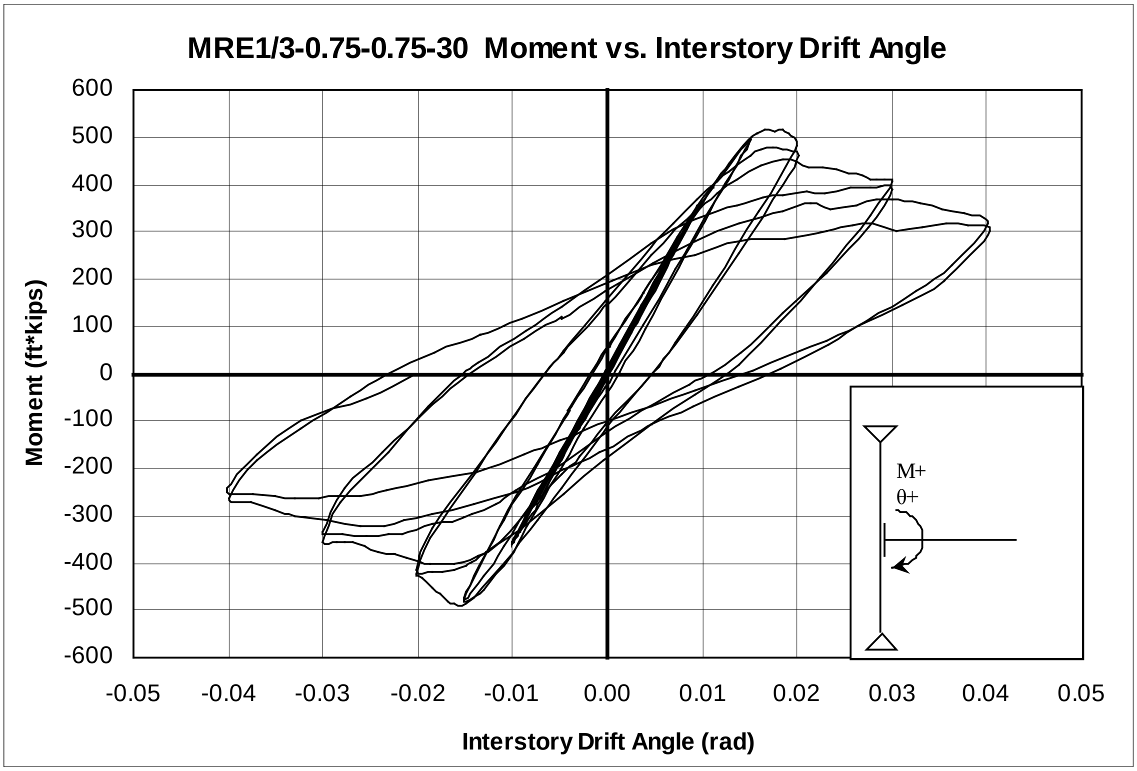
$Y = 291.8 in$

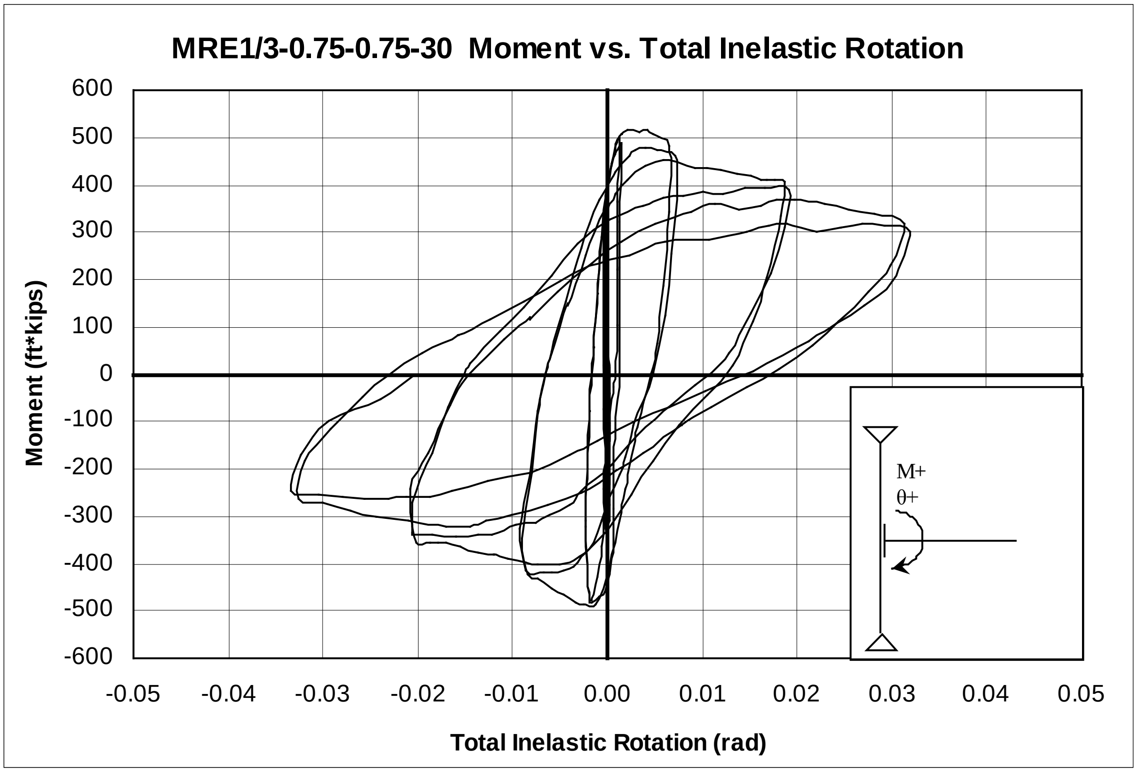
$M_{PL} := t_p^{\,2} \cdot F_{yp} \cdot Y$ $\quad M_{PL} = 733 k \cdot ft$

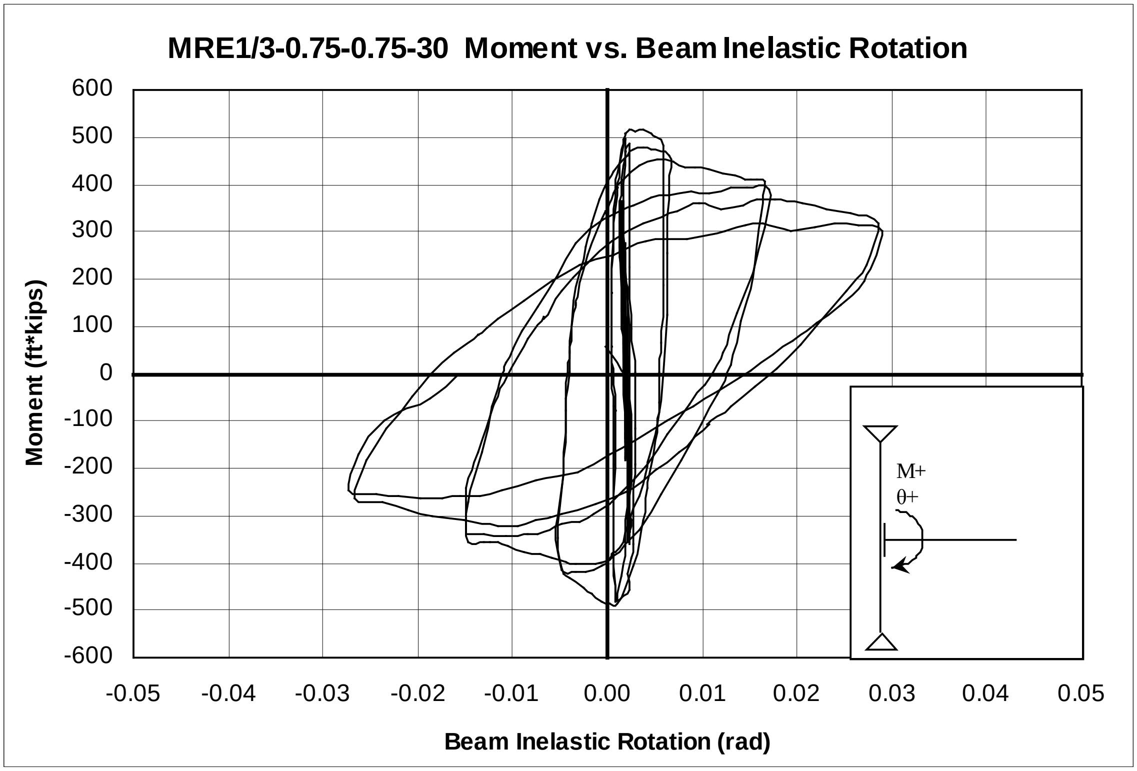
BOLT STRENGTH WITHOUT PRYING, Mnp

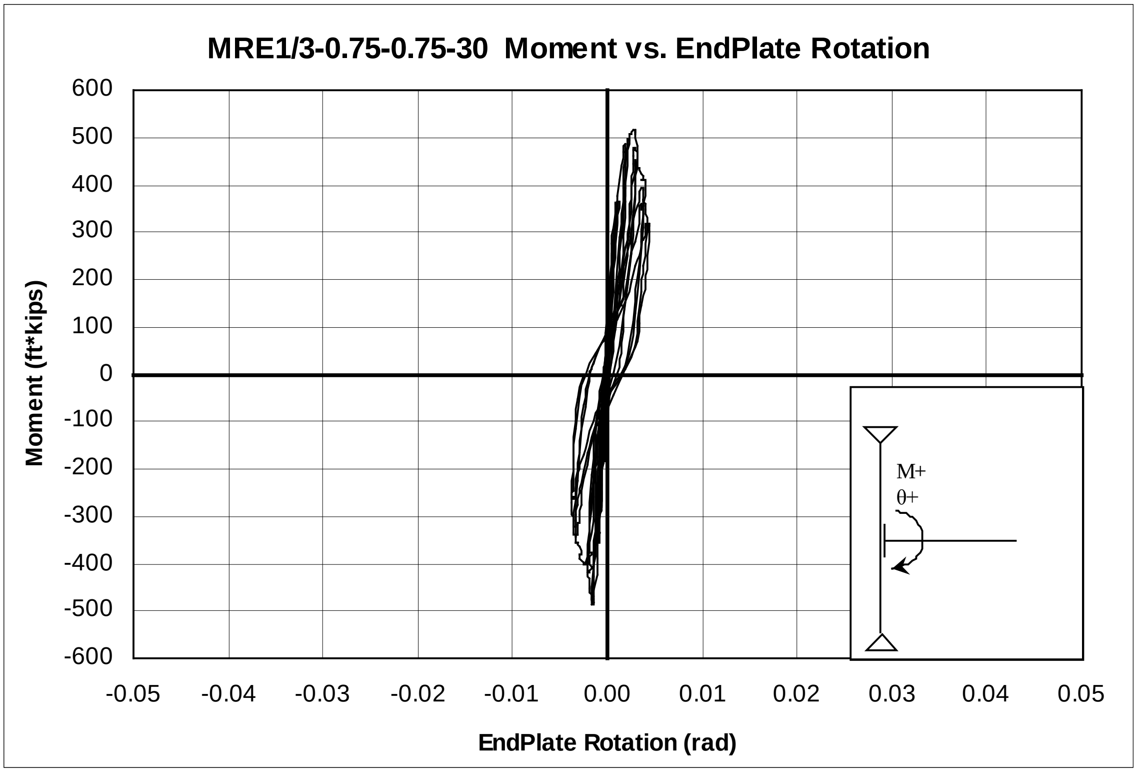
$M_{NP} := 2 \cdot P_t \cdot (d_0 + d_1 + d_2 + d_3)$ $\quad M_{NP} = 699.4 k \cdot ft$

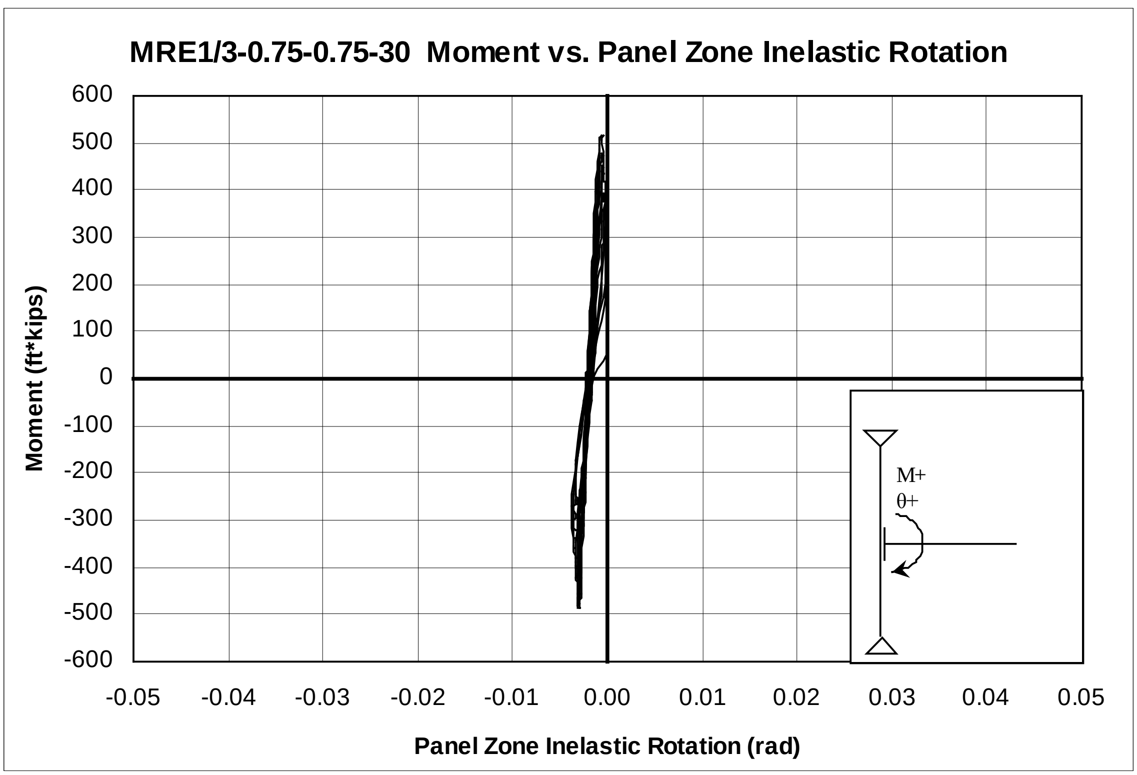

MRE1/3-0.75-0.75-30 Applied Load vs. Displacement
Applied Load (kips)
Displacement (in)
30
24
18
12
6
0
-6
-12
-18
-24
-30
-10
-8
-6
-4
-2
0
2
4
6
8
10
P+
δ+

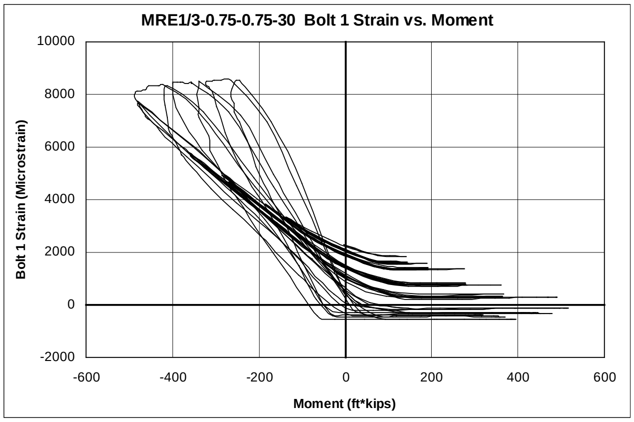
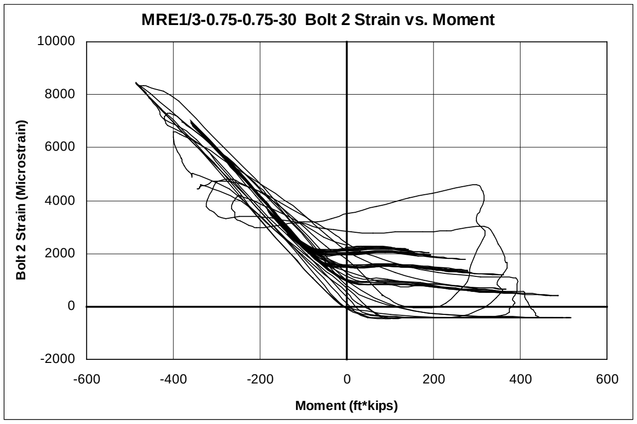
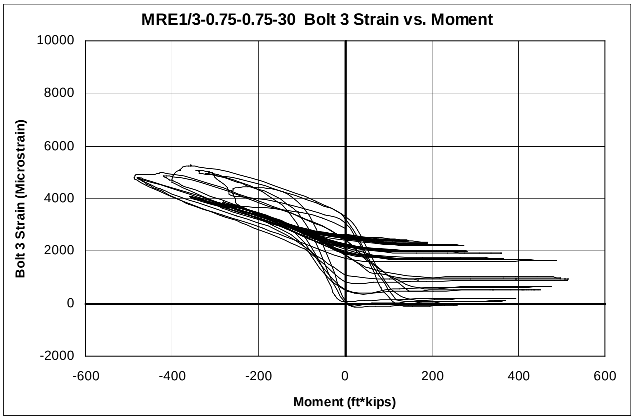
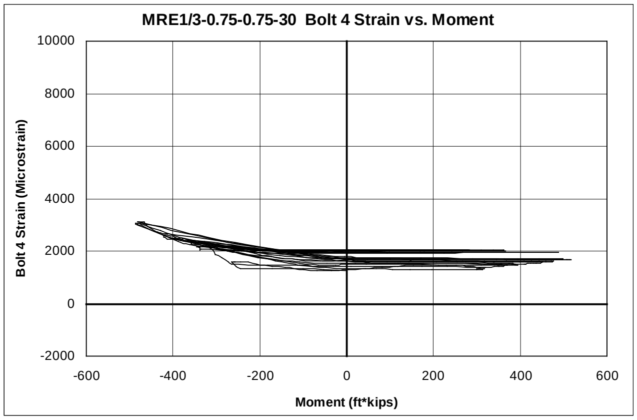
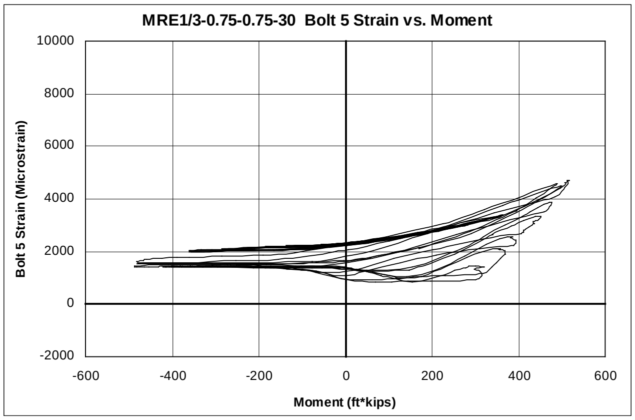
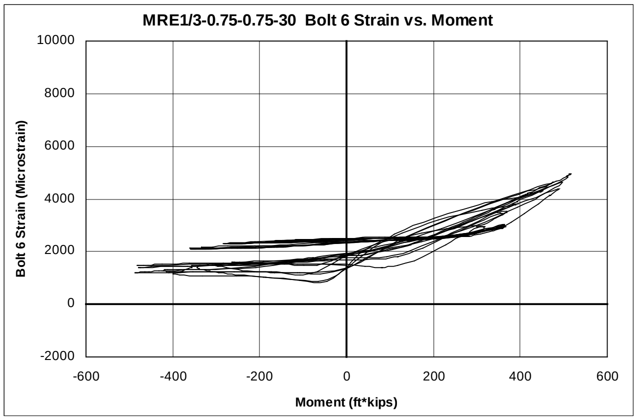
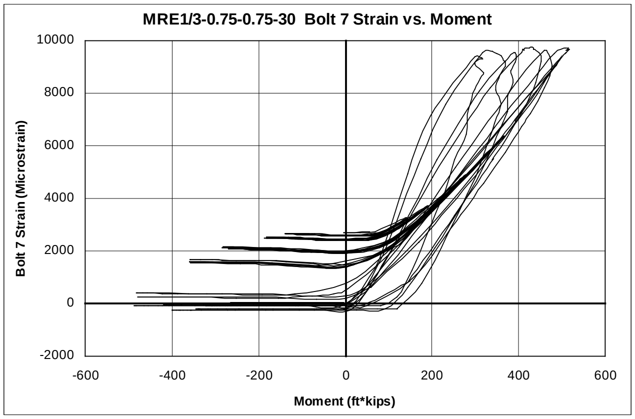
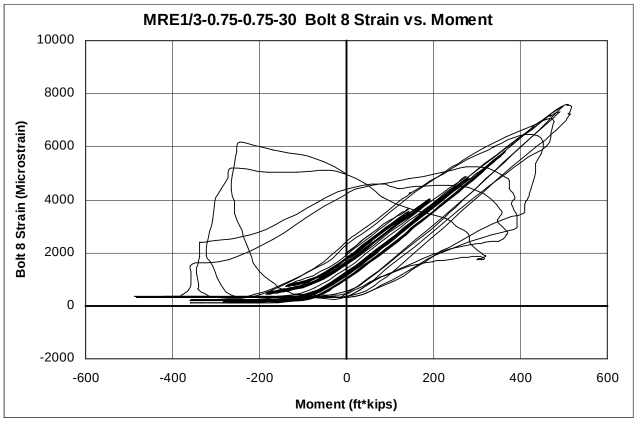

MRE1/3-0.75-0.75-30 Moment vs. Interstory Drift Angle
Moment (ft*kips)
Interstory Drift Angle (rad)
600
500
400
300
200
100
0
-100
-200
-300
-400
-500
-600
-0.05
-0.04
-0.03
-0.02
-0.01
0.00
0.01
0.02
0.03
0.04
0.05
M+
θ+

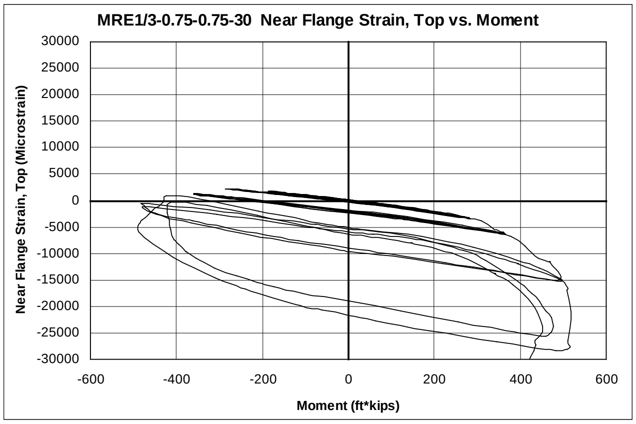

MRE1/3-0.75-0.75-30 Moment vs. Total Inelastic Rotation
Moment (ft*kips)
600
500
400
300
200
100
0
-100
-200
-300
-400
-500
-600
-0.05
-0.04
-0.03
-0.02
-0.01
0.00
0.01
0.02
0.03
0.04
0.05
Total Inelastic Rotation (rad)
M+
θ+

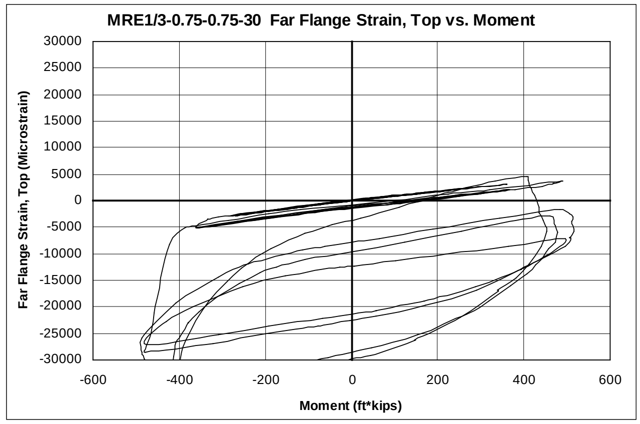

MRE1/3-0.75-0.75-30 Moment vs. Beam Inelastic Rotation
Moment (ft*kips)
600
500
400
300
200
100
0
-100
-200
-300
-400
-500
-600
-0.05
-0.04
-0.03
-0.02
-0.01
0.00
0.01
0.02
0.03
0.04
0.05
Beam Inelastic Rotation (rad)
M+
θ+

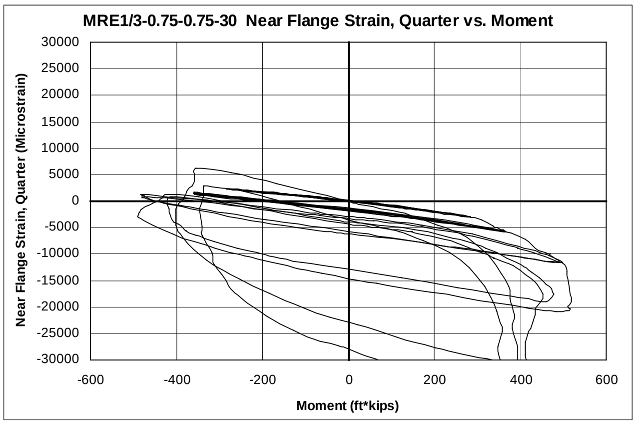

MRE1/3-0.75-0.75-30 Moment vs. EndPlate Rotation
Moment (ft*kips)
600
500
400
300
200
100
0
-100
-200
-300
-400
-500
-600
-0.05
-0.04
-0.03
-0.02
-0.01
0.00
0.01
0.02
0.03
0.04
0.05
EndPlate Rotation (rad)
M+
θ+

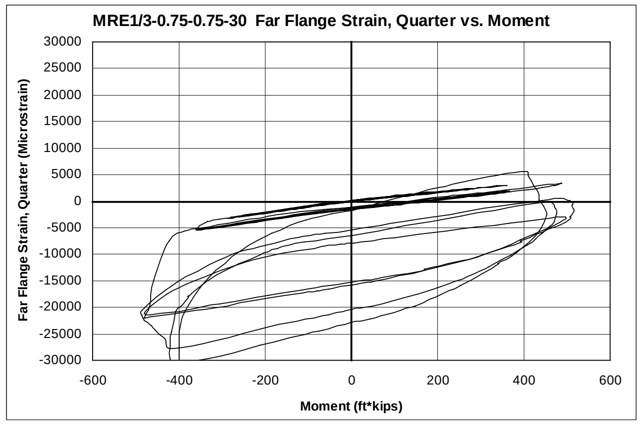

MRE1/3-0.75-0.75-30 Moment vs. Panel Zone Inelastic Rotation
Moment (ft*kips)
600
500
400
300
200
100
0
-100
-200
-300
-400
-500
-600
-0.05
-0.04
-0.03
-0.02
-0.01
0.00
0.01
0.02
0.03
0.04
0.05
Panel Zone Inelastic Rotation (rad)
M+
θ+

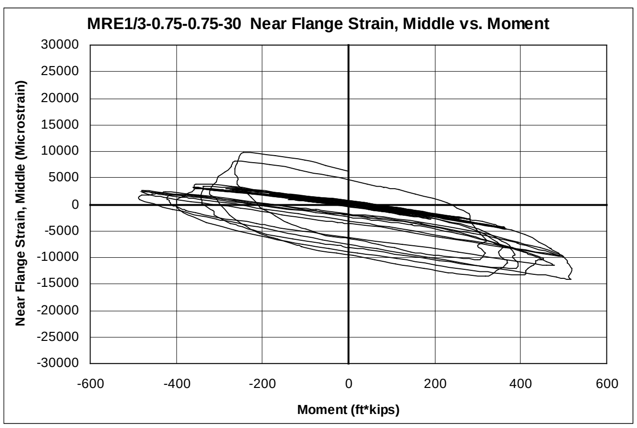

MRE1/3-0.75-0.75-30 Bolt 1 Strain vs. Moment
Bolt 1 Strain (Microstrain)
Moment (ft*kips)

MRE1/3-0.75-0.75-30 Bolt 2 Strain vs. Moment
Bolt 2 Strain (Microstrain)
Moment (ft*kips)

MRE1/3-0.75-0.75-30 Bolt 3 Strain vs. Moment
Bolt 3 Strain (Microstrain)
Moment (ft*kips)

MRE1/3-0.75-0.75-30 Bolt 4 Strain vs. Moment
Bolt 4 Strain (Microstrain)
Moment (ft*kips)

MRE1/3-0.75-0.75-30 Bolt 5 Strain vs. Moment
Bolt 5 Strain (Microstrain)
Moment (ft*kips)

MRE1/3-0.75-0.75-30 Bolt 6 Strain vs. Moment
Bolt 6 Strain (Microstrain)
Moment (ft*kips)

MRE1/3-0.75-0.75-30 Bolt 7 Strain vs. Moment
Bolt 7 Strain (Microstrain)
Moment (ft*kips)

MRE1/3-0.75-0.75-30 Bolt 8 Strain vs. Moment
Bolt 8 Strain (Microstrain)
Moment (ft*kips)

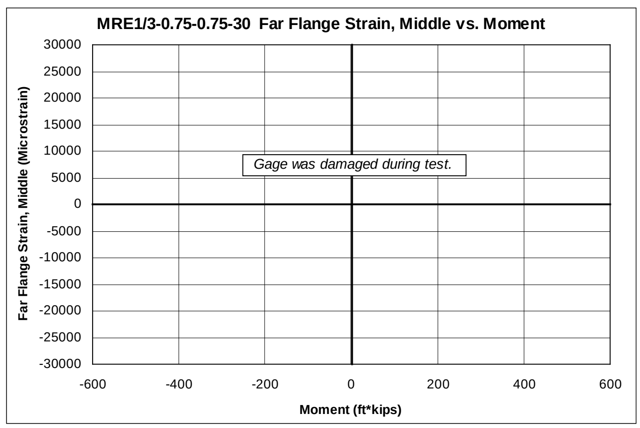

MRE1/3-0.75-0.75-30 Near Flange Strain, Top vs. Moment
Near Flange Strain, Top (Microstrain)
Moment (ft*kips)


MRE1/3-0.75-0.75-30 Far Flange Strain, Top vs. Moment
Far Flange Strain, Top (Microstrain)
Moment (ft*kips)

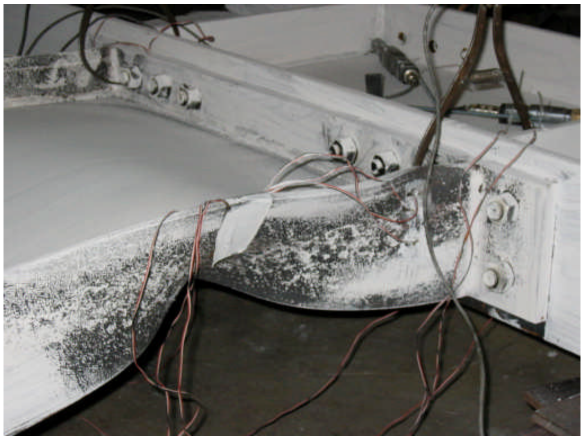

MRE1/3-0.75-0.75-30 Near Flange Strain, Quarter vs. Moment
Near Flange Strain, Quarter (Microstrain)
Moment (ft*kips)


MRE1/3-0.75-0.75-30 Far Flange Strain, Quarter vs. Moment
Far Flange Strain, Quarter (Microstrain)
Moment (ft*kips)

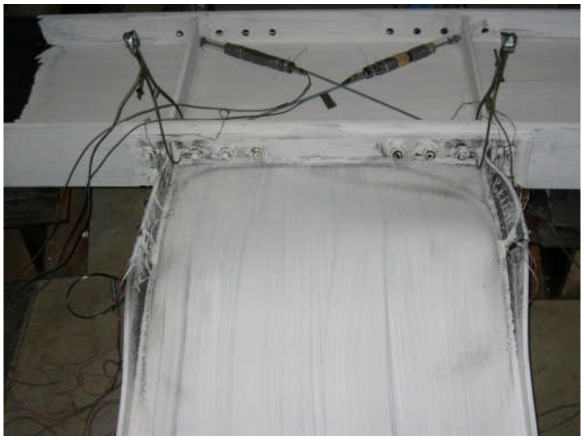

MRE1/3-0.75-0.75-30 Near Flange Strain, Middle vs. Moment
Near Flange Strain, Middle (Microstrain)
Moment (ft*kips)


MRE1/3-0.75-0.75-30 Far Flange Strain, Middle vs. Moment
Far Flange Strain, Middle (Microstrain)
Gage was damaged during test.
Moment (ft*kips)

Near Flange



Far Flange



Top View

MRE1/3-0.75-0.75-30 AFTER TESTING

APPENDIX E

MRE1/3S – 0.75 – 0.625 – 30

RESULTS AND TEST DATA

SPECIMEN PROPERTIES & TEST SUMMARY

TEST NAME: MRE1/3S-0.75-0.625-30
TEST DATE: December 10, 2003

BEAM DATA

DEPTH, d:	30.00	in
FLANGE WIDTH, b_f:	6.00	in
FLANGE THICKNESS, t_f:	0.39	in
WEB THICKNESS, t_w:	0.25	in
FLANGE YIELD STRESS, F_{yf}:	58.5	ksi
FLANGE ULTIMATE STRESS, F_{uf}:	88.1	ksi
WEB YIELD STRESS, F_{yw}:	66.0	ksi
WEB ULTIMATE STRESS, F_{uw}:	81.2	ksi

END-PLATE DATA

END-PLATE THICKNESS, t_p:	0.64	in
END-PLATE WIDTH, b_p:	6.02	in
END-PLATE LENGTH, L_p:	38.00	in
END-PLATE EXTENSION OUTSIDE FLANGE, p_{ext}:	4.00	in
OUTER PITCH, BOLT TO FLANGE, p_{fo}:	1.92	in
INNER PITCH, BOLT TO FLANGE, p_{fi}:	2.16	in
PITCH, BOLT TO BOLT, p_b:	3.00	in
GAGE, g:	3.01	in
END-PLATE YIELD STRESS, F_{yp}:	67.2	ksi
END-PLATE ULTIMATE STRESS, F_{up}:	93.0	ksi

BOLT DATA

BOLT DIAMETER, d_b:	0.75	in
BOLT LENGTH, L_b:	2.25	in
BOLT TYPE:	A325	
NOMINAL BOLT TENSILE STRESS (AISC J3.6), F_t:	90	ksi
NOMINAL BOLT TENSILE STRENGTH, P_t:	40	kips
BOLT PRETENSION, T_b:	28	kips

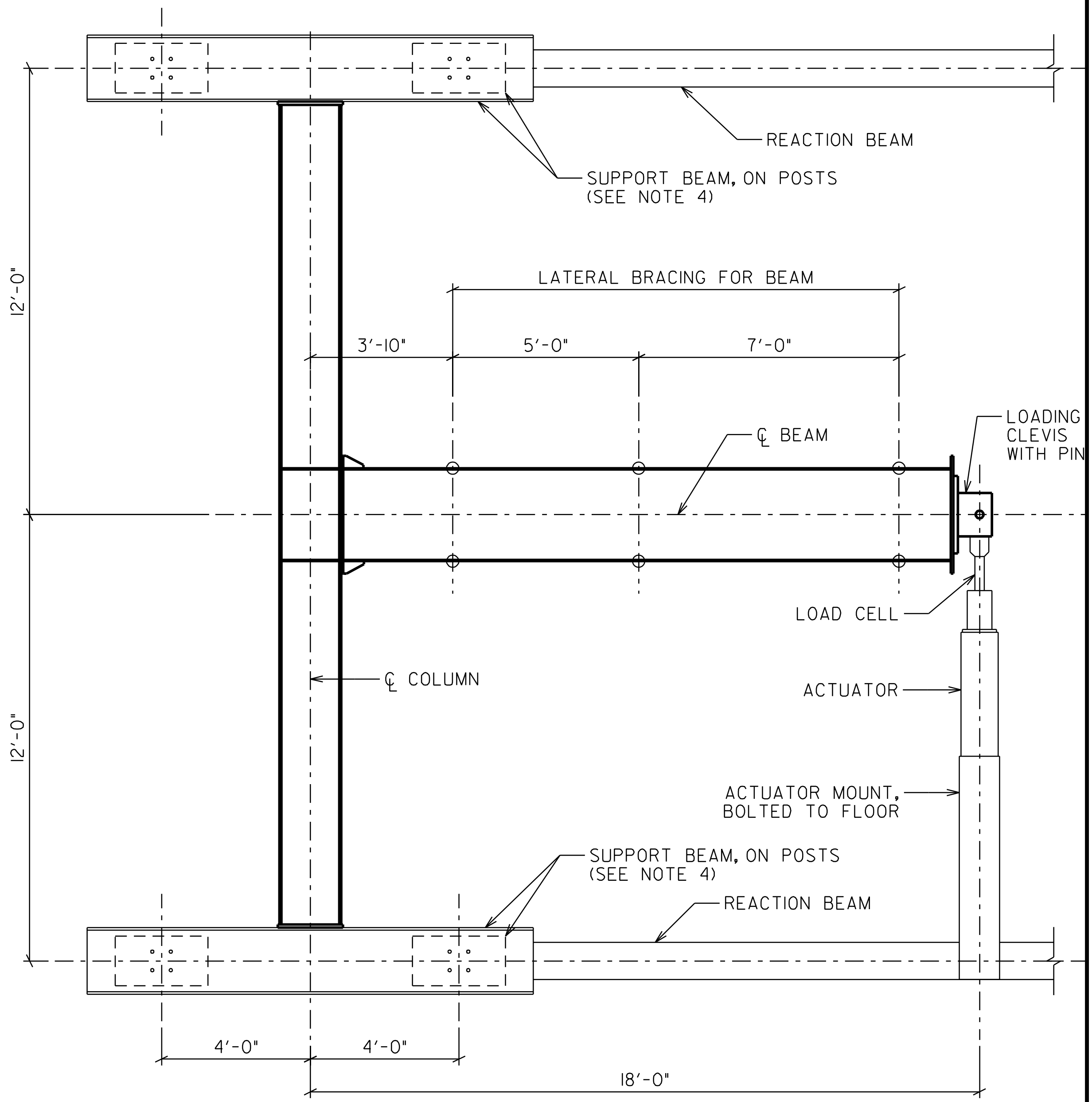
EXPERIMENTAL RESULTS

FAILURE MODE:	Flange Local Buckling	
MAXIMUM MOMENT AT COLUMN CENTERLINE, $M_{max,cc}$:	521.5	ft-kips
MAXIMUM MOMENT AT FAYING SURFACE, $M_{max,fs}$:	497.4	ft-kips
MAXIMUM INTERSTORY DRIFT ANGLE, θ_{max}:	0.040	rad
MAXIMUM INTERSTORY DRIFT ANGLE AT 80% 0F $M_{max,cc}$, θ_{preq}:	0.024	rad

CALCULATED STRENGTHS

BEAM MOMENT STRENGTH [1], M_b:	542.6	ft-kips
BEAM PLASTIC STRENGTH [1], M_p:	631.3	ft-kips
BEAM EXPECTED PLASTIC STRENGTH, M_{pe}:	680.2	ft-kips
END-PLATE STRENGTH [1], M_{PL}:	919.2	ft-kips
BOLT TENSION RUPTURE (w/o Prying, using F_t), M_{NP}:	696.6	ft-kips

1. Measured material properties used.



MREI/3S-0.75-0.625-30 TEST SUBASSEMBLAGE PLAN

BEAM

WEB = $\frac{1}{4}$×29$\frac{1}{4}$
FLANGE = $\frac{3}{8}$×6
LENGTH = 197$\frac{1}{2}$ (INCLUDES ENDPLATES)

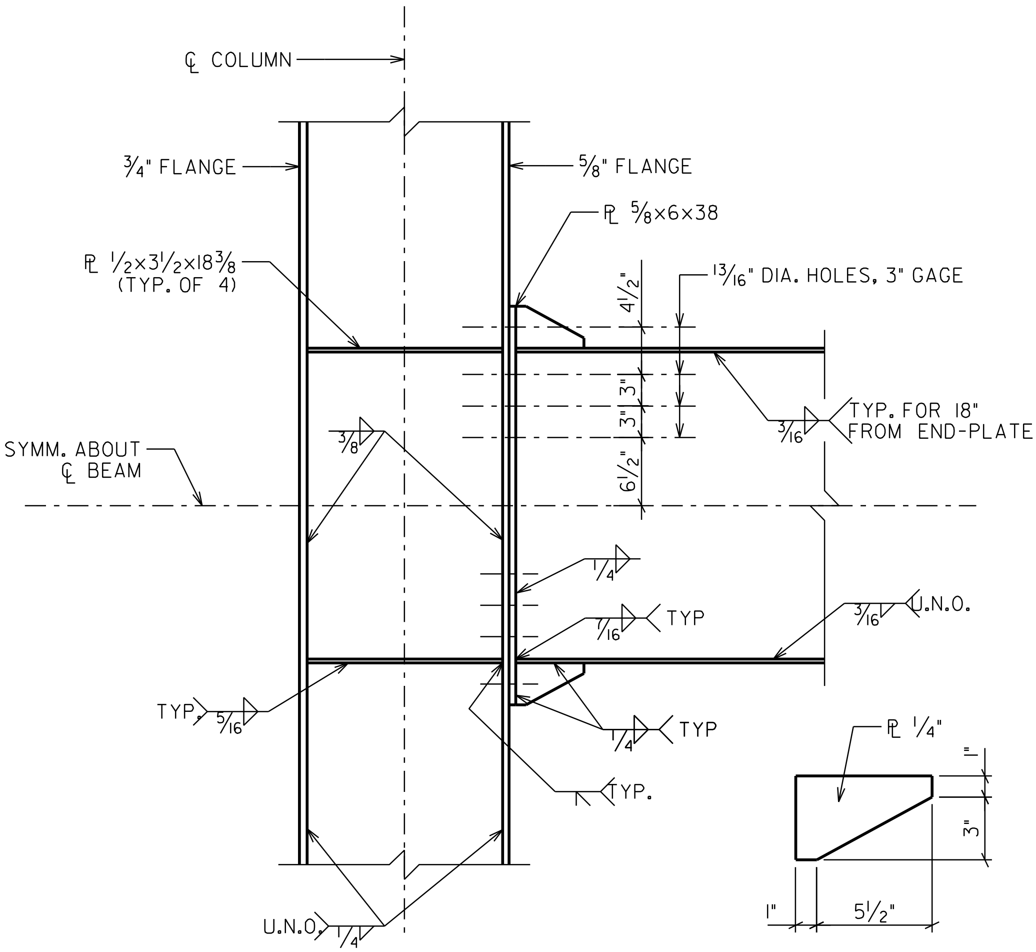
COLUMN

WEB = $\frac{1}{2}$×18$\frac{5}{8}$
FLANGES = $\frac{3}{4}$×8 & $\frac{5}{8}$×8
LENGTH = 266$\frac{1}{2}$ (INCLUDES ENDPLATES)

NOTES

1. COLUMN, BEAM, AND CONNECTION STEEL, A572 Gr55.
2. ALL BOLTS, A325.
3. ALL WELDING, AWS D1.1, E70XX ELECTRODES.
4. COLUMN BOLTED TO SUPPORT BEAM. SUPPORT BEAM BOLTED POSTS. POSTS BOLTED TO REACTION BEAM.

SPECIMEN: MREI/3S-0.75-0.625-30	DRAWN BY: SEB



MREI/3S-0.75-0.625-30
CONNECTION DETAIL

STIFFENER

SPECIMEN: MREI/3S-0.75-0.625-30	DRAWN BY: SEB

SAC PROTOCOL LOADING HISTORY

Test Name: MRE1/3S-0.75-0.625-30 Test By: SEB Date: 12/10/2003

Load Step	Cycle	Max. Pos. Load (kips)	Max. Disp. (in)	Max. Neg. Load (kips)	Max. Disp. (in)	Comments
I (0.00375 rad)	1	7.6	0.828	-8.0	-0.819	
δ=0.81"	2	7.5	0.818	-8.2	-0.835	
	3	7.6	0.845	-8.2	-0.839	
	4	7.5	0.817	-7.9	-0.813	
	5	7.5	0.816	-7.9	-0.812	
	6	7.6	0.826	-8.0	-0.813	
		Permanent Set =	-0.003"			
II (0.005 rad)	1	9.7	1.093	-10.5	-1.084	
δ=1.08"	2	9.8	1.103	-10.6	-1.080	
	3	9.6	1.082	-10.4	-1.072	
	4	9.6	1.089	-10.5	-1.074	
	5	9.6	1.092	-10.7	-1.095	
	6	9.6	1.089	-10.6	-1.082	
		Permanent Set =	-0.005"			
III (0.0075 rad)	1	13.7	1.621	-16.1	-1.620	
δ=1.62"	2	14.0	1.623	-16.4	-1.647	
	3	14.0	1.648	-16.2	-1.624	
	4	14.1	1.648	-16.3	-1.621	
	5	13.9	1.624	-16.1	-1.622	
	6	14.1	1.646	-16.5	-1.636	
		Permanent Set =	-0.005"			
IV (0.01 rad)	1	18.3	2.163	-21.6	-2.196	
δ=2.16"	2	18.4	2.162	-21.4	-2.164	
	3	18.4	2.159	-21.4	-2.171	
	4	18.5	2.169	-20.9	-2.167	
		Permanent Set =	-0.025"			
V (0.015 rad)	1	26.1	3.258	-28.2	-3.242	Slight whitewash flaking
δ=3.24"	2	26.0	3.242	-28.0	-3.241	
		Permanent Set =	-0.310"			
VI (0.02 rad)	1	26.7	4.326	-28.3	-4.349	Flange local buckling
δ=4.32"	2	25.3	4.342	-26.6	-4.348	
		Permanent Set =	-1.255"			
VII (0.03 rad)	1	20.8	6.481	-21.9	-6.476	Flange and web local buckling
δ=6.48"	2	18.1	6.489	-20.5	-6.529	
		Permanent Set =	-2.660"			
VIII (0.04 rad)	1	13.5	8.679	-16.4	-7.144	Test Stopped, due to single bolt rupturing on outside row
δ=8.64"	2					
		Permanent Set =				
IX (0.05 rad)	1					
	2					
		Permanent Set =				

CALCULATED STRENGTHS

TEST NAME: MRE1/3S-0.750.625-30
TEST DATE: December 10, 2003

Specified Grade for all Steel = ASTM A572, Grade 55

MEASURED PROPERTIES

$d := 30 \cdot in$ $F_{yf} := 58.5 \cdot ksi$ $t_p := 0.64 \cdot in$ $p_{fo} := 1.92 \cdot in$ $p_b := 3.00 \cdot in$

$b_f := 6.00 \cdot in$ $F_{uf} := 88.1 \cdot ksi$ $b_p := 6.02 \cdot in$ $p_{fi} := 2.16 \cdot in$

$t_f := 0.39 \cdot in$ $F_{yw} := 66.0 \cdot ksi$ $L_p := 38.00 \cdot in$ $g := 3.01 \cdot in$

$t_w := 0.25 \cdot in$ $F_{uw} := 81.2 \cdot ksi$ $p_{ext} := 4.00 \cdot in$ $F_{yp} := 67.2 \cdot ksi$ $h := d - 2 \cdot t_f$

SPECIFIED PROPERTIES

$R_y := 1.1$ $P_t := 40 \cdot k$

BEAM PLASTIC STRENGTH, Mp

$$M_p := 2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right) \cdot F_{yf} + 2 \cdot t_w \cdot \frac{h}{2} \cdot \frac{h}{4} \cdot F_{yw} \qquad M_p = 631.3 k \cdot ft$$

BEAM EXPECTED PLASTIC STRENGTH, Mpe

$$M_{pe} := 1.1 \cdot R_y \cdot \left[2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right) \cdot 55 \cdot ksi + 2 \cdot t_w \cdot \frac{h}{2} \cdot \frac{h}{4} \cdot 55 \cdot ksi\right] \qquad M_{pe} = 680.2 k \cdot ft$$

BEAM MOMENT STRENGTH, Mb

$\frac{b_f}{2 \cdot t_f} = 7.7$ $\frac{65}{\sqrt{55}} = 8.8$ Flange is compact

$\frac{640}{\sqrt{55}} = 86.3$ $\frac{h}{t_w} = 116.9$ $\frac{970}{\sqrt{55}} = 130.8$ Web is non-compact

Therefore, Mb< Mp

$I_x := 2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right)^2 + \frac{1}{12} \cdot t_w \cdot h^3$ $I_x = 1546 in^4$ $S_x := \frac{I_x}{0.5 \cdot d}$ $S_x = 103 in^3$

$a_r := \frac{h \cdot t_w}{b_f \cdot t_f}$ $m := \frac{55}{55}$ $R_e := \frac{12 + a_r \cdot \left(3 \cdot m - m^3\right)}{12 + 2 \cdot a_r}$ $R_e = 1$

BEAM MOMENT STRENGTH, Mb (CONT'D)

$M_r := R_e \cdot S_x \cdot F_{yf}$ $\qquad M_r = 502.3 k \cdot ft$

$\lambda_p := \frac{640}{\sqrt{55}}$ $\qquad \lambda := \frac{h}{t_w}$ $\qquad \lambda_r := \frac{970}{\sqrt{55}}$

$M_b := M_p - (M_p - M_r) \cdot \left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right)$ $\qquad M_b = 542.6 k \cdot ft$

END-PLATE STRENGTH, Mpl

$d_0 := d - \frac{t_f}{2} + p_{fo}$ $\qquad d_0 = 31.7in$ $\qquad h_0 := d_0 + \frac{t_f}{2}$ $\qquad h_0 = 31.9in$

$d_1 := d - \frac{t_f}{2} - t_f - p_{fi}$ $\qquad d_1 = 27.3in$ $\qquad h_1 := d_1 + \frac{t_f}{2}$ $\qquad h_1 = 27.5in$

$d_2 := d_1 - p_b$ $\qquad d_2 = 24.3in$ $\qquad h_2 := d_2 + \frac{t_f}{2}$ $\qquad h_2 = 24.5in$

$d_3 := d_2 - p_b$ $\qquad d_3 = 21.3in$ $\qquad h_3 := d_3 + \frac{t_f}{2}$ $\qquad h_3 = 21.5in$

$s := \frac{1}{2} \cdot \sqrt{b_p \cdot g}$ $\qquad s = 2.13in$ $\qquad d_e := p_{ext} - p_{fo}$ $\qquad d_e = 2.1in$

d_e< s, therefore Case 2 governs

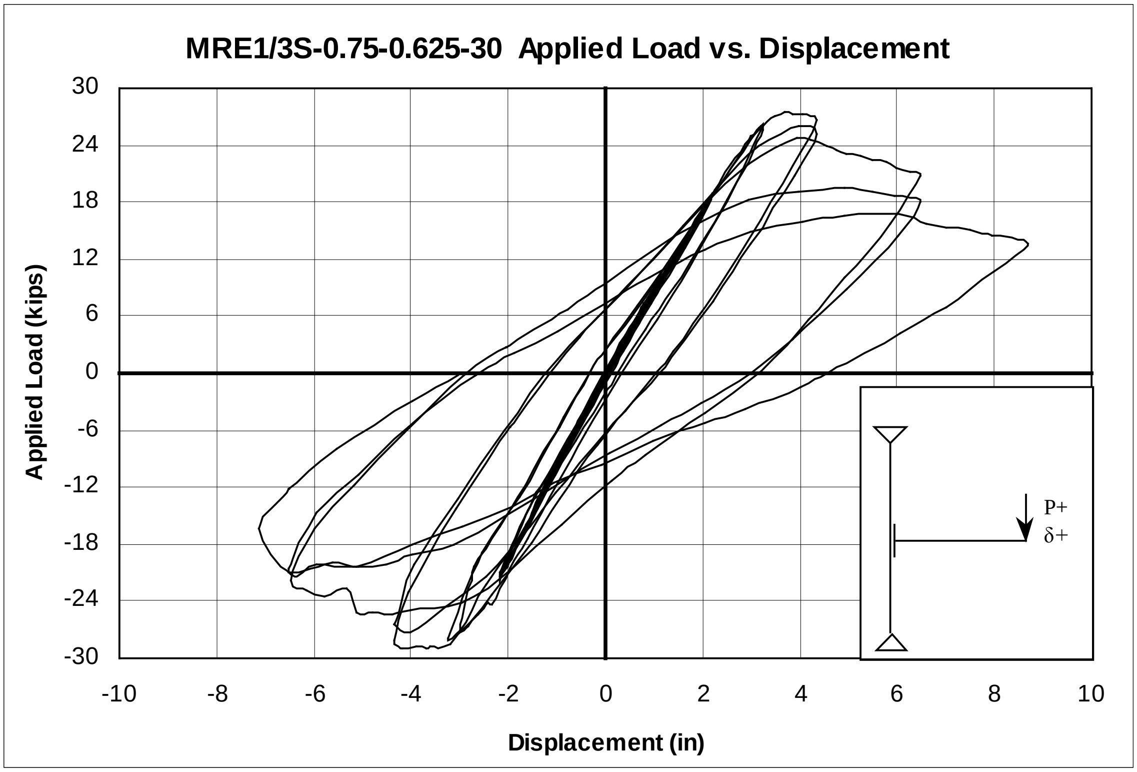
$$Y := \frac{b_p}{2} \cdot \left[\frac{h_1}{p_{fi}} + \frac{h_3}{s} + h_0 \cdot \left(\frac{1}{p_{fo}} + \frac{1}{2 \cdot s} \right) \right] + \frac{2}{g} \cdot \left[h_1 \cdot (p_{fi} + 1.5 \cdot p_b) + h_3 \cdot (s + 0.5 \cdot p_b) + h_0 \cdot (d_e + p_{fo}) \right] + \frac{g}{2}$$

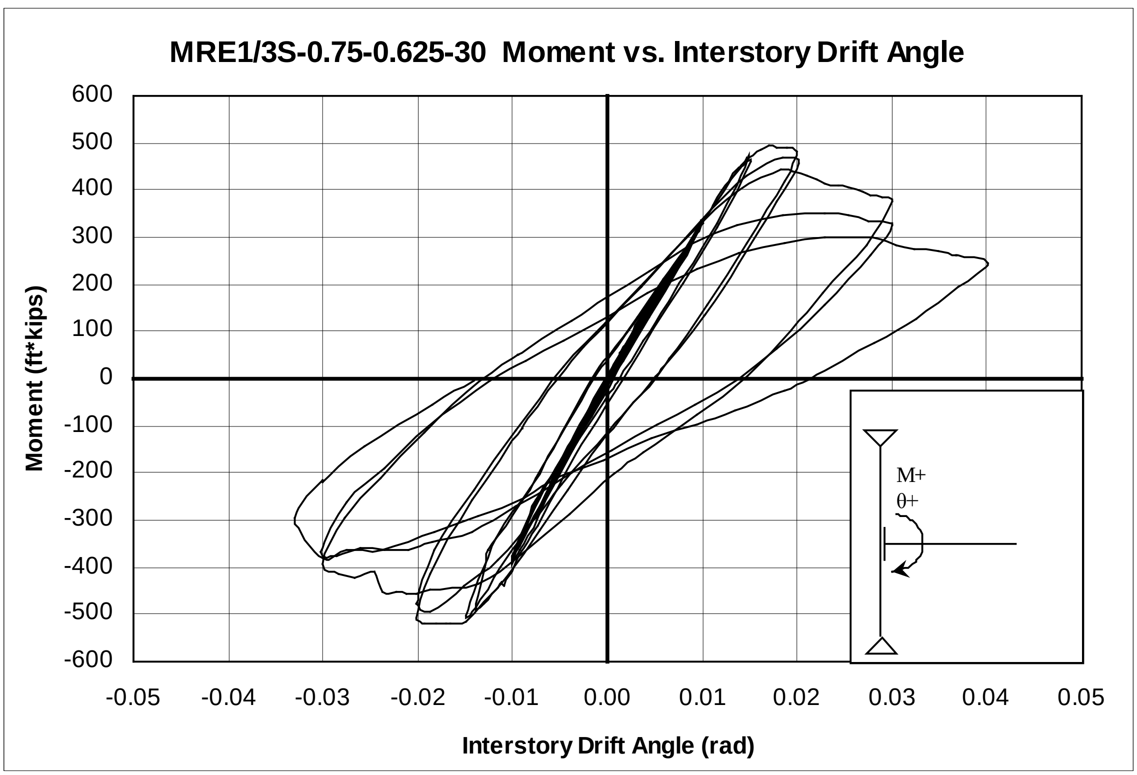
$Y = 400.7in$

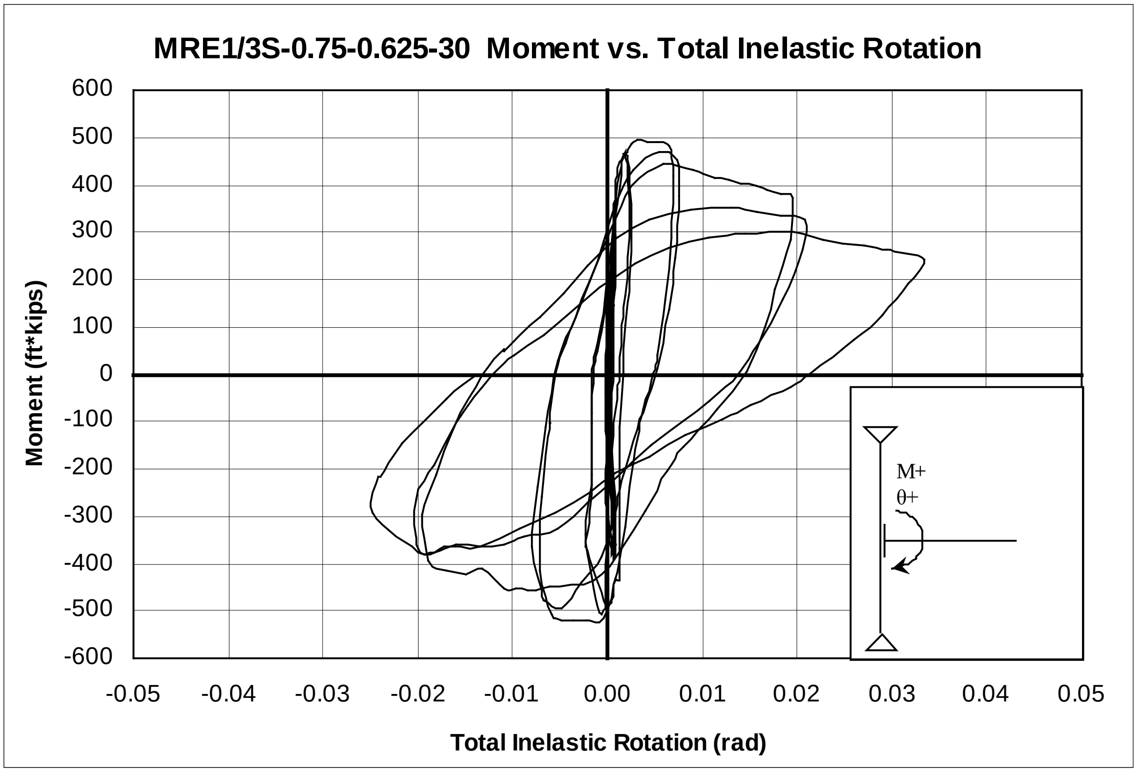
$M_{PL} := t_p^{\,2} \cdot F_{yp} \cdot Y$ $\qquad M_{PL} = 919.2 k \cdot ft$

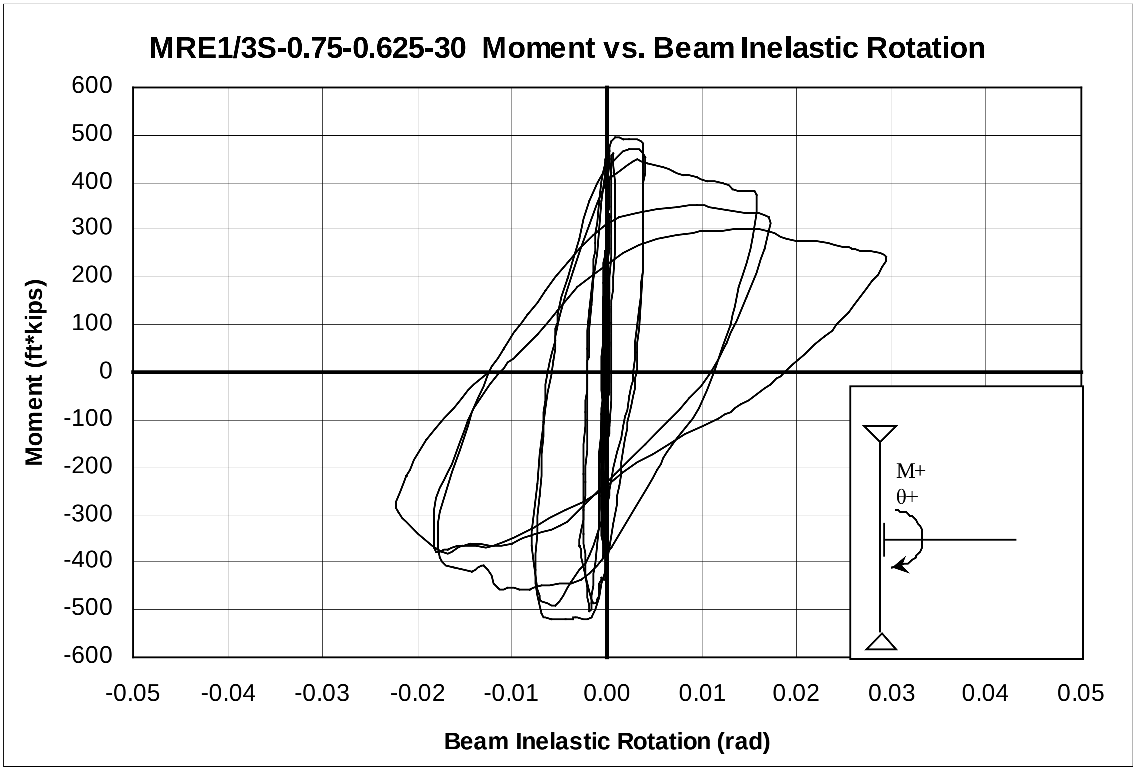
BOLT STRENGTH WITHOUT PRYING, Mnp

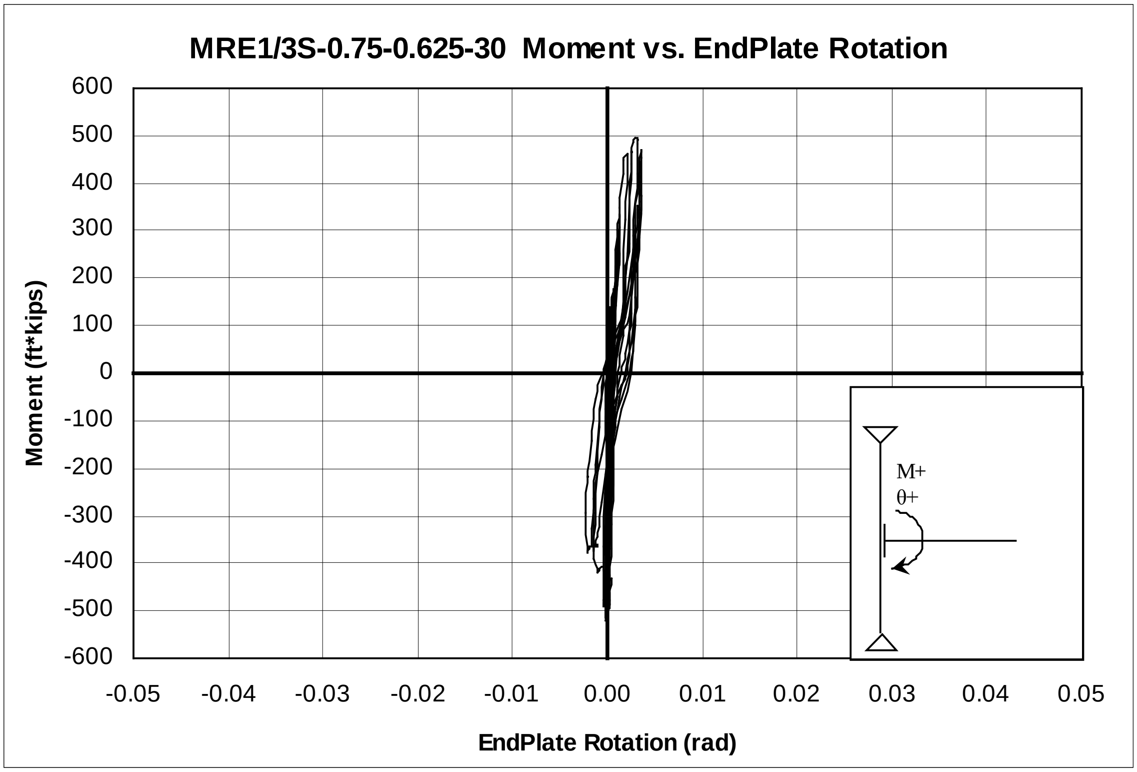
$M_{NP} := 2 \cdot P_t \cdot (d_0 + d_1 + d_2 + d_3)$ $\qquad M_{NP} = 696.6 k \cdot ft$

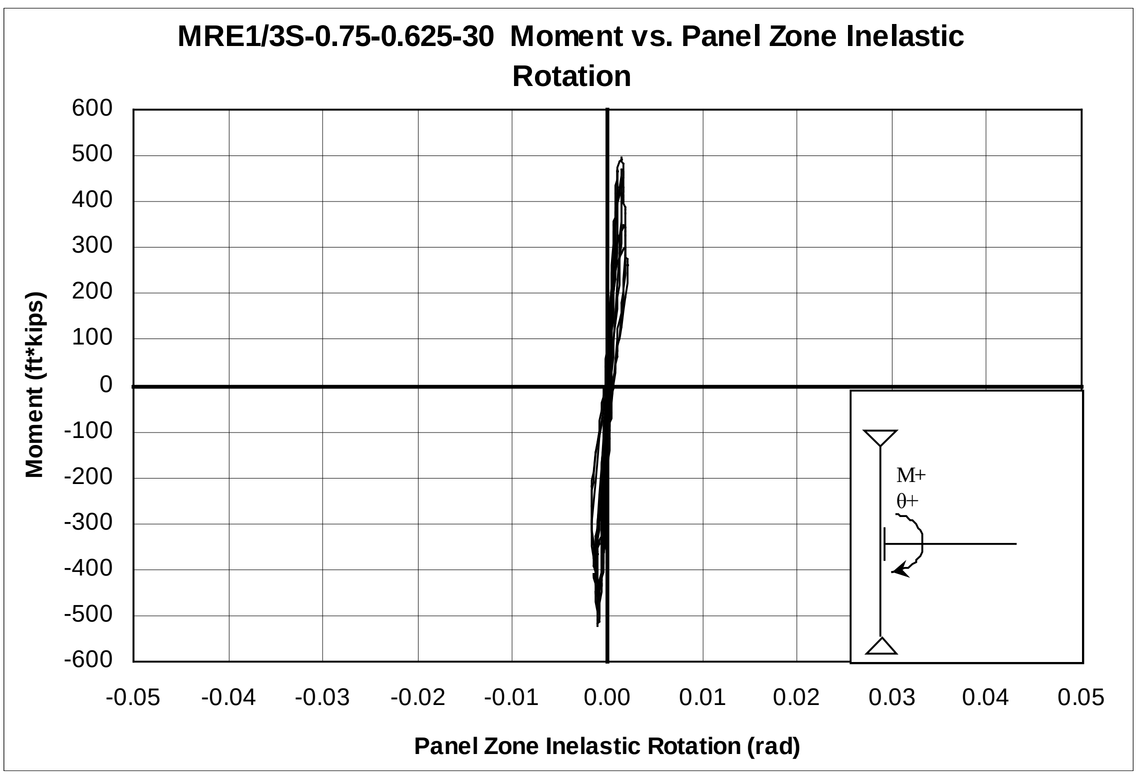

MRE1/3S-0.75-0.625-30 Applied Load vs. Displacement
Applied Load (kips)
30
24
18
12
6
0
-6
-12
-18
-24
-30
-10
-8
-6
-4
-2
0
2
4
6
8
10
Displacement (in)
P+
δ+

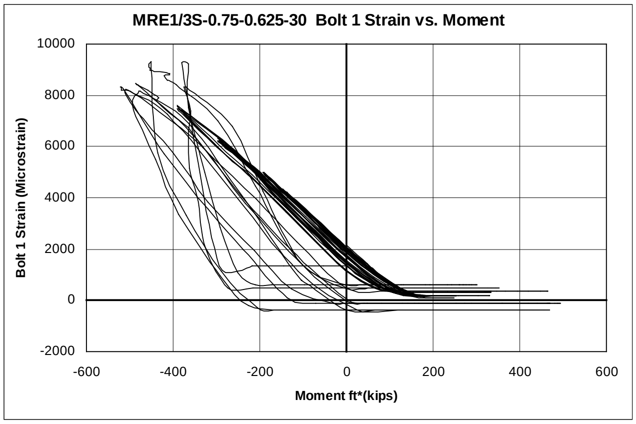

MRE1/3S-0.75-0.625-30 Moment vs. Interstory Drift Angle
Moment (ft*kips)
600
500
400
300
200
100
0
-100
-200
-300
-400
-500
-600
-0.05
-0.04
-0.03
-0.02
-0.01
0.00
0.01
0.02
0.03
0.04
0.05
Interstory Drift Angle (rad)
M+
θ+

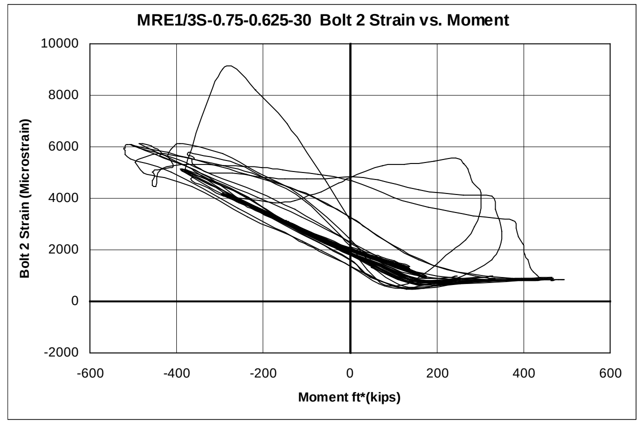

MRE1/3S-0.75-0.625-30 Moment vs. Total Inelastic Rotation
Moment (ft*kips)
600
500
400
300
200
100
0
-100
-200
-300
-400
-500
-600
-0.05
-0.04
-0.03
-0.02
-0.01
0.00
0.01
0.02
0.03
0.04
0.05
Total Inelastic Rotation (rad)
M+
θ+

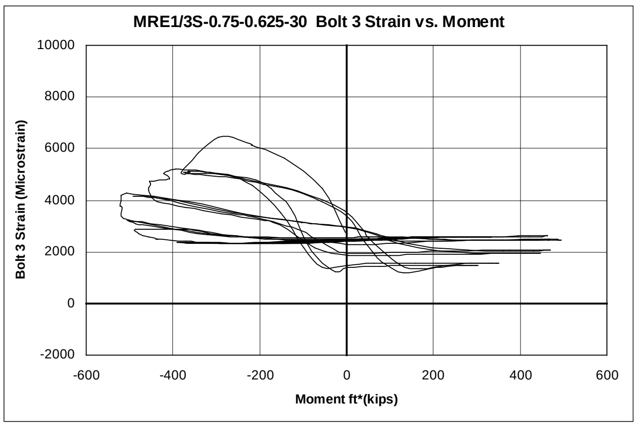

MRE1/3S-0.75-0.625-30 Moment vs. Beam Inelastic Rotation
Moment (ft*kips)
600
500
400
300
200
100
0
-100
-200
-300
-400
-500
-600
-0.05
-0.04
-0.03
-0.02
-0.01
0.00
0.01
0.02
0.03
0.04
0.05
Beam Inelastic Rotation (rad)
M+
θ+

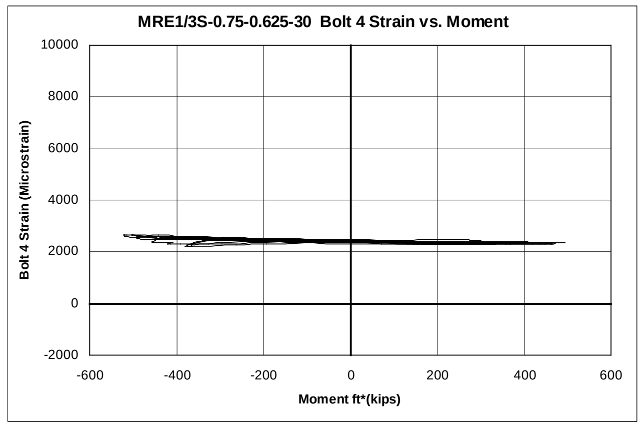

MRE1/3S-0.75-0.625-30 Moment vs. EndPlate Rotation
600
500
400
300
200
100
0
-100
-200
-300
-400
-500
-600
Moment (ft*kips)
-0.05
-0.04
-0.03
-0.02
-0.01
0.00
0.01
0.02
0.03
0.04
0.05
EndPlate Rotation (rad)
M+
θ+

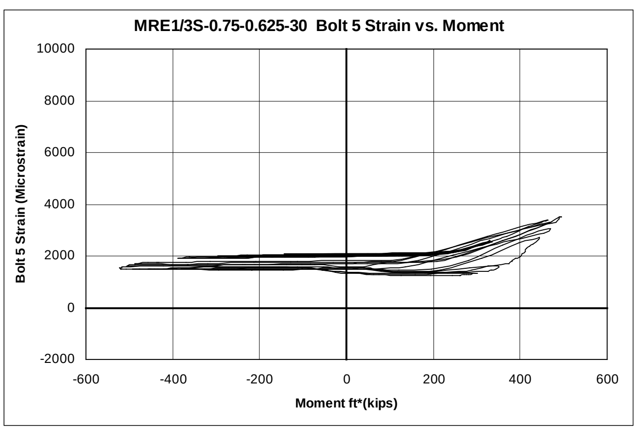

MRE1/3S-0.75-0.625-30 Moment vs. Panel Zone Inelastic Rotation
600
500
400
300
200
100
0
-100
-200
-300
-400
-500
-600
Moment (ft*kips)
-0.05
-0.04
-0.03
-0.02
-0.01
0.00
0.01
0.02
0.03
0.04
0.05
Panel Zone Inelastic Rotation (rad)
M+
θ+

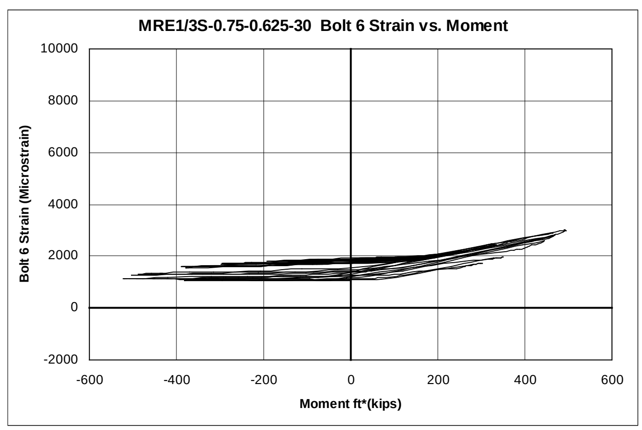

MRE1/3S-0.75-0.625-30 Bolt 1 Strain vs. Moment
Bolt 1 Strain (Microstrain)
Moment ft*(kips)
10000
8000
6000
4000
2000
0
-2000
-600
-400
-200
0
200
400
600

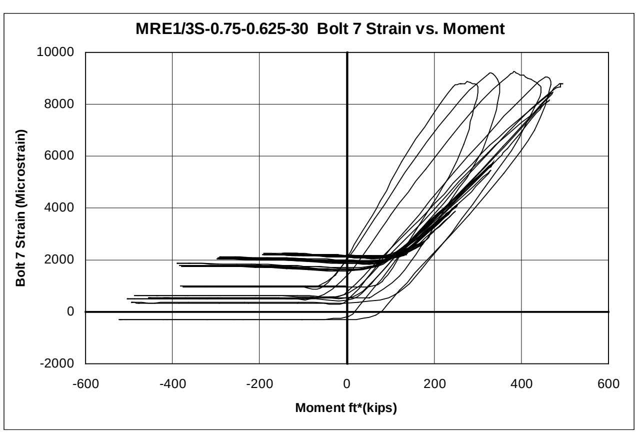

MRE1/3S-0.75-0.625-30 Bolt 2 Strain vs. Moment
Bolt 2 Strain (Microstrain)
Moment ft*(kips)
10000
8000
6000
4000
2000
0
-2000
-600
-400
-200
0
200
400
600

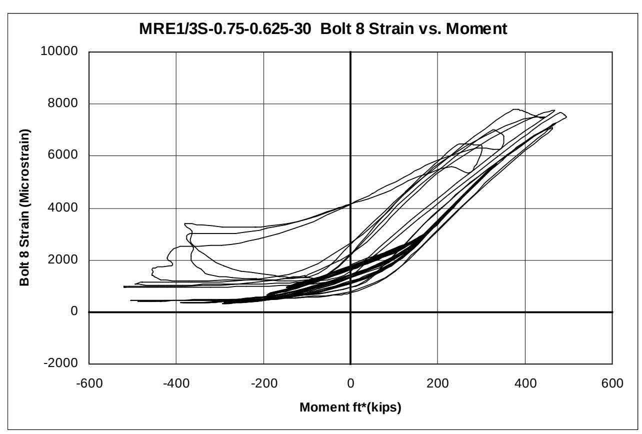

MRE1/3S-0.75-0.625-30 Bolt 3 Strain vs. Moment
Bolt 3 Strain (Microstrain)
Moment ft*(kips)
10000
8000
6000
4000
2000
0
-2000
-600
-400
-200
0
200
400
600

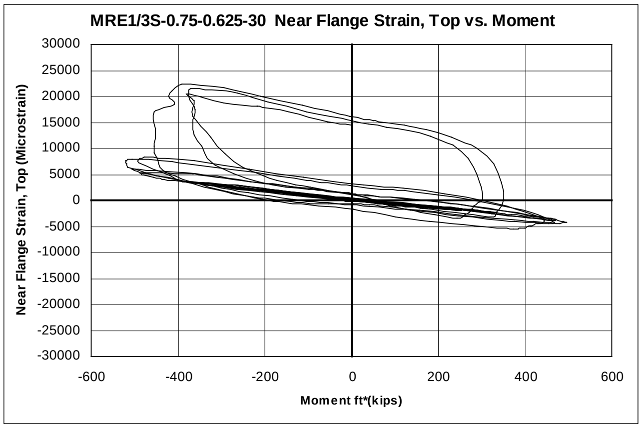

MRE1/3S-0.75-0.625-30 Bolt 4 Strain vs. Moment
Bolt 4 Strain (Microstrain)
Moment ft*(kips)
10000
8000
6000
4000
2000
0
-2000
-600
-400
-200
0
200
400
600

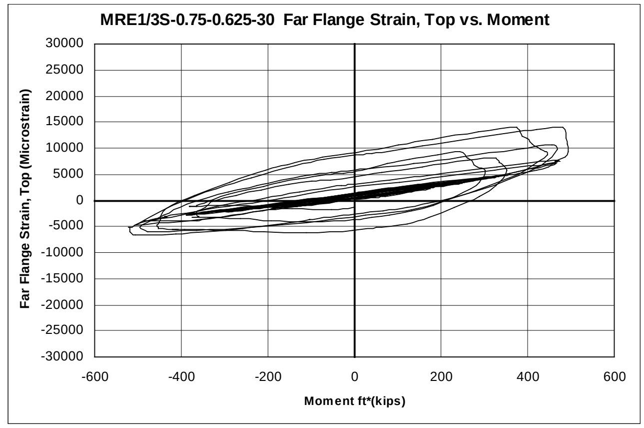

MRE1/3S-0.75-0.625-30 Bolt 5 Strain vs. Moment
Bolt 5 Strain (Microstrain)
Moment ft*(kips)
10000
8000
6000
4000
2000
0
-2000
-600
-400
-200
0
200
400
600

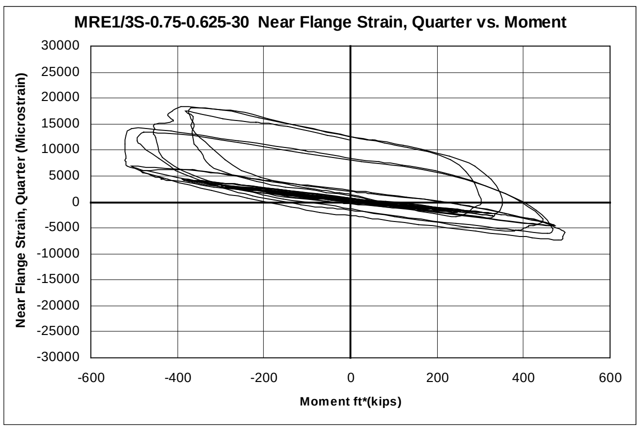

MRE1/3S-0.75-0.625-30 Bolt 6 Strain vs. Moment
Bolt 6 Strain (Microstrain)
Moment ft*(kips)
10000
8000
6000
4000
2000
0
-2000
-600
-400
-200
0
200
400
600

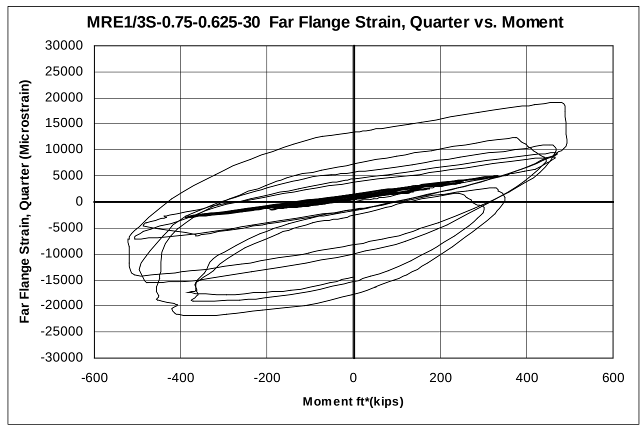

MRE1/3S-0.75-0.625-30 Bolt 7 Strain vs. Moment
Bolt 7 Strain (Microstrain)
Moment ft*(kips)
10000
8000
6000
4000
2000
0
-2000
-600
-400
-200
0
200
400
600

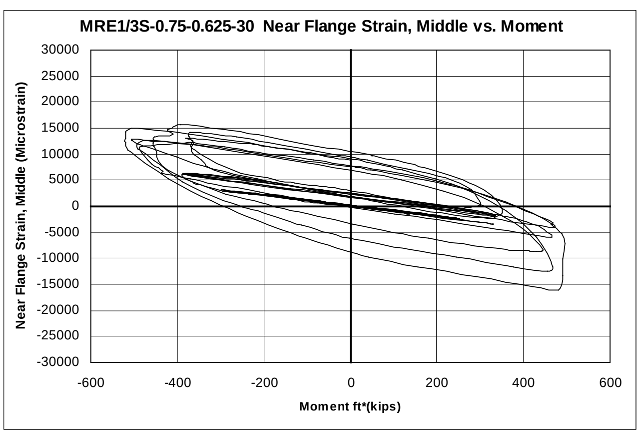

MRE1/3S-0.75-0.625-30 Bolt 8 Strain vs. Moment
Bolt 8 Strain (Microstrain)
Moment ft*(kips)
10000
8000
6000
4000
2000
0
-2000
-600
-400
-200
0
200
400
600

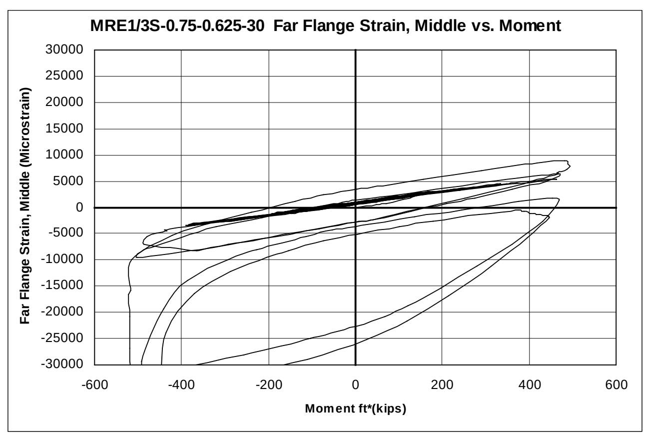

MRE1/3S-0.75-0.625-30 Near Flange Strain, Top vs. Moment
Near Flange Strain, Top (Microstrain)
Moment ft*(kips)
30000
25000
20000
15000
10000
5000
0
-5000
-10000
-15000
-20000
-25000
-30000
-600
-400
-200
0
200
400
600

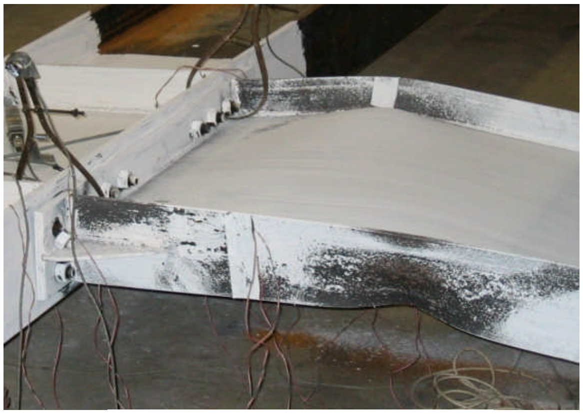

MRE1/3S-0.75-0.625-30 Far Flange Strain, Top vs. Moment
Far Flange Strain, Top (Microstrain)
Moment ft*(kips)
30000
25000
20000
15000
10000
5000
0
-5000
-10000
-15000
-20000
-25000
-30000
-600
-400
-200
0
200
400
600


MRE1/3S-0.75-0.625-30 Near Flange Strain, Quarter vs. Moment
Near Flange Strain, Quarter (Microstrain)
Moment ft*(kips)
30000
25000
20000
15000
10000
5000
0
-5000
-10000
-15000
-20000
-25000
-30000
-600
-400
-200
0
200
400
600


MRE1/3S-0.75-0.625-30 Far Flange Strain, Quarter vs. Moment
Far Flange Strain, Quarter (Microstrain)
Moment ft*(kips)
30000
25000
20000
15000
10000
5000
0
-5000
-10000
-15000
-20000
-25000
-30000
-600
-400
-200
0
200
400
600


MRE1/3S-0.75-0.625-30 Near Flange Strain, Middle vs. Moment
Near Flange Strain, Middle (Microstrain)
Moment ft*(kips)
30000
25000
20000
15000
10000
5000
0
-5000
-10000
-15000
-20000
-25000
-30000
-600
-400
-200
0
200
400
600


MRE1/3S-0.75-0.625-30 Far Flange Strain, Middle vs. Moment
Far Flange Strain, Middle (Microstrain)
Moment ft*(kips)
30000
25000
20000
15000
10000
5000
0
-5000
-10000
-15000
-20000
-25000
-30000
-600
-400
-200
0
200
400
600



Near Flange

Far Flange

Near Flange At Ruptured Bolt

Top View

MRE1/3S-0.75-0.625-30 AFTER TESTING

APPENDIX F

4E – 1.25 – 1.25 – 60 A

RESULTS AND TEST DATA

SPECIMEN PROPERTIES & TEST SUMMARY

TEST NAME: 4E-1.25-1.25-60 A
TEST DATE: August 20, 2003

BEAM DATA

DEPTH, d:	60.00	in
FLANGE WIDTH, b_f:	8.00	in
FLANGE THICKNESS, t_f:	0.63	in
WEB THICKNESS, t_w:	0.38	in
FLANGE YIELD STRESS, F_{yf}:	55.4	ksi
FLANGE ULTIMATE STRESS, F_{uf}:	84.8	ksi
WEB YIELD STRESS, F_{yw}:	60.0	ksi
WEB ULTIMATE STRESS, F_{uw}:	83.7	ksi

END-PLATE DATA

END-PLATE THICKNESS, t_p:	1.25	in
END-PLATE WIDTH, b_p:	8.02	in
END-PLATE LENGTH, L_p:	68.00	in
END-PLATE EXTENSION OUTSIDE FLANGE, p_{ext}:	4.00	in
OUTER PITCH, BOLT TO FLANGE, p_{fo}:	2.06	in
INNER PITCH, BOLT TO FLANGE, p_{fi}:	2.02	in
PITCH, BOLT TO BOLT, p_b:	na	
GAGE, g:	4.03	in
END-PLATE YIELD STRESS, F_{yp}:	62.4	ksi
END-PLATE ULTIMATE STRESS, F_{up}:	82.3	ksi

BOLT DATA

BOLT DIAMETER, d_b:	1.25	in
BOLT LENGTH, L_b:	4.25	in
BOLT TYPE:	A325	
NOMINAL BOLT TENSILE STRESS (AISC J3.6), F_t:	90	ksi
NOMINAL BOLT TENSILE STRENGTH, P_t:	110	kips
BOLT PRETENSION, T_b:	71	kips

EXPERIMENTAL RESULTS

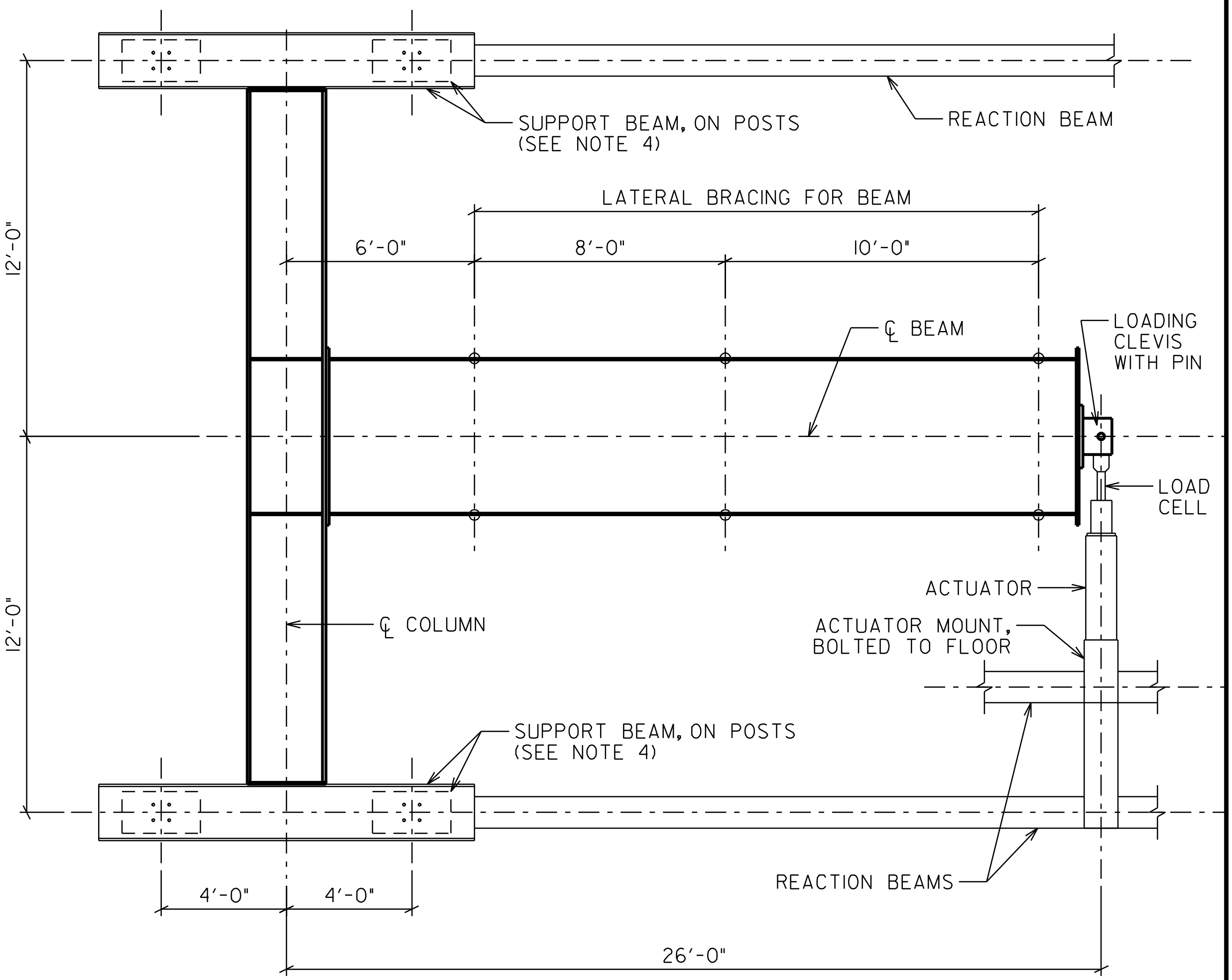
FAILURE MODE:	Bolt Rupture & Flange Local Buckling	
MAXIMUM MOMENT AT COLUMN CENTERLINE, $M_{max,cc}$:	2291.0	ft-kips
MAXIMUM MOMENT AT FAYING SURFACE, $M_{max,fs}$:	2180.9	ft-kips
MAXIMUM INTERSTORY DRIFT ANGLE, θ_{max}:	0.040	rad *
MAXIMUM INTERSTORY DRIFT ANGLE AT 80% 0F $M_{max,cc}$, θ_{preq}:	0.020	rad

* After bolt rupture on positive moment (far) side at 0.02 rad, half cycles conducted on near side to 0.04 rad

CALCULATED STRENGTHS

BEAM MOMENT STRENGTH [1], M_b:	2254.9	ft-kips
BEAM PLASTIC STRENGTH [1], M_p:	3020.4	ft-kips
BEAM EXPECTED PLASTIC STRENGTH, M_{pe}:	3161.2	ft-kips
END-PLATE STRENGTH [1], M_{PL}:	3672.1	ft-kips
BOLT TENSION RUPTURE (w/o Prying, using F_t), M_{NP}:	2177.6	ft-kips

1. Measured material properties used.



4E-1.25-1.25-60 A TEST SUBASSEMBLAGE PLAN

BEAM

WEB = $\frac{3}{8}\times 58\frac{3}{4}$
FLANGE = $\frac{5}{8}\times 8$
LENGTH = $288\frac{1}{2}$ (INCLUDES ENDPLATES)

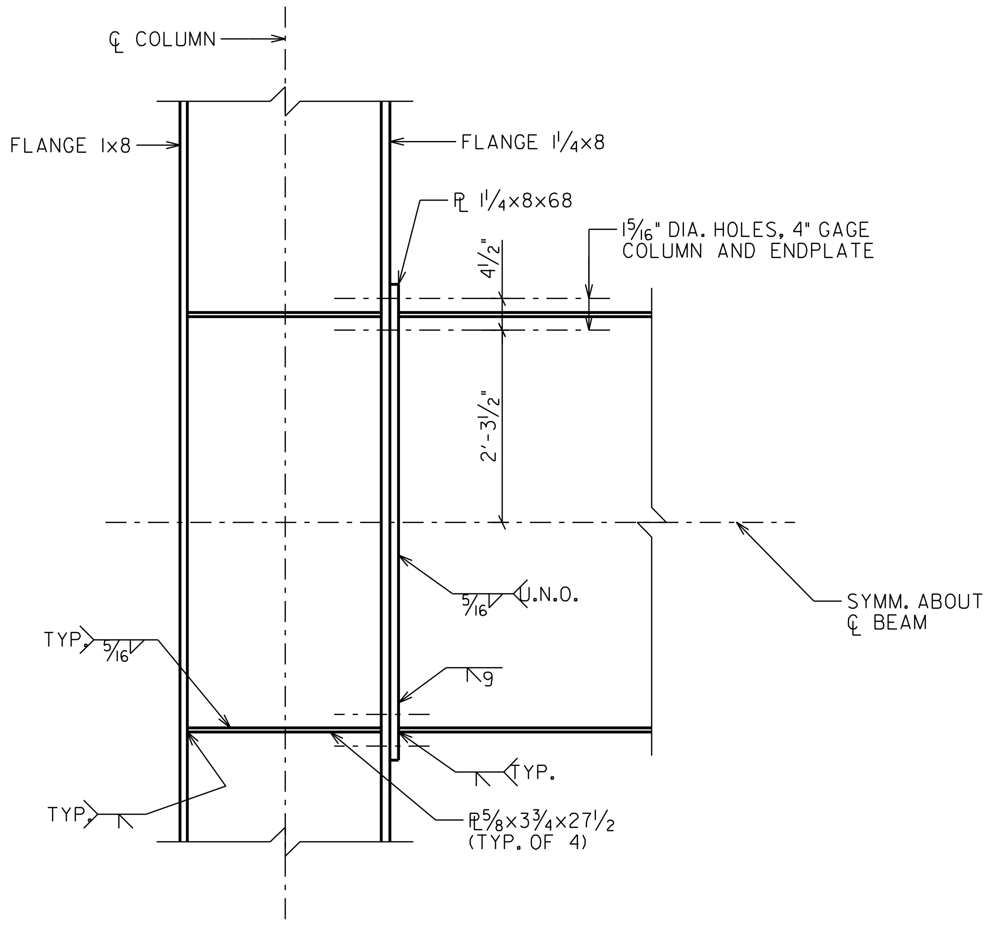
COLUMN

WEB = $\frac{1}{2}\times 27\frac{3}{4}$
FLANGES = $1\frac{1}{4}\times 8$ & 1×8
LENGTH = $266\frac{1}{2}$ (INCLUDES ENDPLATES)

NOTES

1. COLUMN, BEAM, AND CONNECTION STEEL, A572 Gr50.
2. ALL BOLTS, A325.
3. ALL WELDING, AWS D1.1, E70XX ELECTRODES.
4. COLUMN BOLTED TO SUPPORT BEAM. SUPPORT BEAM BOLTED POSTS. POSTS BOLTED TO REACTION BEAM.

SPECIMEN: 4E-1.25-1.25-60 A	DRAWN BY: SEB



4E-1.25-1.25-60 A
CONNECTION DETAIL

SPECIMEN: 4E-1.25-1.25-60 A	DRAWN BY: SEB

SAC PROTOCOL LOADING HISTORY

Test Name: 4E-1.25-1.25-60 A   Test By: SEB   Date: 8/20/2003

Load Step	Cycle	Max. Pos. Load (kips)	Max. Disp. (in)	Max. Neg. Load (kips)	Max. Disp. (in)	Comments
I (0.00375 rad)	1	26.8	1.225	-26.7	-1.202	
δ=1.163"	2	26.9	1.228	-26.3	-1.169	
	3	26.8	1.218	-27.5	-1.224	
	4	26.1	1.197	-27.2	-1.206	
	5	26.7	1.210	-27.7	-1.223	
	6	27.0	1.224	-27.1	-1.190	
		Permanent Set =	0.039"			
II (0.005 rad)	1	35.0	1.607	-35.8	-1.564	
δ=1.55"	2	33.8	1.608	-37.4	-1.557	
	3	31.8	1.596	-39.0	-1.586	
	4	30.2	1.617	-39.4	-1.618	
	5	28.3	1.613	-39.2	-1.603	
	6	26.6	1.598	-39.8	-1.607	
		Permanent Set =	0.201"			
III (0.0075 rad)	1	41.5	2.334	-54.3	-2.348	
δ=2.325"	2	40.3	2.347	-53.7	-2.333	
	3	40.5	2.361	-54.0	-2.339	
	4	39.9	2.348	-53.9	-2.342	
	5	40.0	2.341	-53.9	-2.345	
	6	40.1	2.354	-53.6	-2.335	
		Permanent Set =	0.187"			
IV (0.01 rad)	1	55.3	3.101	-68.7	-3.112	
δ=3.1"	2	56.4	3.111	-68.4	-3.116	
	3	56.2	3.114	-68.4	-3.135	
	4	56.8	3.127	-68.2	-3.134	
		Permanent Set =	0.111"			
V (0.015 rad)	1	78.3	4.661	-83.0	-4.666	
δ=4.65"	2	80.5	4.674	-83.0	-4.672	
		Permanent Set =	-0.510"			
VI (0.02 rad)	1	84.0	6.376	-78.1	-6.250	Flange & web local buckling
δ=6.2"	2	83.6	6.236	-75.5	-6.239	
		Permanent Set =	-1.614"			
VII (0.03 rad)	1	-	-	-59.2	-9.346	Bolt rupture, then bolt replacement
δ=9.3"	2	-	-	-57.2	-9.334	Half cycles performed with new bolts
		Permanent Set =	-4.966"			
VIII (0.04 rad)	1	-	-	-48.0	-12.410	Severe flange & web local buckling
δ=12.4"	2	-	-	-45.4	-12.400	
		Permanent Set =	-7.621"			
IX (0.05 rad)	1					
	2					
		Permanent Set =				

CALCULATED STRENGTHS

TEST NAME: 4E-1.25-1.25-60 A
TEST DATE: August 20, 2003

Specified Grade for all Steel = ASTM A572, Grade 50

MEASURED PROPERTIES

$d := 60 \cdot in$ $\quad F_{yf} := 55.4 \cdot ksi$ $\quad t_p := 1.25 \cdot in$ $\quad p_{fo} := 2.06 \cdot in$

$b_f := 8.00 \cdot in$ $\quad F_{uf} := 84.8 \cdot ksi$ $\quad b_p := 8.02 \cdot in$ $\quad p_{fi} := 2.02 \cdot in$

$t_f := 0.63 \cdot in$ $\quad F_{yw} := 60.0 \cdot ksi$ $\quad L_p := 68 \cdot in$ $\quad g := 4.03 \cdot in$

$t_w := 0.38 \cdot in$ $\quad F_{uw} := 83.7 \cdot ksi$ $\quad p_{ext} := 4.00 \cdot in$ $\quad F_{yp} := 62.4 \cdot ksi$ $\quad h := d - 2 \cdot t_f$

SPECIFIED PROPERTIES

$R_y := 1.1$ $\quad P_t := 110 \cdot k$

BEAM PLASTIC STRENGTH, Mp

$$M_p := 2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right) \cdot F_{yf} + 2 \cdot t_w \cdot \frac{h}{2} \cdot \frac{h}{4} \cdot F_{yw} \qquad M_p = 3020.4 \, k \cdot ft$$

BEAM EXPECTED PLASTIC STRENGTH, Mpe

$$M_{pe} := 1.1 \cdot R_y \cdot \left[2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right) \cdot 50 \cdot ksi + 2 \cdot t_w \cdot \frac{h}{2} \cdot \frac{h}{4} \cdot 50 \cdot ksi\right] \qquad M_{pe} = 3161.2 \, k \cdot ft$$

BEAM NOMINAL STRENGTH, Mn

$\frac{b_f}{2 \cdot t_f} = 6.3$ $\quad \frac{65}{\sqrt{50}} = 9.2$ $\quad$ Flange is compact

$\frac{970}{\sqrt{50}} = 137.2$ $\quad \frac{h}{t_w} = 154.6$ $\quad$ Web is slender, refer to Appendix G

Therefore, Mb< Mp

$$I_x := 2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right)^2 + \frac{1}{12} \cdot t_w \cdot h^3 \qquad I_x = 15301 \, in^4 \qquad S_x := \frac{I_x}{0.5 \cdot d} \qquad S_x = 510 \, in^3$$

$$a_r := \frac{h \cdot t_w}{b_f \cdot t_f} \qquad m := \frac{55}{55} \qquad R_e := \frac{12 + a_r \cdot \left(3 \cdot m - m^3\right)}{12 + 2 \cdot a_r} \qquad R_e = 1$$

BEAM MOMENT STRENGTH, Mb (CONT'D)

$I_T := \frac{1}{12}\cdot\left(\frac{0.5\cdot d}{3}\right)\cdot t_w^{3} + \frac{1}{12}\cdot t_f\cdot b_f^{3}$ $\quad I_T = 26.9in^4$

$A_T := \left(\frac{0.5\cdot d}{3}\right)\cdot t_w + t_f\cdot b_f$ $\quad A_T = 8.8in^2$ $\quad r_T := \sqrt{\frac{I_T}{A_T}}$ $\quad r_T = 1.7in$

Required L_b: $\quad L_b := 40.4 r_T$ $\quad L_b = 5.9ft$ $\quad$ If the unbraced length is less than L_b: $\quad F_{cr} := F_{yf}$

$$R_{PG} := 1 - \left(\frac{a_r}{1200 + 300\cdot a_r}\right)\cdot\left(\frac{h}{t_w} - 5.7\cdot\sqrt{\frac{29000ksi}{F_{cr}}}\right)$$

$M_b := S_x\cdot R_{PG}\cdot R_e\cdot F_{cr}$ $\quad M_b = 2254.9k\cdot ft$

END-PLATE STRENGTH, Mpl

$d_0 := d - \frac{t_f}{2} + p_{fo}$ $\quad d_0 = 61.7in$ $\quad h_0 := d_0 + \frac{t_f}{2}$ $\quad h_0 = 62.1in$

$d_1 := d - \frac{t_f}{2} - t_f - p_{fi}$ $\quad d_1 = 57in$ $\quad h_1 := d_1 + \frac{t_f}{2}$ $\quad h_1 = 57.4in$

$s := \frac{1}{2}\cdot\sqrt{b_p\cdot g}$ $\quad s = 2.84in$

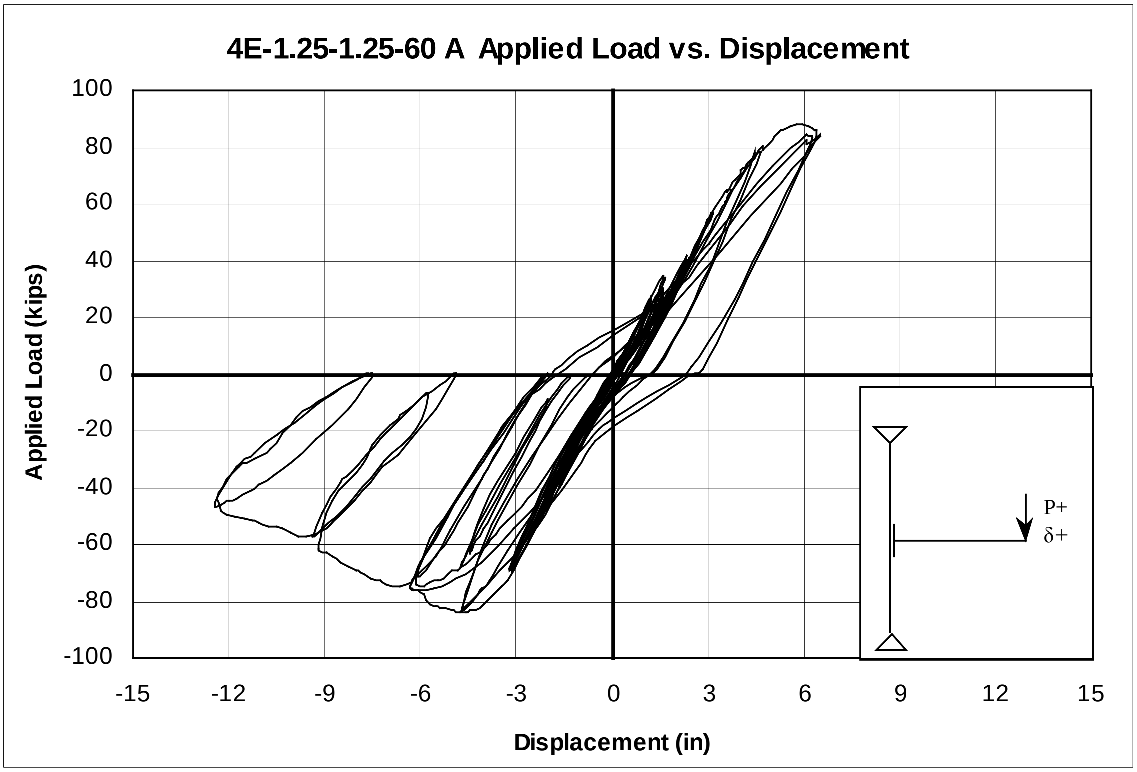
$$Y := \frac{b_p}{2}\cdot\left[h_1\cdot\left(\frac{1}{p_{fi}} + \frac{1}{s}\right) + h_0\cdot\left(\frac{1}{p_{fo}}\right) - \frac{1}{2}\right] + \frac{2}{g}\cdot\left[h_1\cdot\left(p_{fi} + s\right)\right]$$

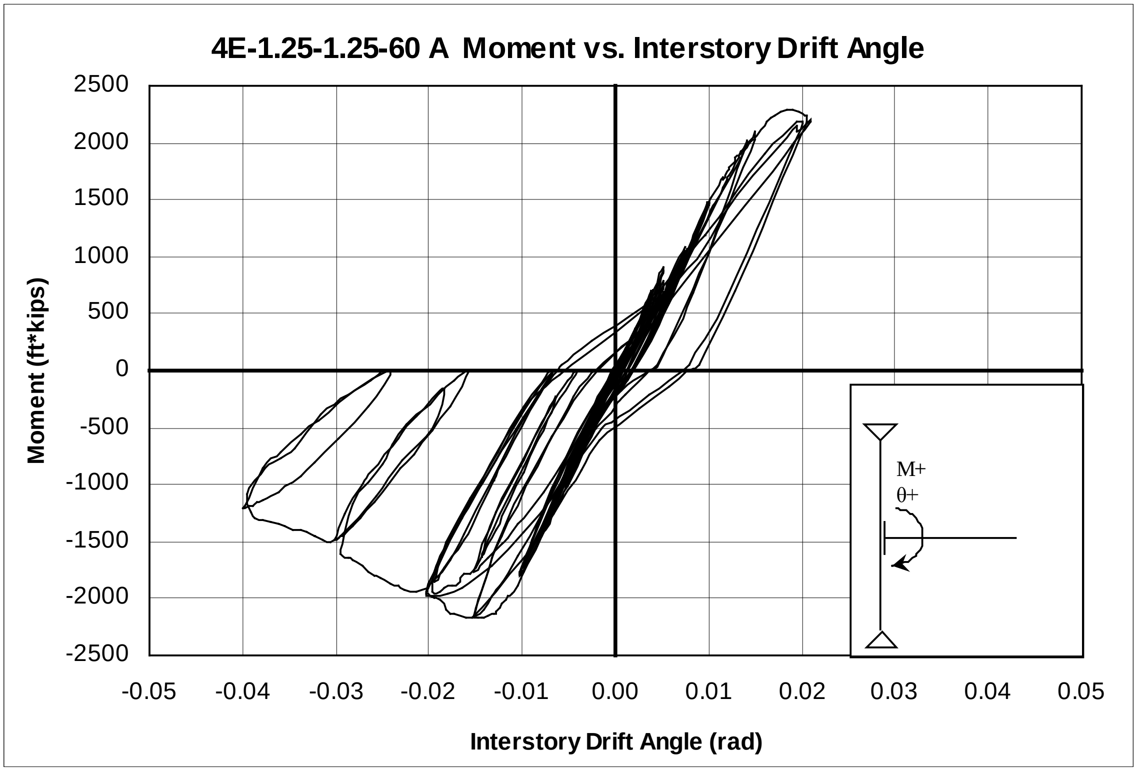
$Y = 451.9in$

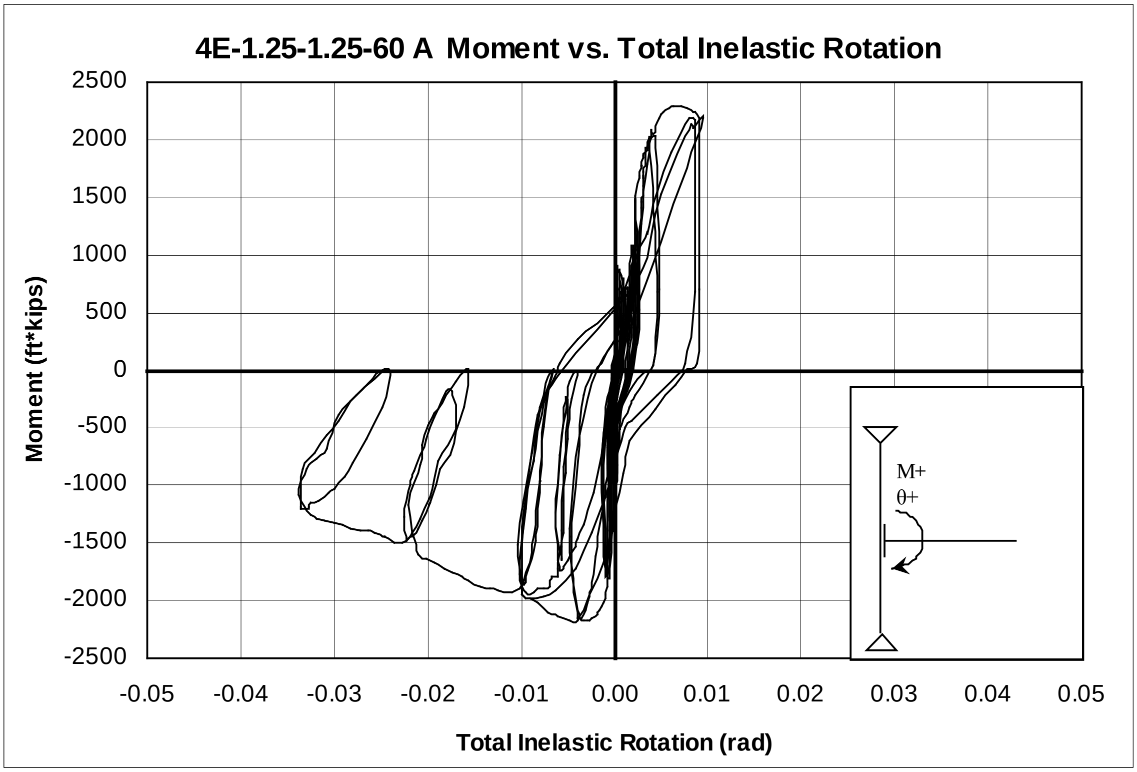
$M_{PL} := t_p^{2}\cdot F_{yp}\cdot Y$ $\quad M_{PL} = 3672.1k\cdot ft$

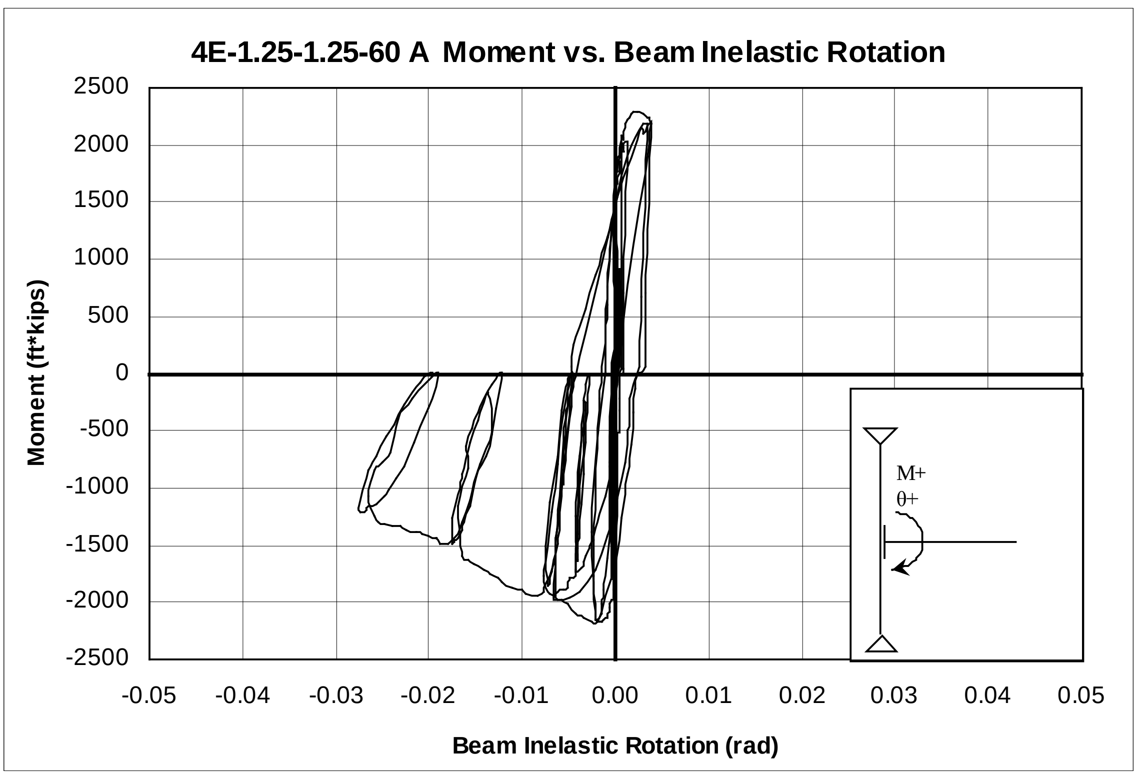
BOLT STRENGTH WITHOUT PRYING, Mnp

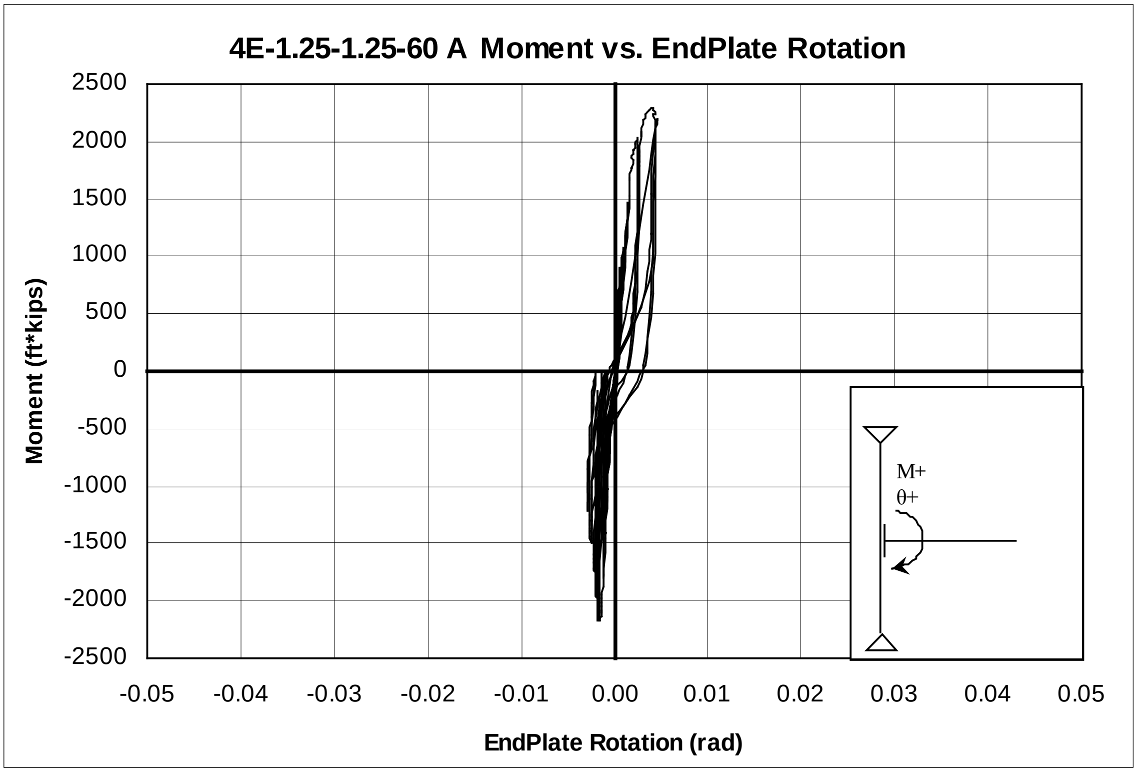
$M_{NP} := 2\cdot P_t\cdot\left(d_0 + d_1\right)$ $\quad M_{NP} = 2177.6k\cdot ft$

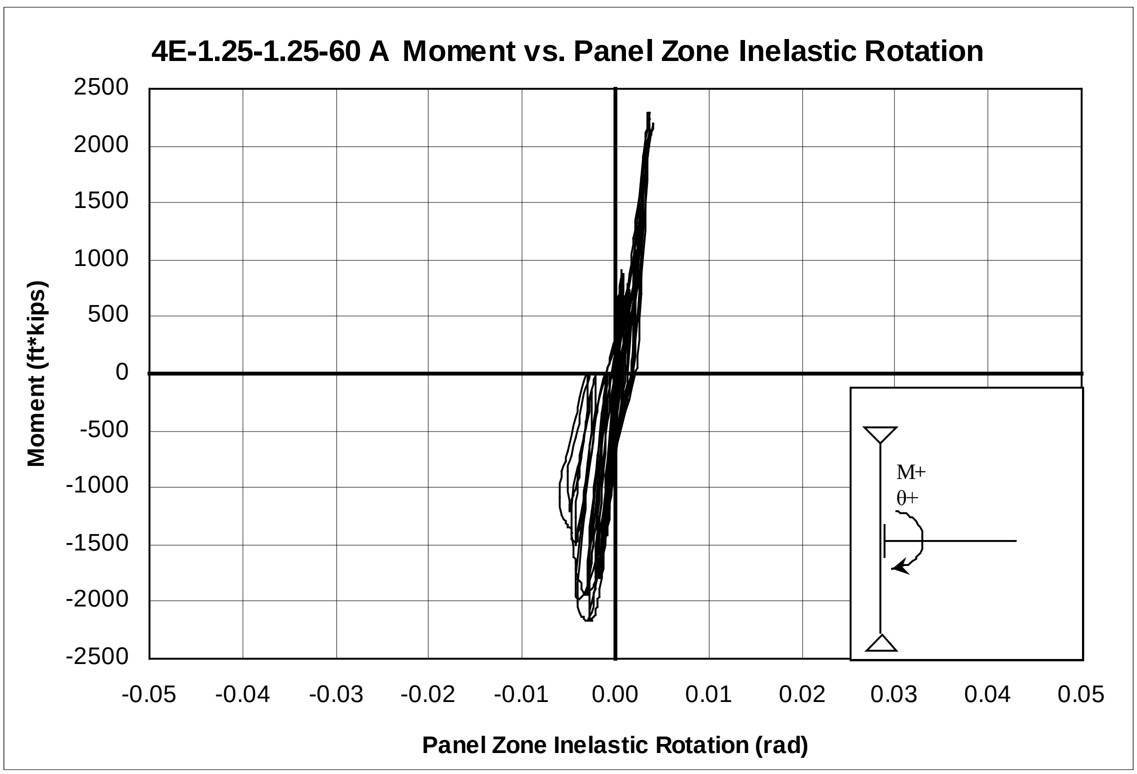

4E-1.25-1.25-60 A Applied Load vs. Displacement
Applied Load (kips)
100
80
60
40
20
0
-20
-40
-60
-80
-100
-15
-12
-9
-6
-3
0
3
6
9
12
15
Displacement (in)
P+
δ+

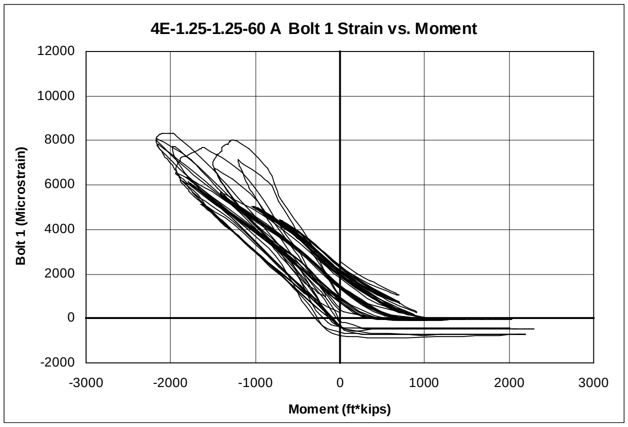

4E-1.25-1.25-60 A Moment vs. Interstory Drift Angle
Moment (ft*kips)
2500
2000
1500
1000
500
0
-500
-1000
-1500
-2000
-2500
-0.05
-0.04
-0.03
-0.02
-0.01
0.00
0.01
0.02
0.03
0.04
0.05
Interstory Drift Angle (rad)
M+
θ+

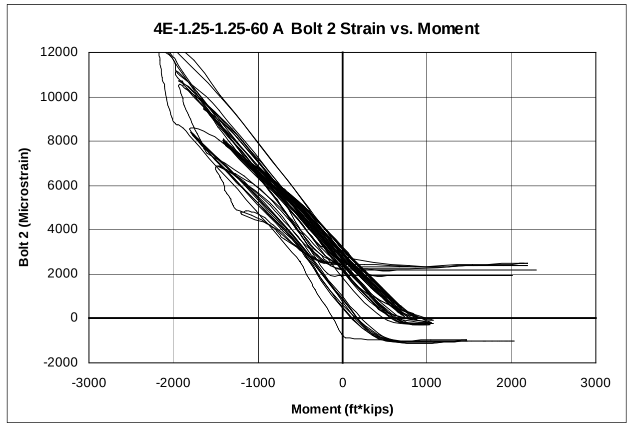

4E-1.25-1.25-60 A Moment vs. Total Inelastic Rotation
Moment (ft*kips)
2500
2000
1500
1000
500
0
-500
-1000
-1500
-2000
-2500
-0.05
-0.04
-0.03
-0.02
-0.01
0.00
0.01
0.02
0.03
0.04
0.05
Total Inelastic Rotation (rad)
M+
θ+

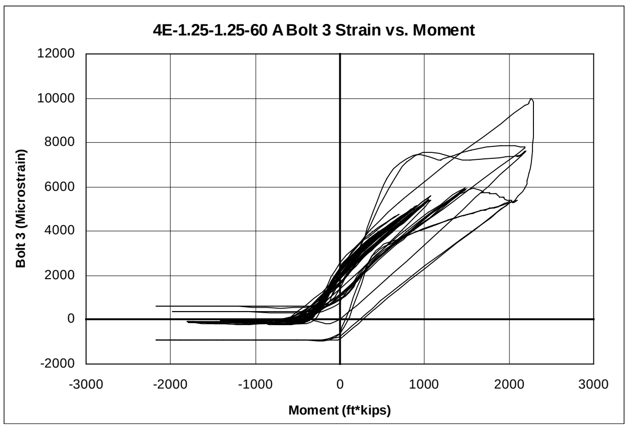

4E-1.25-1.25-60 A Moment vs. Beam Inelastic Rotation
Moment (ft*kips)
2500
2000
1500
1000
500
0
-500
-1000
-1500
-2000
-2500
-0.05
-0.04
-0.03
-0.02
-0.01
0.00
0.01
0.02
0.03
0.04
0.05
Beam Inelastic Rotation (rad)
M+
θ+

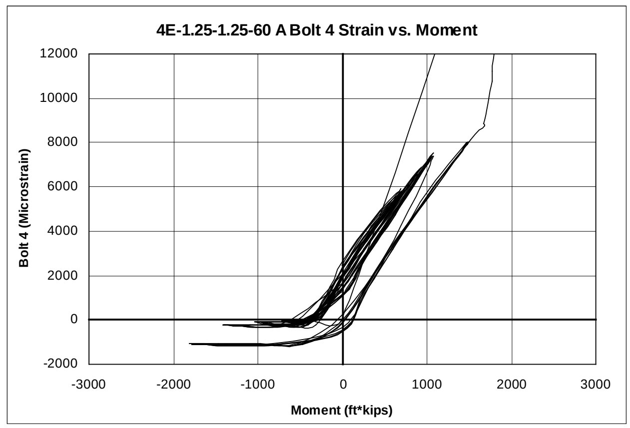

4E-1.25-1.25-60 A Moment vs. EndPlate Rotation
Moment (ft*kips)
2500
2000
1500
1000
500
0
-500
-1000
-1500
-2000
-2500
-0.05
-0.04
-0.03
-0.02
-0.01
0.00
0.01
0.02
0.03
0.04
0.05
EndPlate Rotation (rad)
M+
θ+

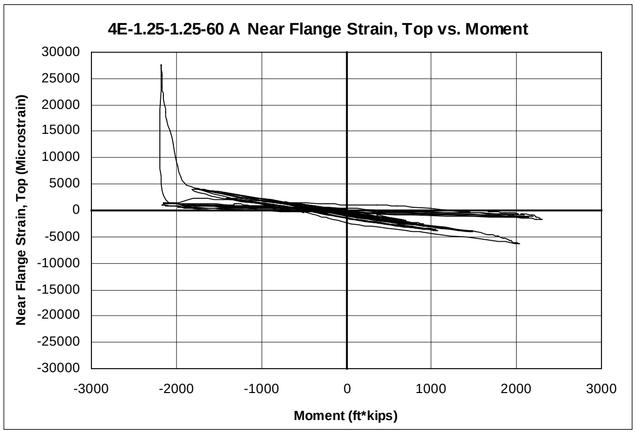

4E-1.25-1.25-60 A Moment vs. Panel Zone Inelastic Rotation
Moment (ft*kips)
2500
2000
1500
1000
500
0
-500
-1000
-1500
-2000
-2500
-0.05
-0.04
-0.03
-0.02
-0.01
0.00
0.01
0.02
0.03
0.04
0.05
Panel Zone Inelastic Rotation (rad)
M+
θ+

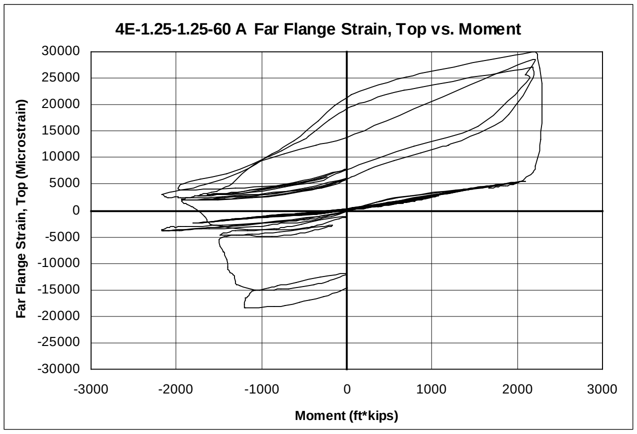

4E-1.25-1.25-60 A Bolt 1 Strain vs. Moment
Bolt 1 (Microstrain)
Moment (ft*kips)

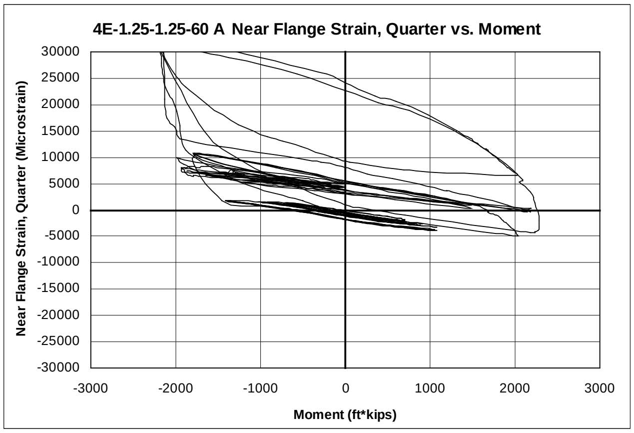

4E-1.25-1.25-60 A Bolt 2 Strain vs. Moment
Bolt 2 (Microstrain)
Moment (ft*kips)

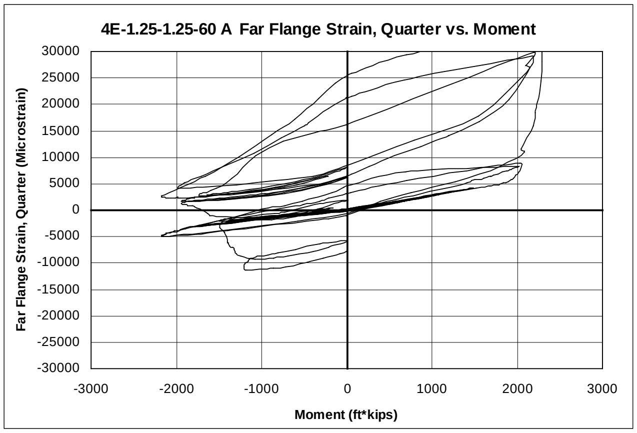

4E-1.25-1.25-60 A Bolt 3 Strain vs. Moment
Bolt 3 (Microstrain)
Moment (ft*kips)

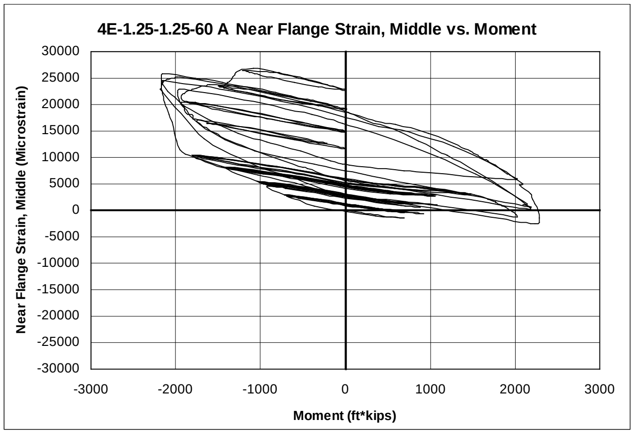

4E-1.25-1.25-60 A Bolt 4 Strain vs. Moment
Bolt 4 (Microstrain)
Moment (ft*kips)

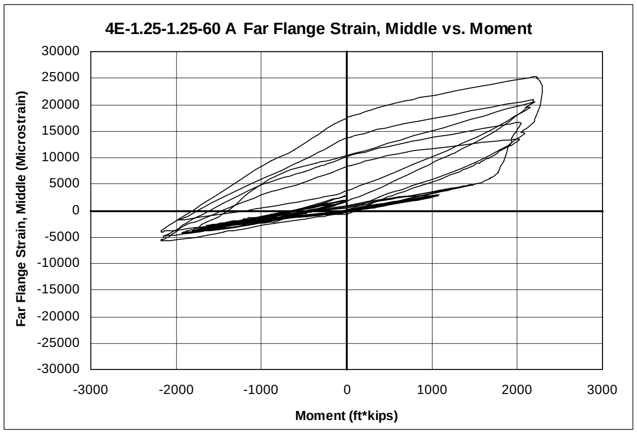

4E-1.25-1.25-60 A Near Flange Strain, Top vs. Moment
Near Flange Strain, Top (Microstrain)
30000
25000
20000
15000
10000
5000
0
-5000
-10000
-15000
-20000
-25000
-30000
-3000
-2000
-1000
0
1000
2000
3000
Moment (ft*kips)

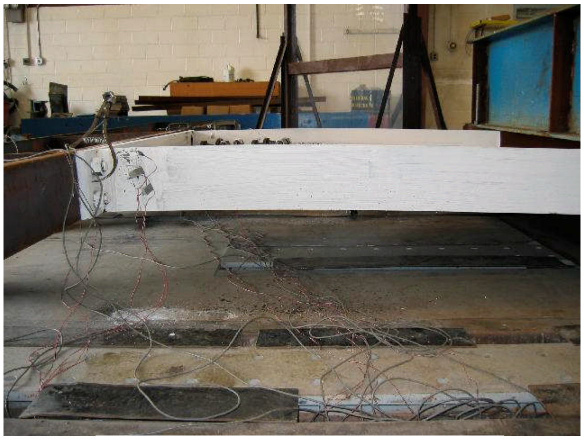

4E-1.25-1.25-60 A Far Flange Strain, Top vs. Moment
Far Flange Strain, Top (Microstrain)
30000
25000
20000
15000
10000
5000
0
-5000
-10000
-15000
-20000
-25000
-30000
-3000
-2000
-1000
0
1000
2000
3000
Moment (ft*kips)


4E-1.25-1.25-60 A Near Flange Strain, Quarter vs. Moment
Near Flange Strain, Quarter (Microstrain)
30000
25000
20000
15000
10000
5000
0
-5000
-10000
-15000
-20000
-25000
-30000
-3000
-2000
-1000
0
1000
2000
3000
Moment (ft*kips)


4E-1.25-1.25-60 A Far Flange Strain, Quarter vs. Moment
Far Flange Strain, Quarter (Microstrain)
30000
25000
20000
15000
10000
5000
0
-5000
-10000
-15000
-20000
-25000
-30000
-3000
-2000
-1000
0
1000
2000
3000
Moment (ft*kips)

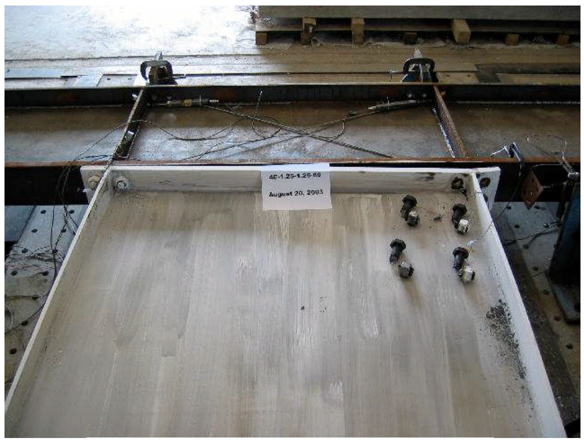

4E-1.25-1.25-60 A Near Flange Strain, Middle vs. Moment
Near Flange Strain, Middle (Microstrain)
30000
25000
20000
15000
10000
5000
0
-5000
-10000
-15000
-20000
-25000
-30000
-3000
-2000
-1000
0
1000
2000
3000
Moment (ft*kips)


4E-1.25-1.25-60 A Far Flange Strain, Middle vs. Moment
Far Flange Strain, Middle (Microstrain)
30000
25000
20000
15000
10000
5000
0
-5000
-10000
-15000
-20000
-25000
-30000
-3000
-2000
-1000
0
1000
2000
3000
Moment (ft*kips)



Near Flange

Far Flange



Top View

4E-1.25-1.25-60 A AFTER TESTING

APPENDIX G

4E – 1.25 – 1.25 – 60 B

RESULTS AND TEST DATA

SPECIMEN PROPERTIES & TEST SUMMARY

TEST NAME: 4E-1.25-1.25-60 B
TEST DATE: May 19, 2004

BEAM DATA

DEPTH, d:	60.00	in
FLANGE WIDTH, b_f:	8.00	in
FLANGE THICKNESS, t_f:	0.63	in
WEB THICKNESS, t_w:	0.38	in
FLANGE YIELD STRESS, F_{yf}:	66.8	ksi
FLANGE ULTIMATE STRESS, F_{uf}:	90.5	ksi
WEB YIELD STRESS, F_{yw}:	66.0	ksi
WEB ULTIMATE STRESS, F_{uw}:	77.1	ksi

END-PLATE DATA

END-PLATE THICKNESS, t_p:	1.26	in
END-PLATE WIDTH, b_p:	8.05	in
END-PLATE LENGTH, L_p:	68.00	in
END-PLATE EXTENSION OUTSIDE FLANGE, p_{ext}:	3.98	in
OUTER PITCH, BOLT TO FLANGE, p_{fo}:	2.01	in
INNER PITCH, BOLT TO FLANGE, p_{fi}:	2.06	in
PITCH, BOLT TO BOLT, p_b:	na	
GAGE, g:	4.05	in
END-PLATE YIELD STRESS, F_{yp}:	57.7	ksi
END-PLATE ULTIMATE STRESS, F_{up}:	80.9	ksi

BOLT DATA

BOLT DIAMETER, d_b:	1.25	in
BOLT LENGTH, L_b:	4.25	in
BOLT TYPE:	A490	
NOMINAL BOLT TENSILE STRESS (AISC J3.6), F_t:	113	ksi
NOMINAL BOLT TENSILE STRENGTH, P_t:	139	kips
BOLT PRETENSION, T_b:	102	kips

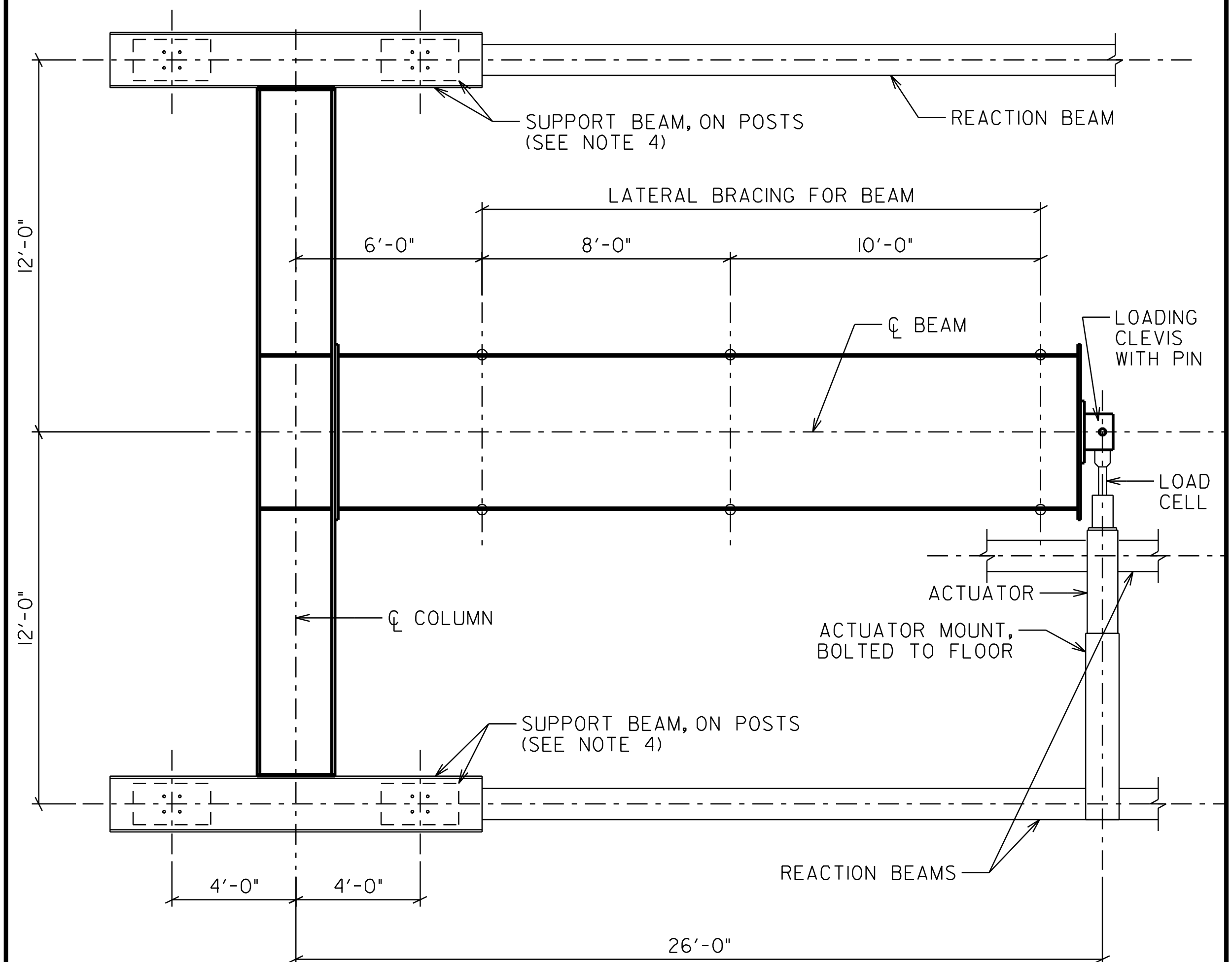
EXPERIMENTAL RESULTS

FAILURE MODE:	Flange Local Buckling	
MAXIMUM MOMENT AT COLUMN CENTERLINE, $M_{max,cc}$:	2801.0	ft-kips
MAXIMUM MOMENT AT FAYING SURFACE, $M_{max,fs}$:	2666.3	ft-kips
MAXIMUM INTERSTORY DRIFT ANGLE, θ_{max}:	0.030	rad
MAXIMUM INTERSTORY DRIFT ANGLE AT 80% 0F $M_{max,cc}$, θ_{preq}:	0.020	rad

CALCULATED STRENGTHS

BEAM MOMENT STRENGTH [1], M_b:	2661.0	ft-kips
BEAM PLASTIC STRENGTH [1], M_p:	3468.5	ft-kips
BEAM EXPECTED PLASTIC STRENGTH, M_{pe}:	3161.2	ft-kips
END-PLATE STRENGTH [1], M_{PL}:	3466.0	ft-kips
BOLT TENSION RUPTURE (w/o Prying, using F_t), M_{NP}:	2749.7	ft-kips

1. Measured material properties used.



4E-1.25-1.25-60 B TEST SUBASSEMBLAGE PLAN

BEAM

WEB = 3/8x58 3/4
FLANGE = 5/8x8
LENGTH = 288 1/2 (INCLUDES ENDPLATES)

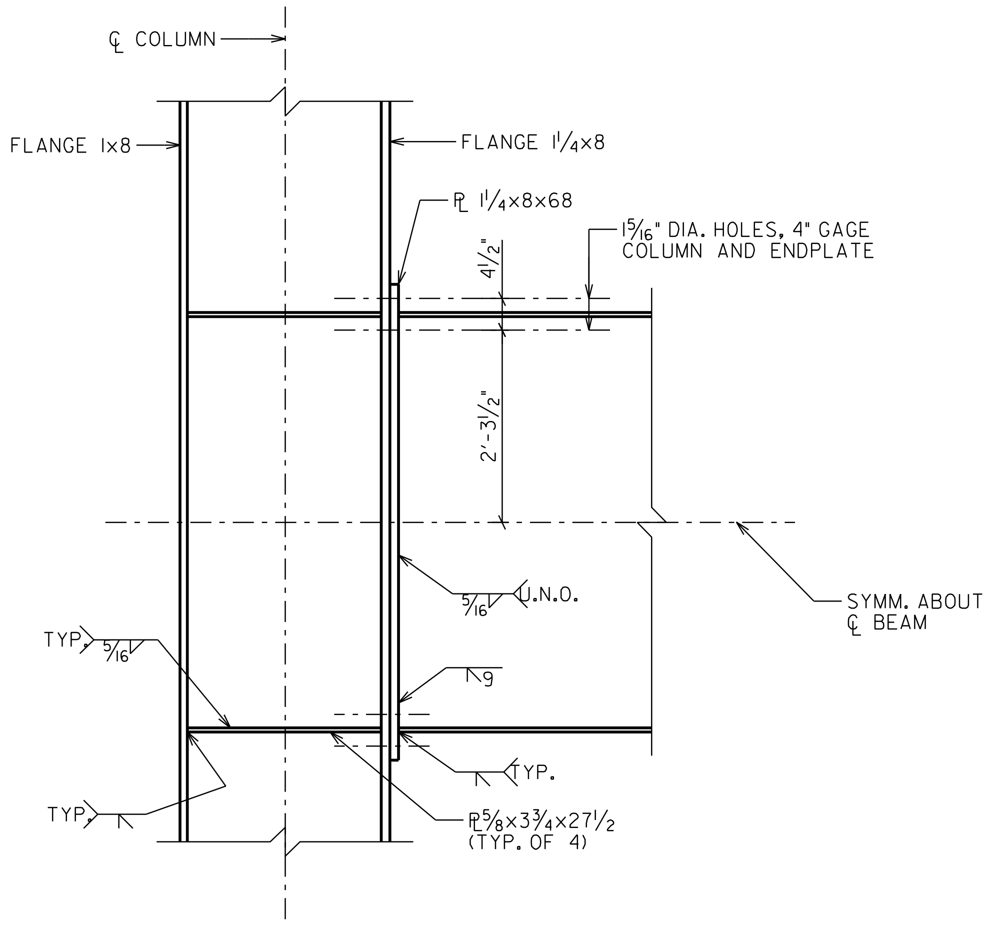
COLUMN

WEB = 1/2x27 3/4
FLANGES = 1 1/4x8 & 1x8
LENGTH = 266 1/2 (INCLUDES ENDPLATES)

NOTES

1. COLUMN, BEAM, AND CONNECTION STEEL, A572 Gr50.
2. ALL BOLTS, A490.
3. ALL WELDING, AWS D1.1, E70XX ELECTRODES.
4. COLUMN BOLTED TO SUPPORT BEAM. SUPPORT BEAM BOLTED POSTS. POSTS BOLTED TO REACTION BEAM.

SPECIMEN: 4E-1.25-1.25-60 B	DRAWN BY: SEB



4E-1.25-1.25-60 B
CONNECTION DETAIL

SPECIMEN: 4E-1.25-1.25-60 B	DRAWN BY: SEB

SAC PROTOCOL LOADING HISTORY

Test Name: 4E-1.25-1.25-60 B   Test By: SEB   Date: 5/19/2004

Load Step	Cycle	Max. Pos. Load (kips)	Max. Disp. (in)	Max. Neg. Load (kips)	Max. Disp. (in)	Comments
I (0.00375 rad)	1	30.8	1.164	-36.5	-1.171	
δ=1.163"	2	30.7	1.160	-37.3	-1.159	
	3	30.7	1.161	-37.5	-1.160	
	4	30.6	1.163	-37.7	-1.162	
	5	31.0	1.164	-37.9	-1.159	
	6	30.6	1.166	-37.9	-1.161	
		Permanent Set =	0.044"			
II (0.005 rad)	1	40.8	1.545	-48.4	-1.549	
δ=1.55"	2	41.2	1.554	-48.3	-1.551	
	3	41.3	1.553	-48.3	-1.541	
	4	41.5	1.558	-48.6	-1.545	
	5	41.1	1.556	-48.4	-1.540	
	6	41.2	1.555	-48.7	-1.550	
		Permanent Set =	0.059"			
III (0.0075 rad)	1	62.1	2.331	-69.1	-2.321	
δ=2.325"	2	62.9	2.328	-69.2	-2.330	
	3	63.2	2.328	-69.2	-2.327	
	4	63.4	2.345	-69.4	-2.329	
	5	63.4	2.336	-69.7	-2.337	
	6	63.5	2.335	-69.5	-2.320	
		Permanent Set =	0.015"			
IV (0.01 rad)	1	81.5	3.107	-88.7	-3.110	
δ=3.1"	2	81.3	3.110	-88.1	-3.101	
	3	80.7	3.102	-88.3	-3.106	
	4	81.2	3.106	-88.5	-3.113	Flange flaking near end-plate
		Permanent Set =	0.017"			
V (0.015 rad)	1	102.5	4.650	-105.5	-4.655	
δ=4.65"	2	102.5	4.646	-105.0	-4.656	Increased flange flaking
		Permanent Set =	-0.510"			
VI (0.02 rad)	1	97.2	6.202	-98.3	-6.205	Comp. flange local buckling
δ=6.2"	2	95.3	6.200	-94.9	-6.205	
		Permanent Set =	-1.848"			
VII (0.03 rad)	1	65.9	9.306	-63.9	-9.301	
δ=9.3"	2	52.9	9.310	-51.7	-9.302	Stop test, 50% strength reduction
		Permanent Set =	-4.466"			
VIII (0.04 rad)	1					
	2					
		Permanent Set =				
IX (0.05 rad)	1					
	2					
		Permanent Set =				

CALCULATED STRENGTHS

TEST NAME: 4E-1.25-1.25-60 B
TEST DATE: May 19, 2004

Specified Grade for all Steel = ASTM A572, Grade 50

MEASURED PROPERTIES

$d := 60 \cdot in$ $\quad F_{yf} := 66.8 \cdot ksi$ $\quad t_p := 1.26 \cdot in$ $\quad p_{fo} := 2.01 \cdot in$

$b_f := 8.00 \cdot in$ $\quad F_{uf} := 90.5 \cdot ksi$ $\quad b_p := 8.05 \cdot in$ $\quad p_{fi} := 2.06 \cdot in$

$t_f := 0.63 \cdot in$ $\quad F_{yw} := 66.0 \cdot ksi$ $\quad L_p := 68 \cdot in$ $\quad g := 4.05 \cdot in$

$t_w := 0.38 \cdot in$ $\quad F_{uw} := 77.1 \cdot ksi$ $\quad p_{ext} := 3.98 \cdot in$ $\quad F_{yp} := 57.7 \cdot ksi$ $\quad h := d - 2 \cdot t_f$

SPECIFIED PROPERTIES

$R_y := 1.1$ $\quad P_t := 139 \cdot k$

BEAM PLASTIC STRENGTH, Mp

$$M_p := 2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right) \cdot F_{yf} + 2 \cdot t_w \cdot \frac{h}{2} \cdot \frac{h}{4} \cdot F_{yw} \qquad M_p = 3468.5 k \cdot ft$$

BEAM EXPECTED PLASTIC STRENGTH, Mpe

$$M_{pe} := 1.1 \cdot R_y \cdot \left[2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right) \cdot 50 \cdot ksi + 2 \cdot t_w \cdot \frac{h}{2} \cdot \frac{h}{4} \cdot 50 \cdot ksi\right] \qquad M_{pe} = 3161.2 k \cdot ft$$

BEAM MOMENT STRENGTH, Mb

$\frac{b_f}{2 \cdot t_f} = 6.3$ $\quad \frac{65}{\sqrt{50}} = 9.2$ $\quad$ Flange is compact

$\frac{970}{\sqrt{50}} = 137.2$ $\quad \frac{h}{t_w} = 154.6$ $\quad$ Web is slender, refer to Appendix G

Therefore, Mb< Mp

$$I_x := 2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right)^2 + \frac{1}{12} \cdot t_w \cdot h^3 \qquad I_x = 15301 in^4 \qquad S_x := \frac{I_x}{0.5 \cdot d} \qquad S_x = 510 in^3$$

$$a_r := \frac{h \cdot t_w}{b_f \cdot t_f} \qquad m := \frac{55}{55} \qquad R_e := \frac{12 + a_r \cdot \left(3 \cdot m - m^3\right)}{12 + 2 \cdot a_r} \qquad R_e = 1$$

BEAM MOMENT STRENGTH, Mb (CONT'D)

$I_T := \frac{1}{12}\cdot\left(\frac{0.5\cdot d}{3}\right)\cdot t_w^{3} + \frac{1}{12}\cdot t_f\cdot b_f^{3}$ $\quad I_T = 26.9in^4$

$A_T := \left(\frac{0.5\cdot d}{3}\right)\cdot t_w + t_f\cdot b_f$ $\quad A_T = 8.8in^2$ $\quad r_T := \sqrt{\frac{I_T}{A_T}}$ $\quad r_T = 1.7in$

Required L_b: $L_b := 40.4 r_T$ $\quad L_b = 5.9ft$ $\quad$ If the unbraced length is less than L_b: $F_{cr} := F_{yf}$

$$R_{PG} := 1 - \left(\frac{a_r}{1200 + 300\cdot a_r}\right)\cdot\left(\frac{h}{t_w} - 5.7\cdot\sqrt{\frac{29000ksi}{F_{cr}}}\right)$$

$M_b := S_x\cdot R_{PG}\cdot R_e\cdot F_{cr}$ $\quad M_b = 2661k\cdot ft$

END-PLATE STRENGTH, Mpl

$d_0 := d - \frac{t_f}{2} + p_{fo}$ $\quad d_0 = 61.7in$ $\quad h_0 := d_0 + \frac{t_f}{2}$ $\quad h_0 = 62in$

$d_1 := d - \frac{t_f}{2} - t_f - p_{fi}$ $\quad d_1 = 57in$ $\quad h_1 := d_1 + \frac{t_f}{2}$ $\quad h_1 = 57.3in$

$s := \frac{1}{2}\cdot\sqrt{b_p\cdot g}$ $\quad s = 2.85in$

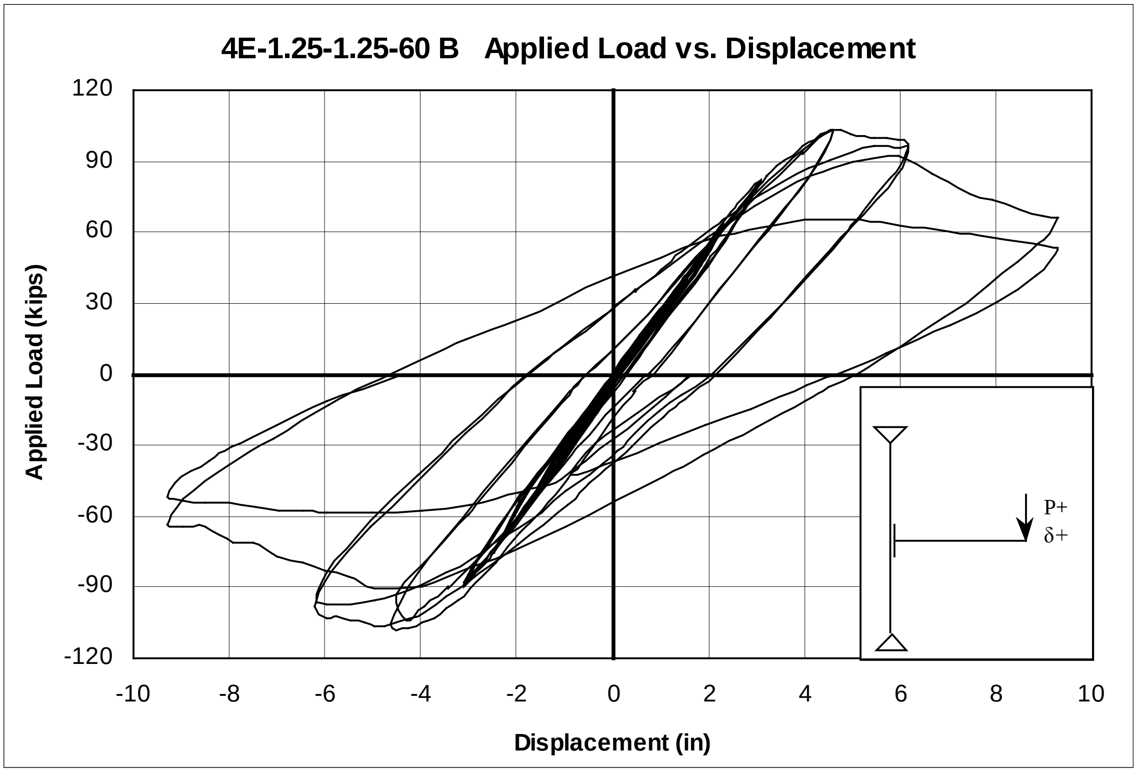
$$Y := \frac{b_p}{2}\cdot\left[h_1\cdot\left(\frac{1}{p_{fi}} + \frac{1}{s}\right) + h_0\cdot\left(\frac{1}{p_{fo}}\right) - \frac{1}{2}\right] + \frac{2}{g}\cdot\left[h_1\cdot\left(p_{fi} + s\right)\right]$$

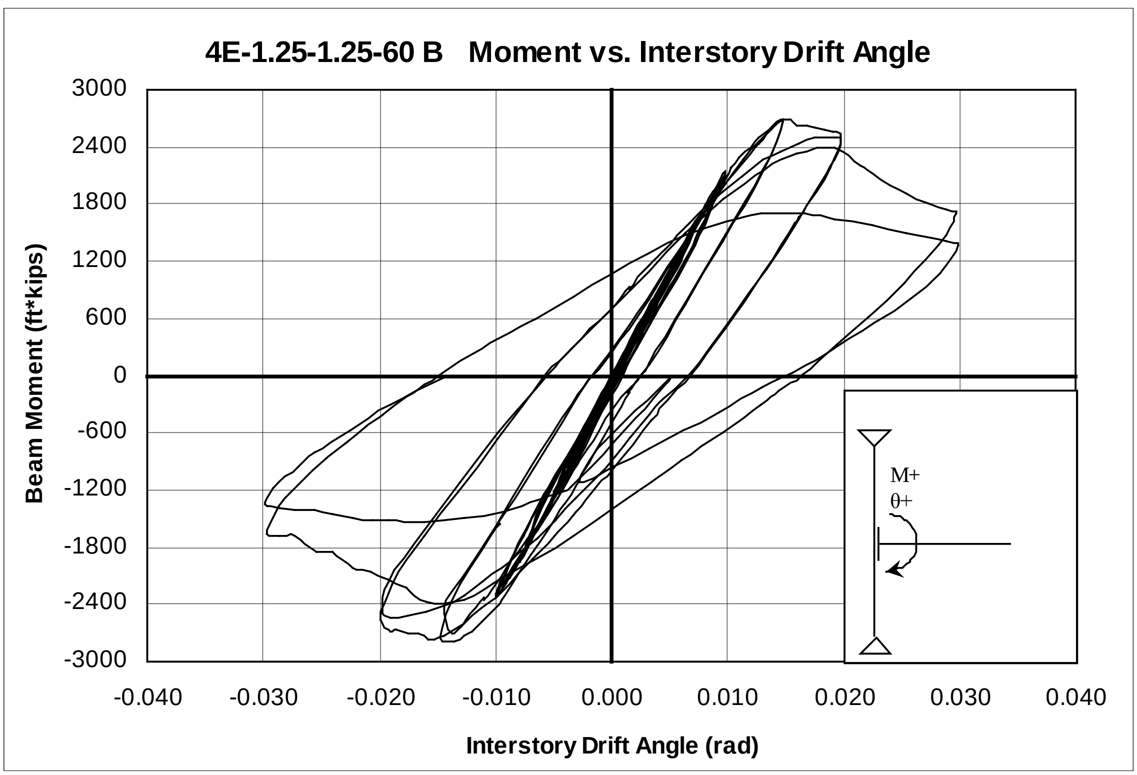
$Y = 454in$

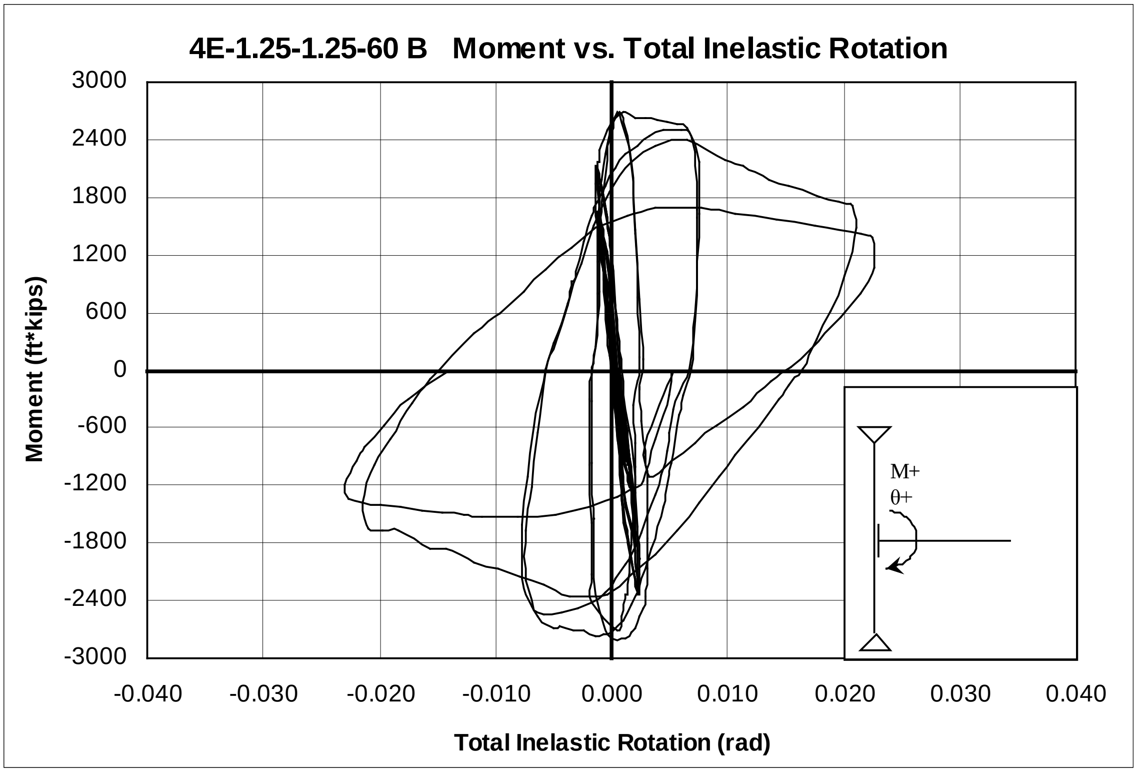
$M_{PL} := t_p^{2}\cdot F_{yp}\cdot Y$ $\quad M_{PL} = 3466k\cdot ft$

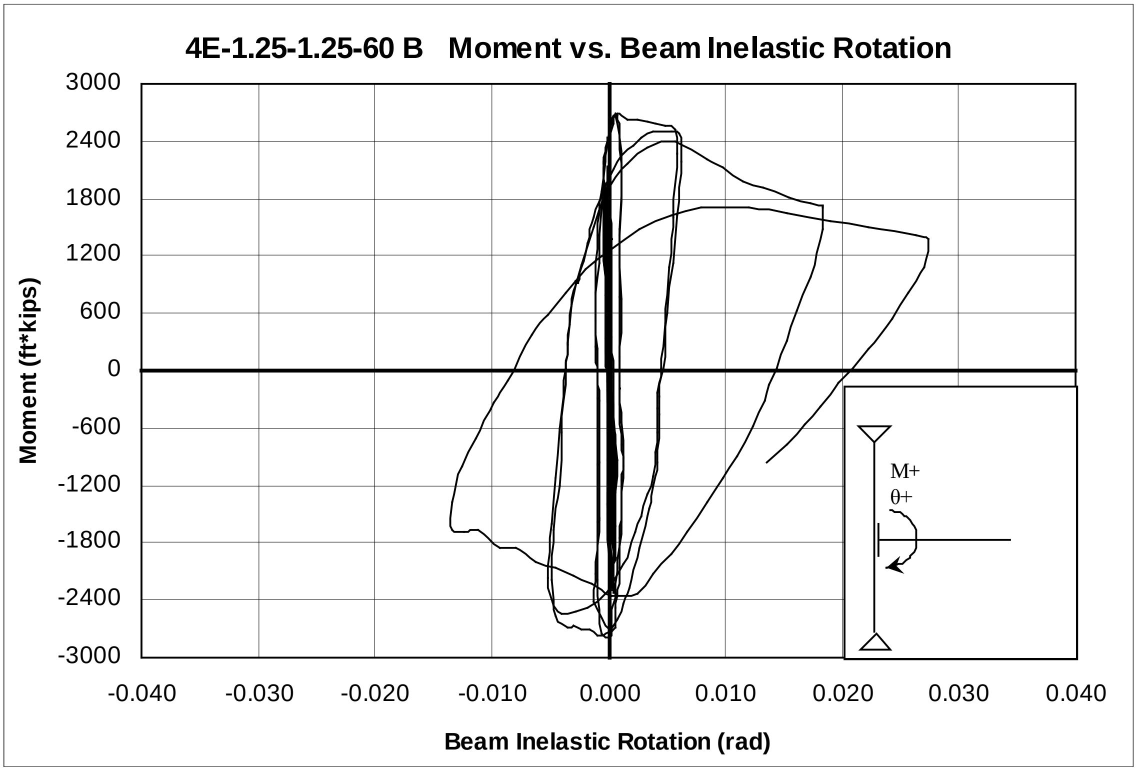
BOLT STRENGTH WITHOUT PRYING, Mnp

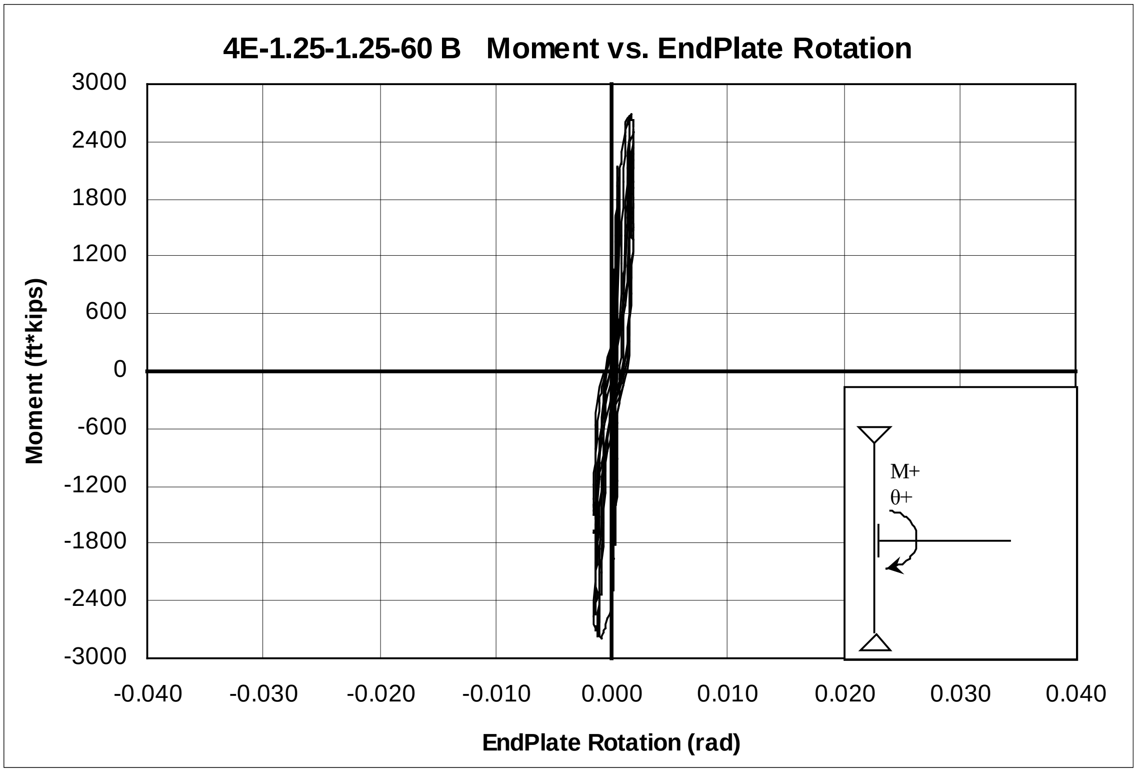
$M_{NP} := 2\cdot P_t\cdot\left(d_0 + d_1\right)$ $\quad M_{NP} = 2749.7k\cdot ft$

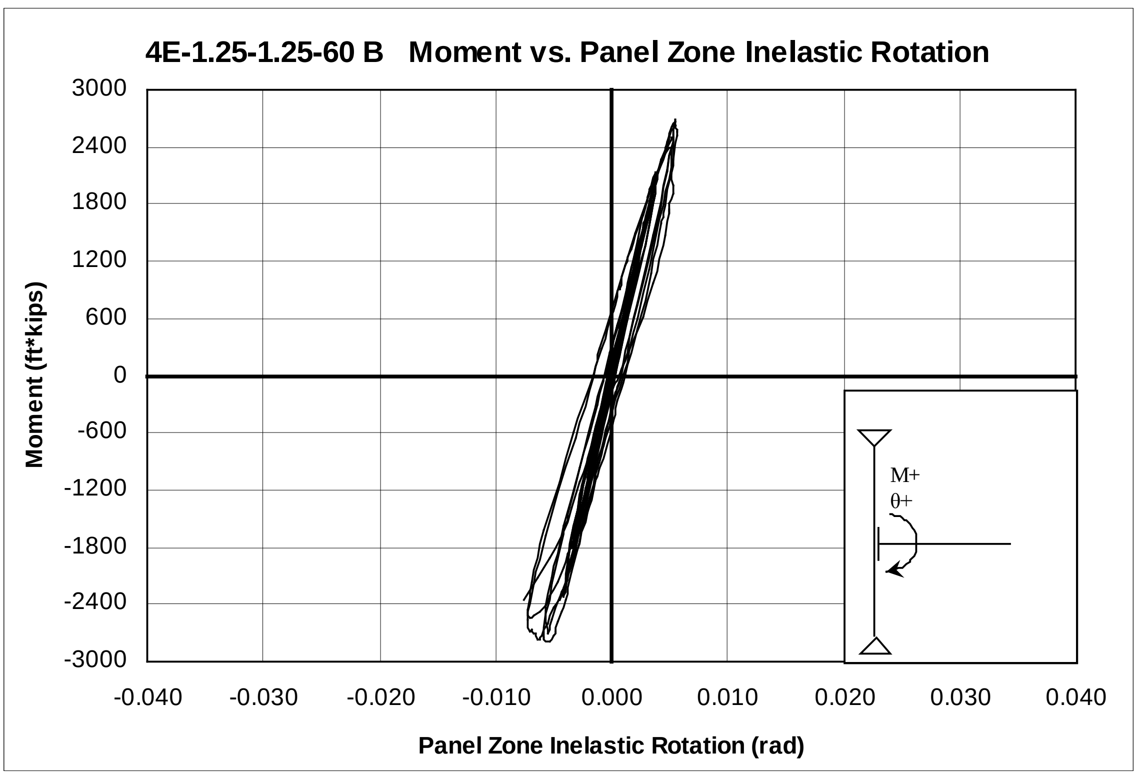

4E-1.25-1.25-60 B Applied Load vs. Displacement
Applied Load (kips)
Displacement (in)
120
90
60
30
0
-30
-60
-90
-120
-10
-8
-6
-4
-2
0
2
4
6
8
10
P+
δ+

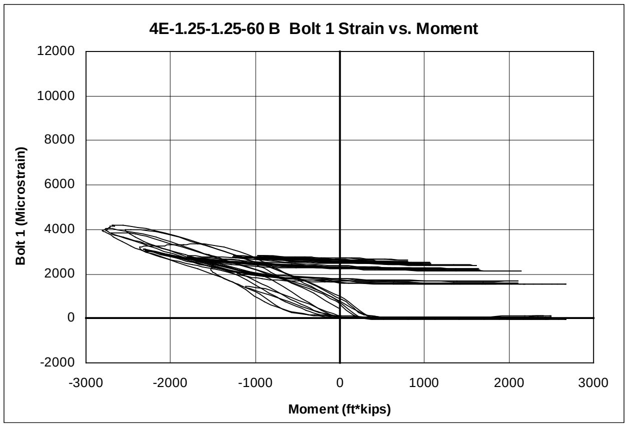

4E-1.25-1.25-60 B Moment vs. Interstory Drift Angle
Beam Moment (ft*kips)
Interstory Drift Angle (rad)
3000
2400
1800
1200
600
0
-600
-1200
-1800
-2400
-3000
-0.040
-0.030
-0.020
-0.010
0.000
0.010
0.020
0.030
0.040
M+
θ+

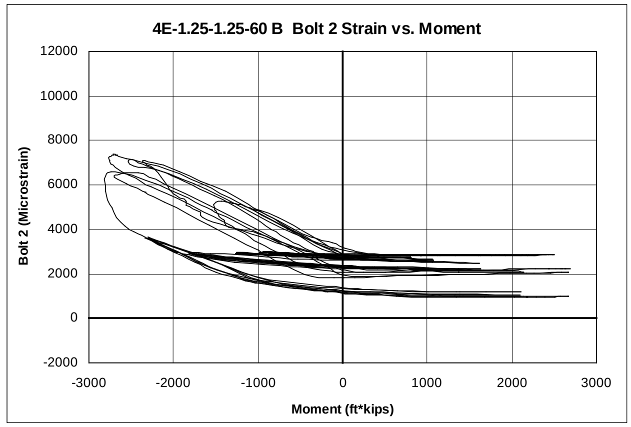

4E-1.25-1.25-60 B Moment vs. Total Inelastic Rotation
3000
2400
1800
1200
600
0
-600
-1200
-1800
-2400
-3000
Moment (ft*kips)
-0.040
-0.030
-0.020
-0.010
0.000
0.010
0.020
0.030
0.040
Total Inelastic Rotation (rad)
M+
θ+

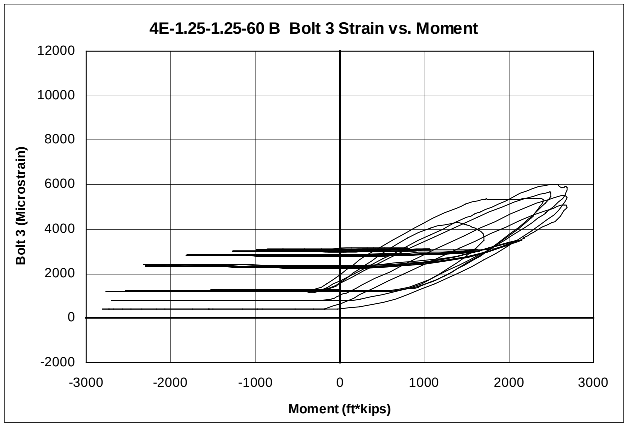

4E-1.25-1.25-60 B Moment vs. Beam Inelastic Rotation
3000
2400
1800
1200
600
0
-600
-1200
-1800
-2400
-3000
Moment (ft*kips)
-0.040
-0.030
-0.020
-0.010
0.000
0.010
0.020
0.030
0.040
Beam Inelastic Rotation (rad)
M+
θ+

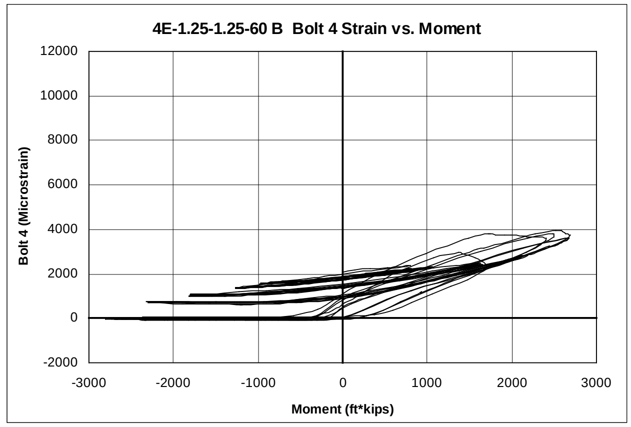

4E-1.25-1.25-60 B Moment vs. EndPlate Rotation
Moment (ft*kips)
3000
2400
1800
1200
600
0
-600
-1200
-1800
-2400
-3000
-0.040
-0.030
-0.020
-0.010
0.000
0.010
0.020
0.030
0.040
EndPlate Rotation (rad)
M+
θ+

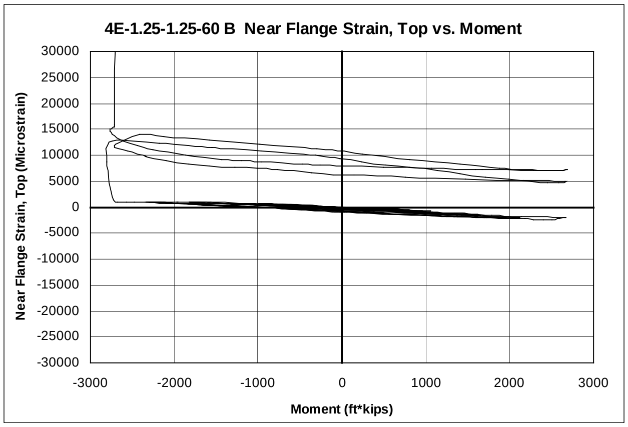

4E-1.25-1.25-60 B Moment vs. Panel Zone Inelastic Rotation
Moment (ft*kips)
3000
2400
1800
1200
600
0
-600
-1200
-1800
-2400
-3000
-0.040
-0.030
-0.020
-0.010
0.000
0.010
0.020
0.030
0.040
Panel Zone Inelastic Rotation (rad)
M+
θ+

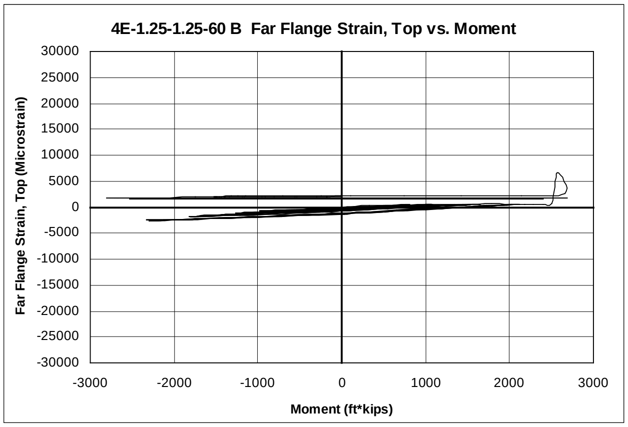

4E-1.25-1.25-60 B Bolt 1 Strain vs. Moment
Bolt 1 (Microstrain)
Moment (ft*kips)
12000
10000
8000
6000
4000
2000
0
-2000
-3000
-2000
-1000
0
1000
2000
3000

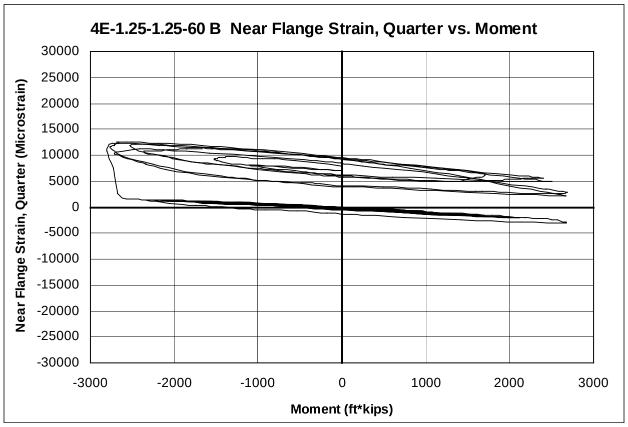

4E-1.25-1.25-60 B Bolt 2 Strain vs. Moment
Bolt 2 (Microstrain)
Moment (ft*kips)
12000
10000
8000
6000
4000
2000
0
-2000
-3000
-2000
-1000
0
1000
2000
3000

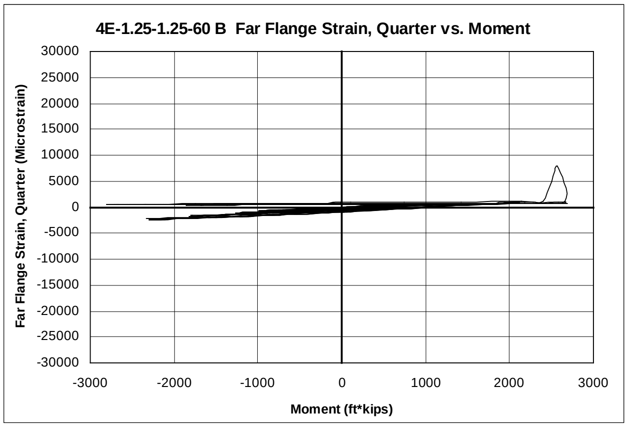

4E-1.25-1.25-60 B Bolt 3 Strain vs. Moment
Bolt 3 (Microstrain)
Moment (ft*kips)
12000
10000
8000
6000
4000
2000
0
-2000
-3000
-2000
-1000
0
1000
2000
3000

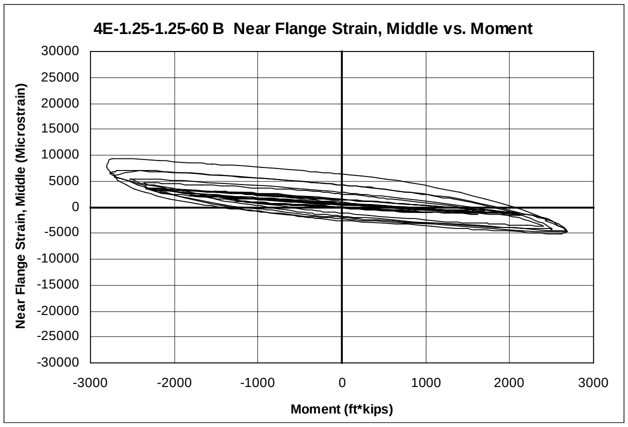

4E-1.25-1.25-60 B Bolt 4 Strain vs. Moment
Bolt 4 (Microstrain)
Moment (ft*kips)
12000
10000
8000
6000
4000
2000
0
-2000
-3000
-2000
-1000
0
1000
2000
3000

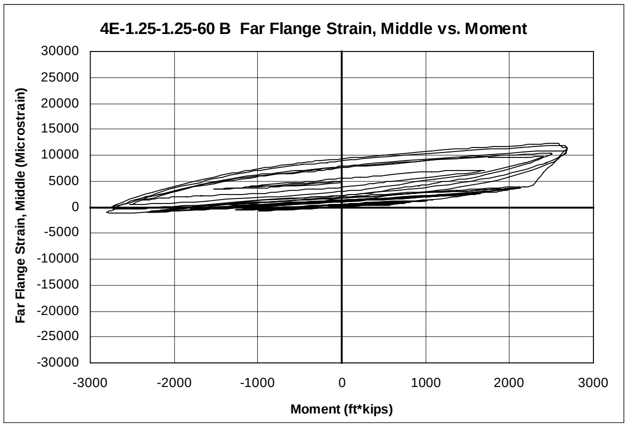

4E-1.25-1.25-60 B Near Flange Strain, Top vs. Moment
Near Flange Strain, Top (Microstrain)
Moment (ft*kips)


4E-1.25-1.25-60 B Far Flange Strain, Top vs. Moment
Far Flange Strain, Top (Microstrain)
Moment (ft*kips)


4E-1.25-1.25-60 B Near Flange Strain, Quarter vs. Moment
Near Flange Strain, Quarter (Microstrain)
Moment (ft*kips)

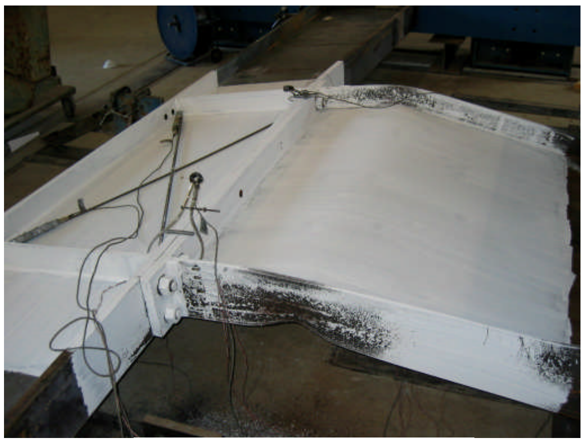

4E-1.25-1.25-60 B Far Flange Strain, Quarter vs. Moment
Far Flange Strain, Quarter (Microstrain)
Moment (ft*kips)


4E-1.25-1.25-60 B Near Flange Strain, Middle vs. Moment
Near Flange Strain, Middle (Microstrain)
Moment (ft*kips)


4E-1.25-1.25-60 B Far Flange Strain, Middle vs. Moment
Far Flange Strain, Middle (Microstrain)
Moment (ft*kips)

Near Flange

Far Flange



Top View

4E-1.25-1.25-B AFTER TESTING

APPENDIX H

MRE1/2 – 1.0 – 1.0 – 60 A

RESULTS AND TEST DATA

SPECIMEN PROPERTIES & TEST SUMMARY

TEST NAME: MRE1/2-1.0-1.0-60 A
TEST DATE: February 20, 2004

BEAM DATA

DEPTH, d:	60.00	in
FLANGE WIDTH, b_f:	8.00	in
FLANGE THICKNESS, t_f:	0.63	in
WEB THICKNESS, t_w:	0.38	in
FLANGE YIELD STRESS, F_{yf}:	66.8	ksi
FLANGE ULTIMATE STRESS, F_{uf}:	90.5	ksi
WEB YIELD STRESS, F_{yw}:	66.0	ksi
WEB ULTIMATE STRESS, F_{uw}:	77.1	ksi

END-PLATE DATA

END-PLATE THICKNESS, t_p:	1.04	in
END-PLATE WIDTH, b_p:	8.03	in
END-PLATE LENGTH, L_p:	68.00	in
END-PLATE EXTENSION OUTSIDE FLANGE, p_{ext}:	3.99	in
OUTER PITCH, BOLT TO FLANGE, p_{fo}:	2.00	in
INNER PITCH, BOLT TO FLANGE, p_{fi}:	2.02	in
PITCH, BOLT TO BOLT, p_b:	3.00	in
GAGE, g:	4.04	in
END-PLATE YIELD STRESS, F_{yp}:	62.0	ksi
END-PLATE ULTIMATE STRESS, F_{up}:	86.2	ksi

BOLT DATA

BOLT DIAMETER, d_b:	1.00	in
BOLT LENGTH, L_b:	3.25	in
BOLT TYPE:	A325	
NOMINAL BOLT TENSILE STRESS (AISC J3.6), F_t:	90	ksi
NOMINAL BOLT TENSILE STRENGTH, P_t:	71	kips
BOLT PRETENSION, T_b:	51	kips

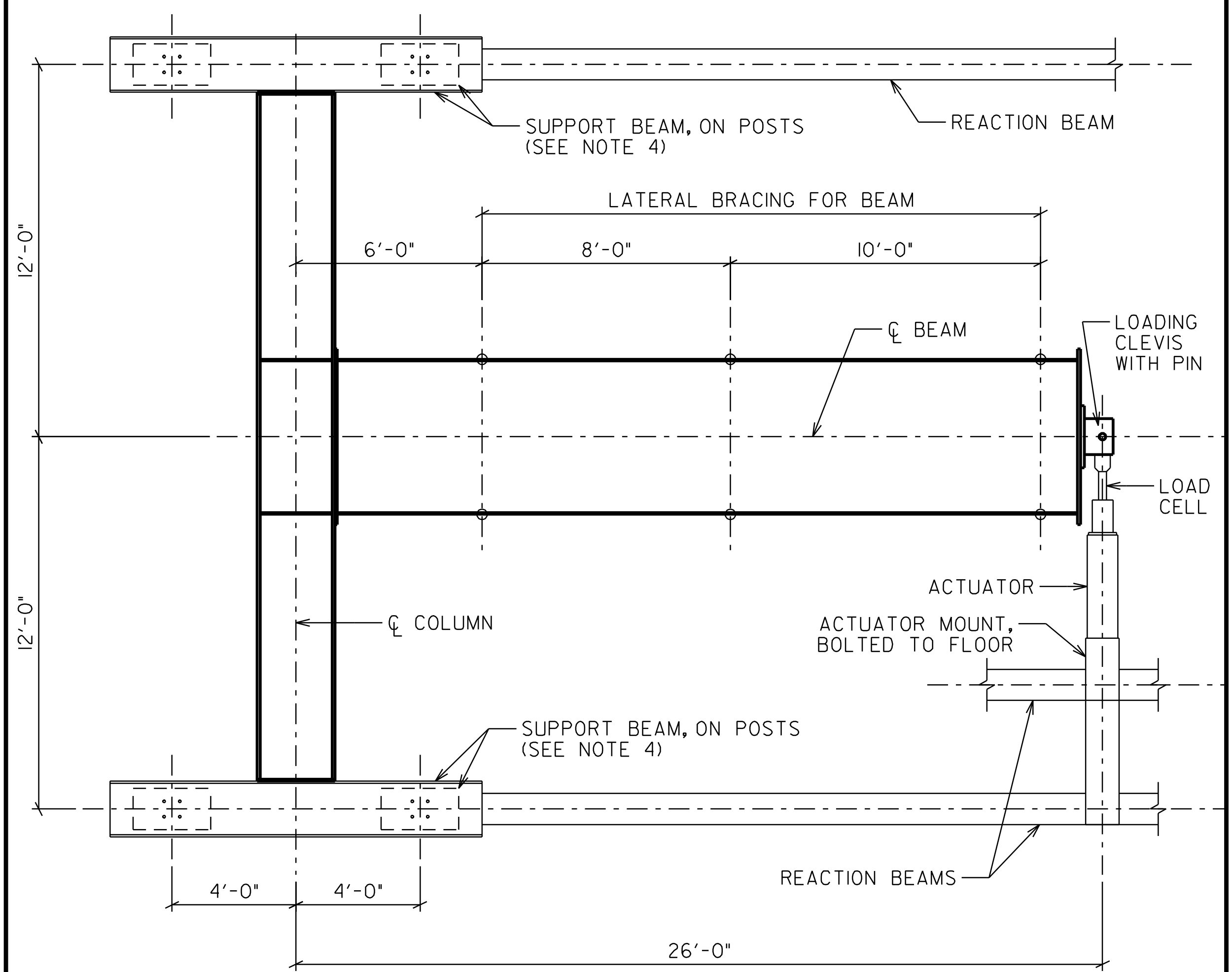
EXPERIMENTAL RESULTS

FAILURE MODE:	Bolt Tension Rupture & Weld Rupture	
MAXIMUM MOMENT AT COLUMN CENTERLINE, $M_{max,cc}$:	2310.6	ft-kips
MAXIMUM MOMENT AT FAYING SURFACE, $M_{max,fs}$:	2199.5	ft-kips
MAXIMUM INTERSTORY DRIFT ANGLE, θ_{max}:	0.014	rad
MAXIMUM INTERSTORY DRIFT ANGLE AT 80% 0F $M_{max,cc}$, θ_{preq}:	0.014	rad

CALCULATED STRENGTHS

BEAM MOMENT STRENGTH [1], M_b:	2661.0	ft-kips
BEAM PLASTIC STRENGTH [1], M_p:	3468.5	ft-kips
BEAM EXPECTED PLASTIC STRENGTH, M_{pe}:	3161.2	ft-kips
END-PLATE STRENGTH [1], M_{PL}:	2979.2	ft-kips
BOLT TENSION RUPTURE (w/o Prying), M_{NP}:	2044.3	ft-kips

1. Measured material properties used.



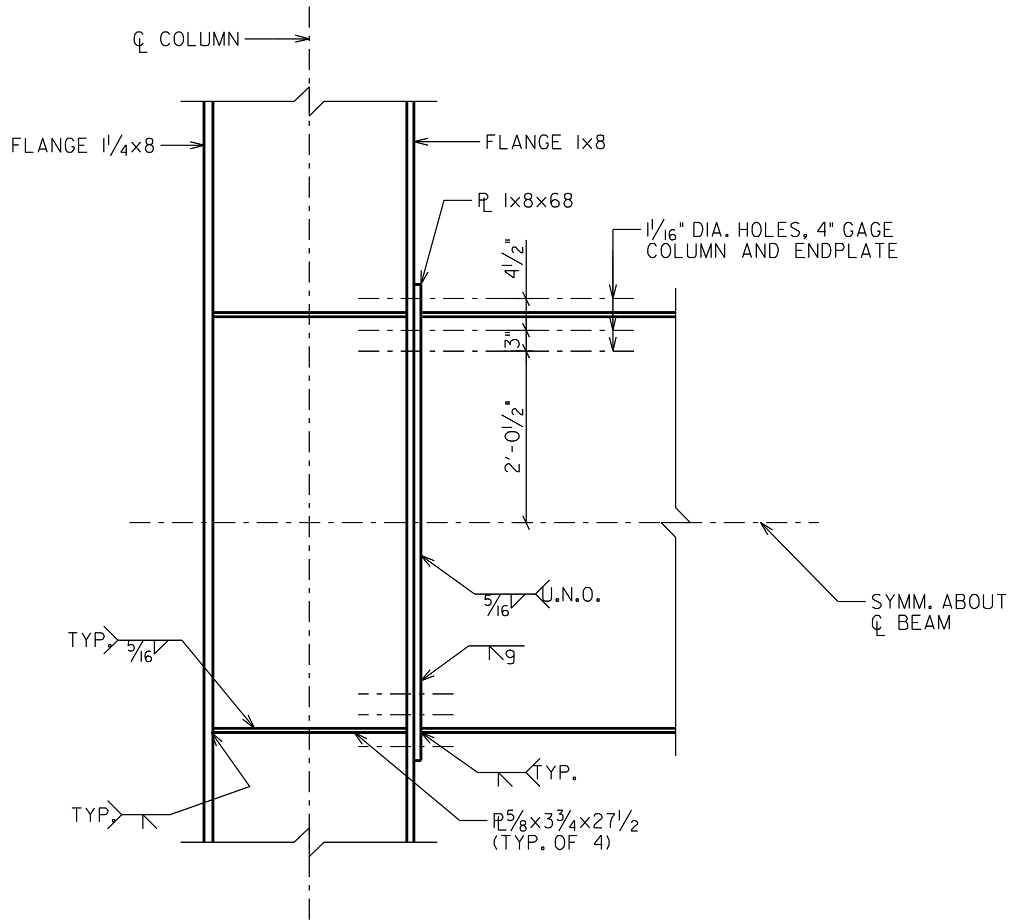
MRE1/2-1.0-1.0-60 A TEST SUBASSEMBLAGE PLAN

BEAM
WEB = 3/8×58 3/4
FLANGE = 5/8×8
LENGTH = 288 1/2 (INCLUDES ENDPLATES)

COLUMN
WEB = 1/2×27 3/4
FLANGES = 1 1/4×8 & 1×8
LENGTH = 266 1/2 (INCLUDES ENDPLATES)

NOTES
1. COLUMN, BEAM, AND CONNECTION STEEL, A572 Gr50.
2. ALL BOLTS, A325.
3. ALL WELDING, AWS D1.1, E70XX ELECTRODES.
4. COLUMN BOLTED TO SUPPORT BEAM. SUPPORT BEAM BOLTED POSTS. POSTS BOLTED TO REACTION BEAM.

SPECIMEN: MRE1/2-1.0-1.0-60 A	DRAWN BY: SEB



MREI/2-1.0-1.0-60 A
CONNECTION DETAIL

SPECIMEN: MREI/2-1.0-1.0-60 A	DRAWN BY: SEB

SAC PROTOCOL LOADING HISTORY

Test Name: MRE1/2-1.0-1.0-60 A   Test By: SEB   Date: 2/21/2004

Load Step	Cycle	Max. Pos. Load (kips)	Max. Disp. (in)	Max. Neg. Load (kips)	Max. Disp. (in)	Comments
I (0.00375 rad)	1	30.1	1.157	-33.1	-1.177	
δ=1.163"	2	31.0	1.178	-32.3	-1.158	
	3	30.7	1.168	-32.3	-1.157	
	4	30.3	1.163	-32.0	-1.165	
	5	30.3	1.166	-31.7	-1.169	
	6	30.4	1.167	-31.9	-1.168	
		Permanent Set =	-0.055"			
II (0.005 rad)	1	38.5	1.554	-42.0	-1.565	
δ=1.55"	2	37.7	1.557	-42.9	-1.558	
	3	37.9	1.553	-43.2	-1.551	
	4	38.0	1.556	-42.9	-1.555	
	5	37.8	1.553	-43.5	-1.563	
	6	37.8	1.551	-42.5	-1.551	
		Permanent Set =	-0.034"			
III (0.0075 rad)	1	53.9	2.328	-62.1	-2.327	
δ=2.325"	2	51.7	2.339	-64.8	-2.328	
	3	49.2	2.334	-65.7	-2.314	<-- Restored position of Lat. Trans.
	4	57.7	2.329	-58.4	-2.327	Plunger (was jostled due to bolt
	5	58.1	2.331	-58.7	-2.341	banging). Also tightened bolts on
	6	57.8	2.353	-58.4	-2.330	support frame to eliminate banging.
		Permanent Set =	-0.220"			
IV (0.01 rad)	1	74.4	3.111	-76.8	-3.104	
δ=3.1"	2	72.3	3.101	-76.7	-3.118	
	3	72.3	3.101	-76.3	-3.108	
	4	72.8	3.118	-75.3	-3.106	
		Permanent Set =	-.265"			
V (0.015 rad)	1	~ 90	~ 4.3			4 outermost bolts ruptured, along
δ=4.65"	2					with 13" of panel zone weld on Col.
		Permanent Set =				
VI (0.02 rad)	1					
	2					
		Permanent Set =				
VII (0.03 rad)	1					
	2					
		Permanent Set =				
VIII (0.04 rad)	1					
	2					
		Permanent Set =				
IX (0.05 rad)	1					
	2					
		Permanent Set =				

CALCULATED STRENGTHS

TEST NAME: MRE1/2-1.0-1.0-60 A
TEST DATE: February 21, 2004

Specified Grade for all Steel = ASTM A572, Grade 50

MEASURED PROPERTIES

$d := 60 \cdot in$ $\quad F_{yf} := 66.8 \cdot ksi$ $\quad t_p := 1.04 \cdot in$ $\quad p_{fo} := 2.00 \cdot in$ $\quad p_b := 3.00 \cdot in$

$b_f := 8.00 \cdot in$ $\quad F_{uf} := 90.5 \cdot ksi$ $\quad b_p := 8.03 \cdot in$ $\quad p_{fi} := 2.02 \cdot in$

$t_f := 0.63 \cdot in$ $\quad F_{yw} := 66.0 \cdot ksi$ $\quad L_p := 68.00 \cdot in$ $\quad g := 4.04 \cdot in$

$t_w := 0.38 \cdot in$ $\quad F_{uw} := 77.1 \cdot ksi$ $\quad p_{ext} := 3.99 \cdot in$ $\quad F_{yp} := 62.0 \cdot ksi$ $\quad h := d - 2 \cdot t_f$

SPECIFIED PROPERTIES

$R_y := 1.1$ $\quad P_t := 71 \cdot k$

BEAM PLASTIC STRENGTH, Mp

$$M_p := 2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right) \cdot F_{yf} + 2 \cdot t_w \cdot \frac{h}{2} \cdot \frac{h}{4} \cdot F_{yw} \qquad M_p = 3468.5 k \cdot ft$$

BEAM EXPECTED PLASTIC STRENGTH, Mpr

$$M_{pe} := 1.1 \cdot R_y \cdot \left[2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right) \cdot 50 \cdot ksi + 2 \cdot t_w \cdot \frac{h}{2} \cdot \frac{h}{4} \cdot 50 \cdot ksi\right] \qquad M_{pe} = 3161.2 k \cdot ft$$

BEAM MOMENT STRENGTH, Mb

$\frac{b_f}{2 \cdot t_f} = 6.3$ $\quad \frac{65}{\sqrt{50}} = 9.2$ $\quad$ Flange is compact

$\frac{970}{\sqrt{50}} = 137.2$ $\quad \frac{h}{t_w} = 154.6$ $\quad$ Web is slender, refer to Appendix G

Therefore, Mb < Mp

$$I_x := 2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right)^2 + \frac{1}{12} \cdot t_w \cdot h^3 \qquad I_x = 15301 in^4 \qquad S_x := \frac{I_x}{0.5 \cdot d} \qquad S_x = 510 in^3$$

$$a_r := \frac{h \cdot t_w}{b_f \cdot t_f} \qquad m := \frac{55}{55} \qquad R_e := \frac{12 + a_r \cdot \left(3 \cdot m - m^3\right)}{12 + 2 \cdot a_r} \qquad R_e = 1$$

BEAM MOMENT STRENGTH, Mb (CONT'D)

$I_T := \frac{1}{12}\cdot\left(\frac{0.5\cdot d}{3}\right)\cdot t_w^3 + \frac{1}{12}\cdot t_f\cdot b_f^3$ $\quad I_T = 26.9\,in^4$

$A_T := \left(\frac{0.5\cdot d}{3}\right)\cdot t_w + t_f\cdot b_f$ $\quad A_T = 8.8\,in^2$ $\quad r_T := \sqrt{\frac{I_T}{A_T}}$ $\quad r_T = 1.7\,in$

Required L_b: $L_b := 40.4\,r_T$ $\quad L_b = 5.9\,ft$ $\quad$ If the unbraced length is less than L_b: $F_{cr} := F_{yf}$

$$R_{PG} := 1 - \left(\frac{a_r}{1200 + 300\cdot a_r}\right)\cdot\left(\frac{h}{t_w} - 5.7\cdot\sqrt{\frac{29000\,ksi}{F_{cr}}}\right)$$

$M_b := S_x\cdot R_{PG}\cdot R_e\cdot F_{cr}$ $\qquad M_b = 2661\,k\cdot ft$

END-PLATE STRENGTH, Mpl

$d_0 := d - \frac{t_f}{2} + p_{fo}$ $\quad d_0 = 61.7\,in$ $\qquad h_0 := d_0 + \frac{t_f}{2}$ $\quad h_0 = 62\,in$

$d_1 := d - \frac{t_f}{2} - t_f - p_{fi}$ $\quad d_1 = 57\,in$ $\qquad h_1 := d_1 + \frac{t_f}{2}$ $\quad h_1 = 57.4\,in$

$d_2 := d_1 - p_b$ $\quad d_2 = 54\,in$ $\qquad h_2 := d_2 + \frac{t_f}{2}$ $\quad h_2 = 54.4\,in$

$s := \frac{1}{2}\cdot\sqrt{b_p\cdot g}$ $\quad s = 2.85\,in$

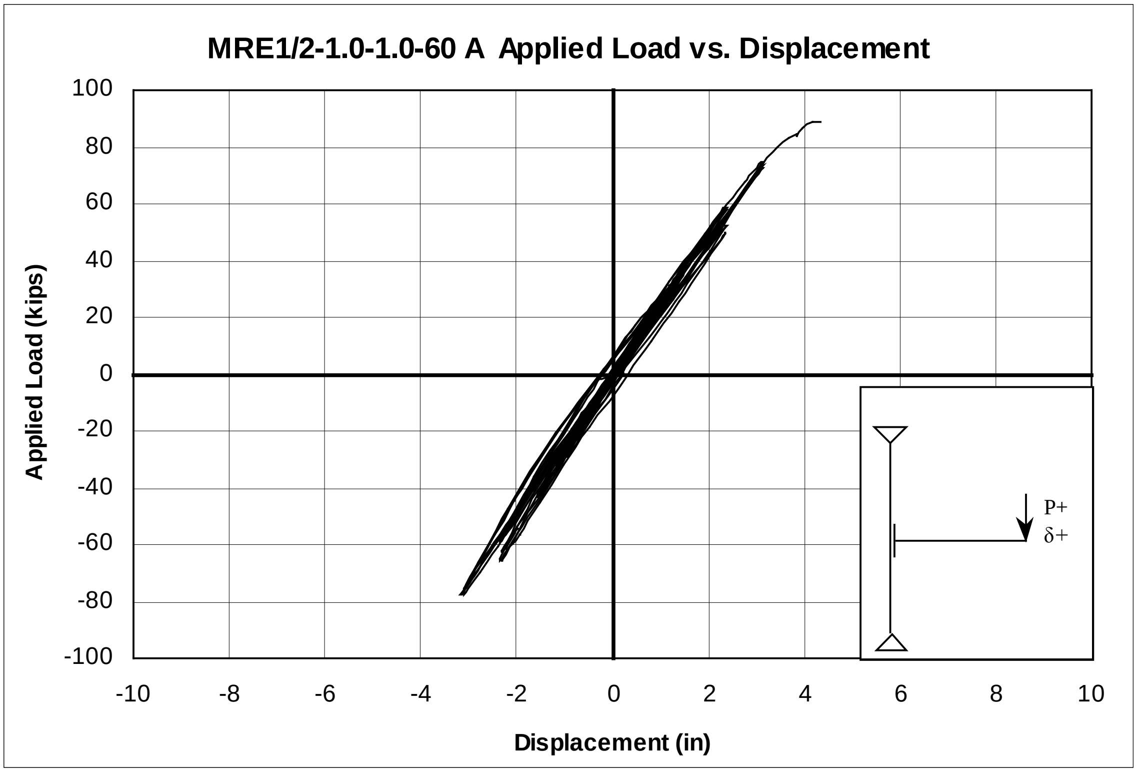
$$Y := \frac{b_p}{2}\cdot\left(\frac{h_1}{p_{fi}} + \frac{h_2}{s} + \frac{h_0}{p_{fo}} - \frac{1}{2}\right) + \frac{2}{g}\cdot\left[h_1\cdot\left(p_{fi} + 0.75\cdot p_b\right) + h_2\cdot\left(s + 0.25\cdot p_b\right)\right] + \frac{g}{2}$$

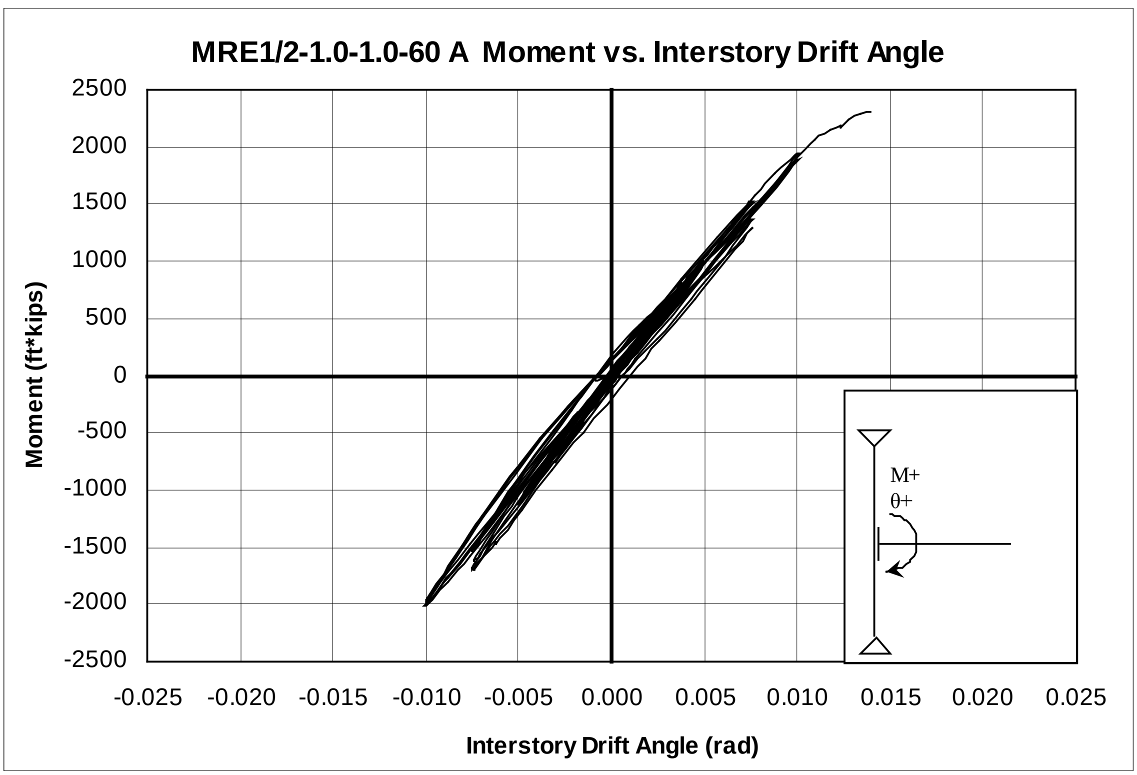
$Y = 533.1\,in$

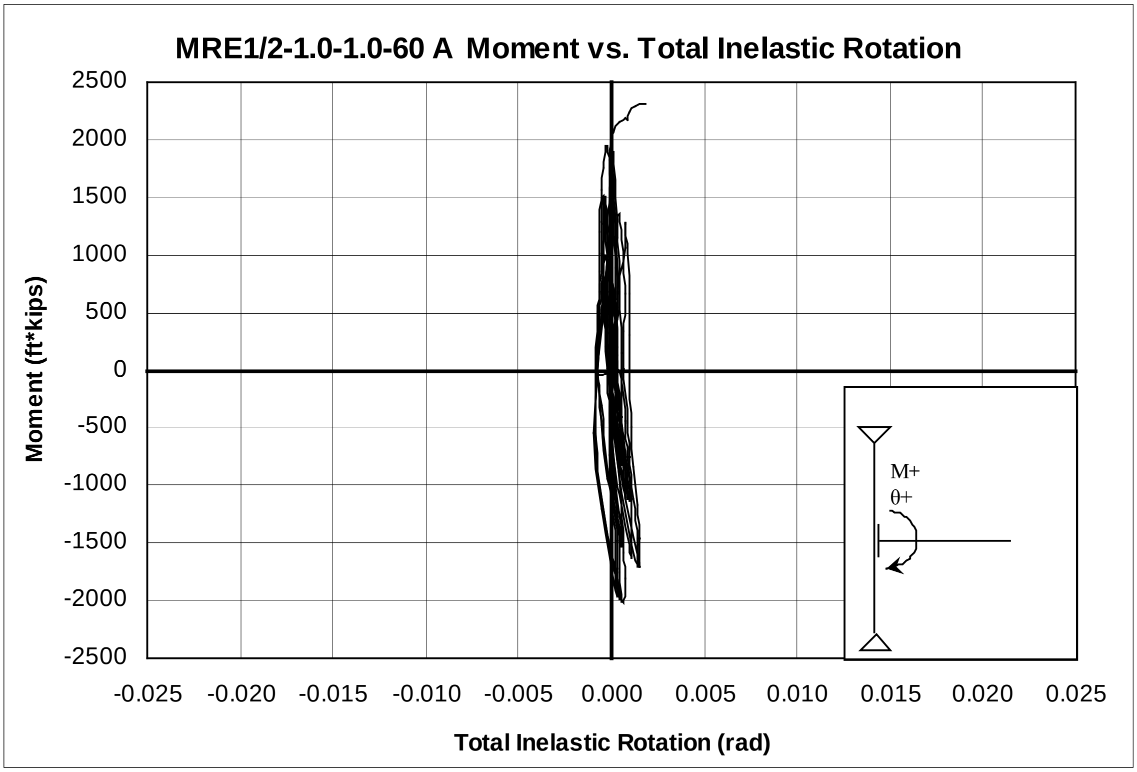
$M_{PL} := t_p^2\cdot F_{yp}\cdot Y$ $\qquad M_{PL} = 2979.2\,k\cdot ft$

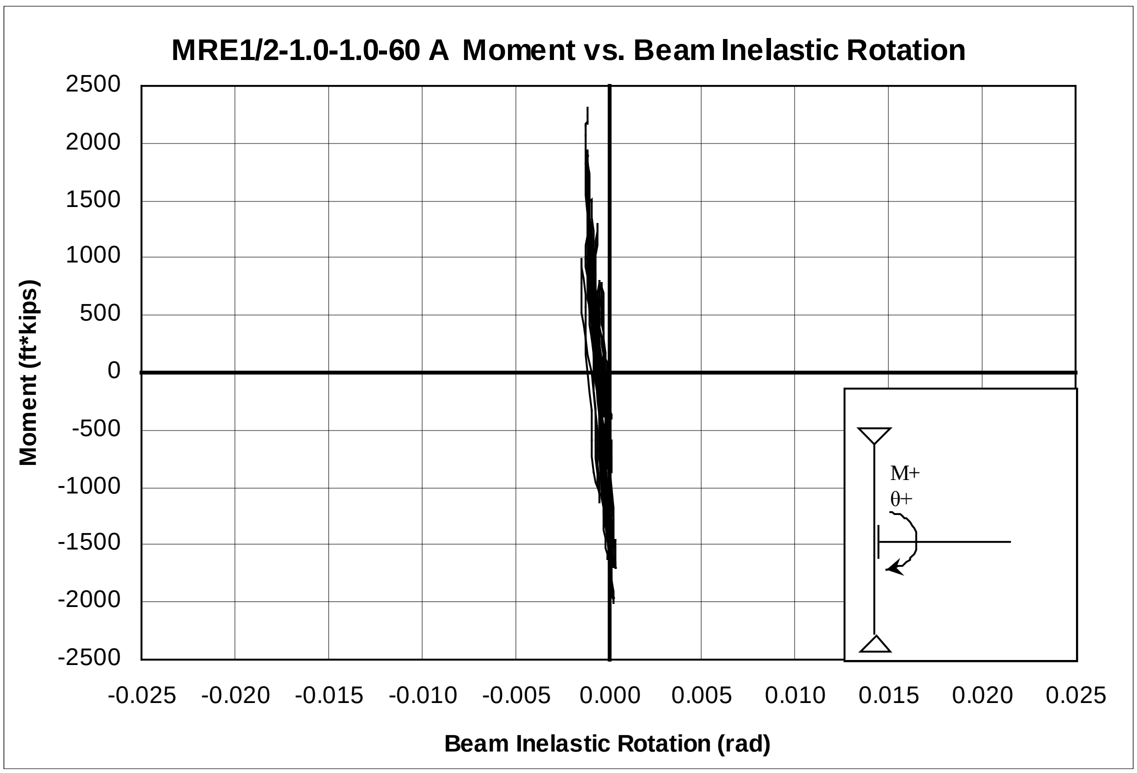
BOLT STRENGTH WITHOUT PRYING, Mnp

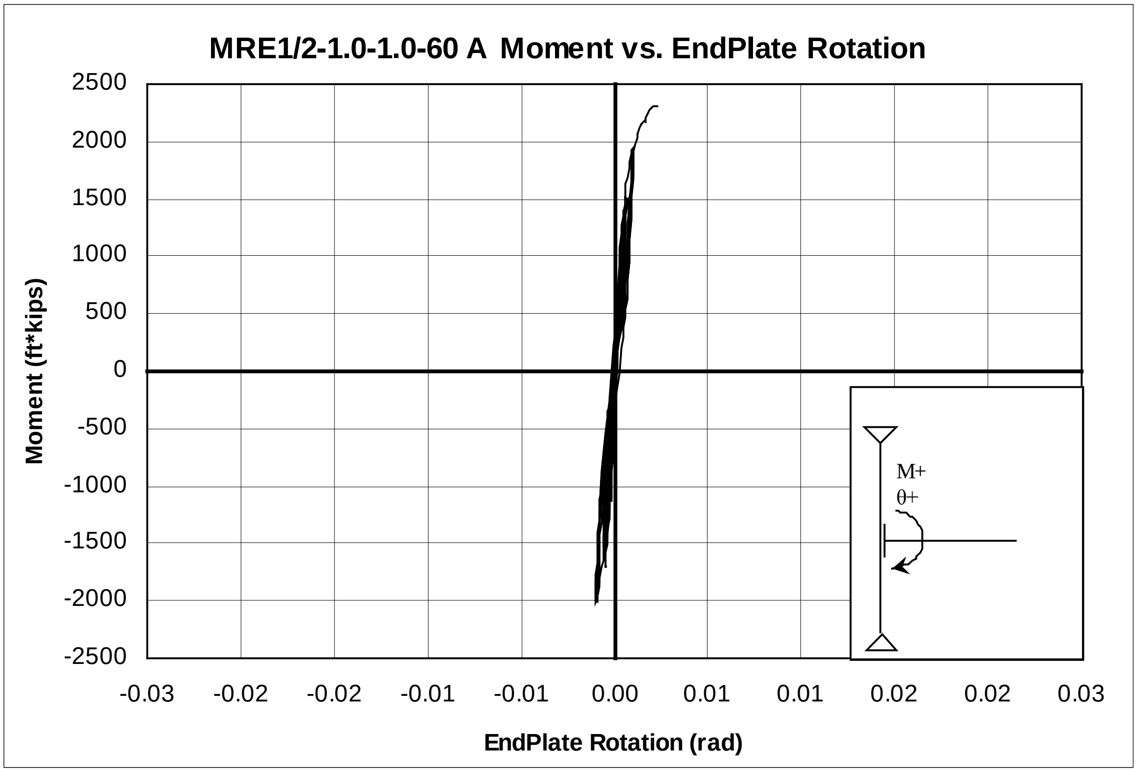
$M_{NP} := 2\cdot P_t\cdot\left(d_0 + d_1 + d_2\right)$ $\qquad M_{NP} = 2044.3\,k\cdot ft$

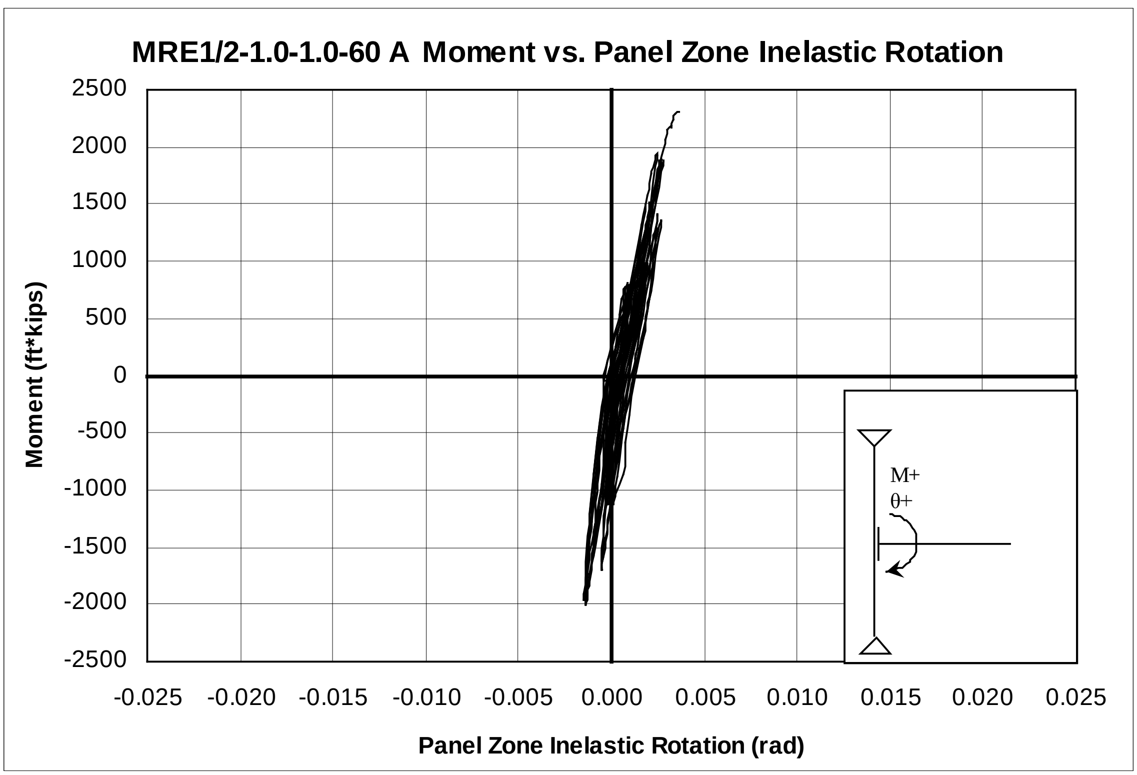

MRE1/2-1.0-1.0-60 A Applied Load vs. Displacement
Applied Load (kips)
Displacement (in)
100
80
60
40
20
0
-20
-40
-60
-80
-100
-10
-8
-6
-4
-2
0
2
4
6
8
10
P+
δ+

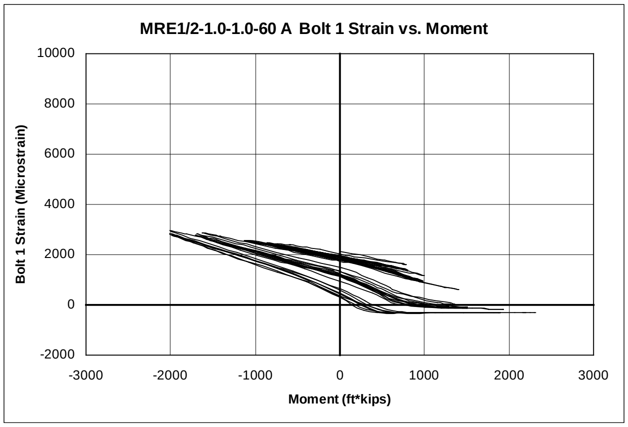

MRE1/2-1.0-1.0-60 A Moment vs. Interstory Drift Angle
Moment (ft*kips)
Interstory Drift Angle (rad)
2500
2000
1500
1000
500
0
-500
-1000
-1500
-2000
-2500
-0.025
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.010
0.015
0.020
0.025
M+
θ+

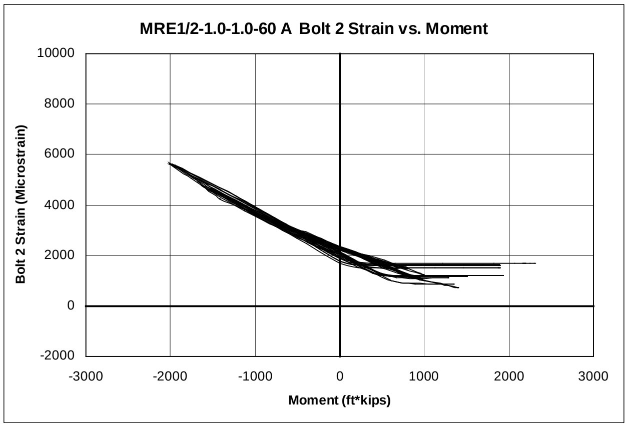

MRE1/2-1.0-1.0-60 A Moment vs. Total Inelastic Rotation
Moment (ft*kips)
2500
2000
1500
1000
500
0
-500
-1000
-1500
-2000
-2500
-0.025
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.010
0.015
0.020
0.025
Total Inelastic Rotation (rad)
M+
θ+

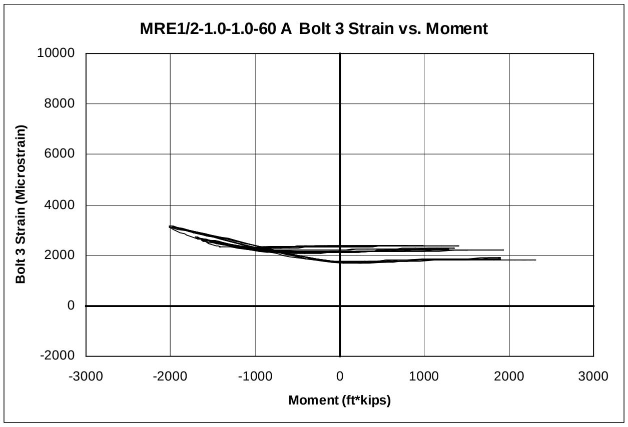

MRE1/2-1.0-1.0-60 A Moment vs. Beam Inelastic Rotation
Moment (ft*kips)
2500
2000
1500
1000
500
0
-500
-1000
-1500
-2000
-2500
-0.025
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.010
0.015
0.020
0.025
Beam Inelastic Rotation (rad)
M+
θ+

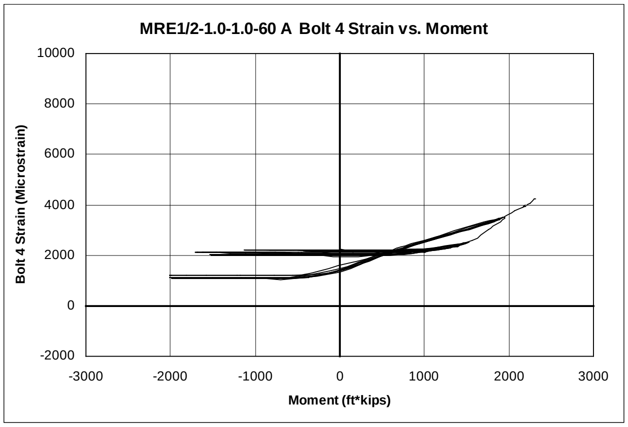

MRE1/2-1.0-1.0-60 A Moment vs. EndPlate Rotation
Moment (ft*kips)
2500
2000
1500
1000
500
0
-500
-1000
-1500
-2000
-2500
-0.03
-0.02
-0.02
-0.01
-0.01
0.00
0.01
0.01
0.02
0.02
0.03
EndPlate Rotation (rad)
M+
θ+

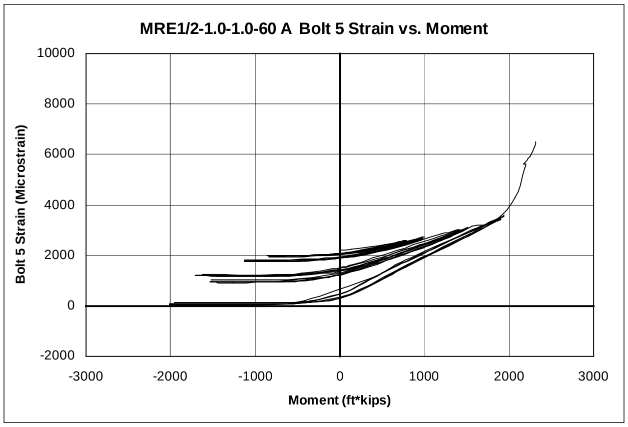

MRE1/2-1.0-1.0-60 A Moment vs. Panel Zone Inelastic Rotation
Moment (ft*kips)
2500
2000
1500
1000
500
0
-500
-1000
-1500
-2000
-2500
-0.025
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.010
0.015
0.020
0.025
Panel Zone Inelastic Rotation (rad)
M+
θ+

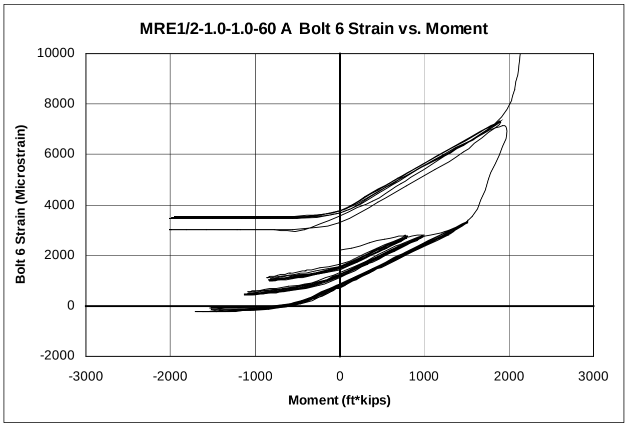

MRE1/2-1.0-1.0-60 A Bolt 1 Strain vs. Moment
Bolt 1 Strain (Microstrain)
Moment (ft*kips)

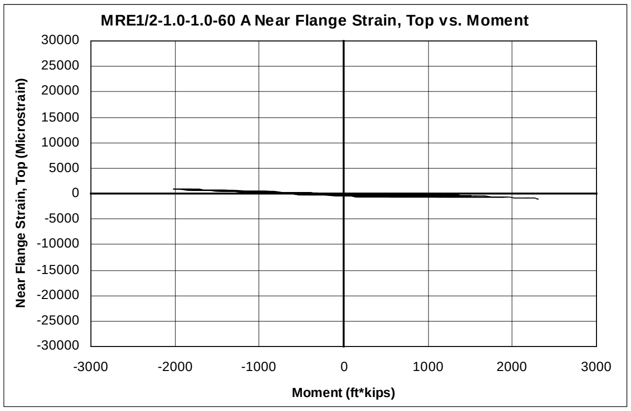

MRE1/2-1.0-1.0-60 A Bolt 2 Strain vs. Moment
Bolt 2 Strain (Microstrain)
Moment (ft*kips)

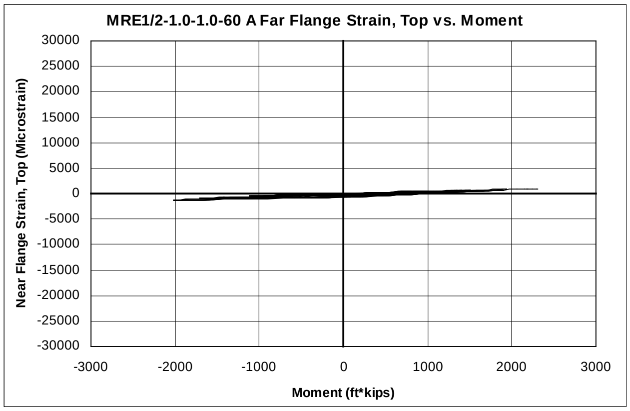

MRE1/2-1.0-1.0-60 A Bolt 3 Strain vs. Moment
Bolt 3 Strain (Microstrain)
Moment (ft*kips)

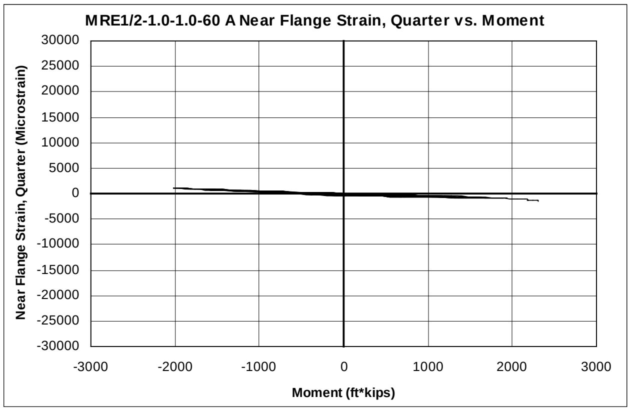

MRE1/2-1.0-1.0-60 A Bolt 4 Strain vs. Moment
Bolt 4 Strain (Microstrain)
Moment (ft*kips)

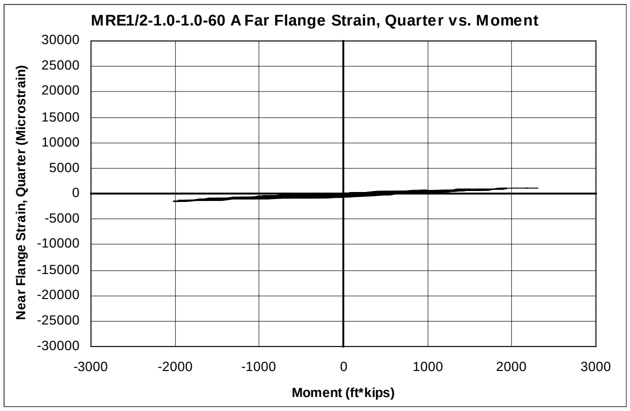

MRE1/2-1.0-1.0-60 A Bolt 5 Strain vs. Moment
Bolt 5 Strain (Microstrain)
Moment (ft*kips)

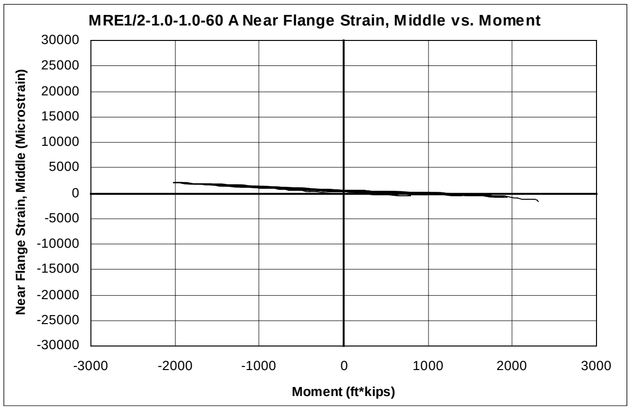

MRE1/2-1.0-1.0-60 A Bolt 6 Strain vs. Moment
Bolt 6 Strain (Microstrain)
Moment (ft*kips)

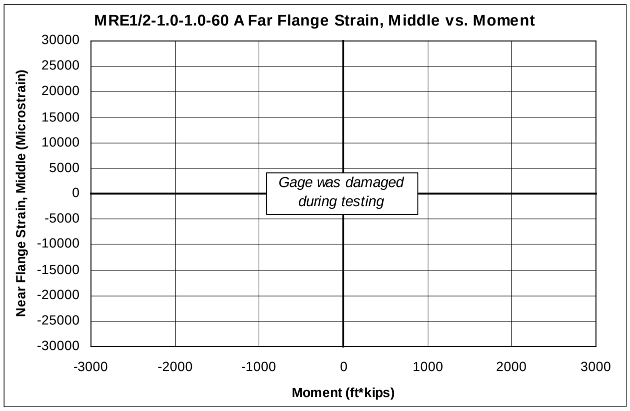

MRE1/2-1.0-1.0-60 A Near Flange Strain, Top vs. Moment
Near Flange Strain, Top (Microstrain)
Moment (ft*kips)


MRE1/2-1.0-1.0-60 A Far Flange Strain, Top vs. Moment
Near Flange Strain, Top (Microstrain)
Moment (ft*kips)


MRE1/2-1.0-1.0-60 A Near Flange Strain, Quarter vs. Moment
Near Flange Strain, Quarter (Microstrain)
Moment (ft*kips)

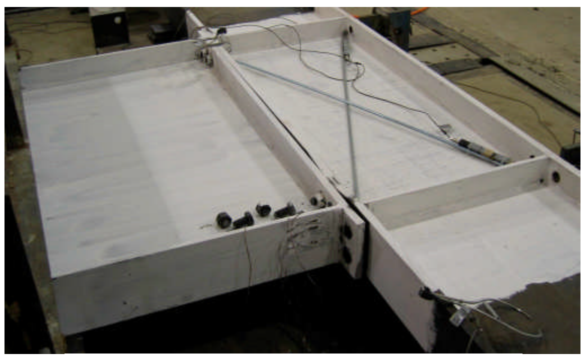

MRE1/2-1.0-1.0-60 A Far Flange Strain, Quarter vs. Moment
Near Flange Strain, Quarter (Microstrain)
Moment (ft*kips)


MRE1/2-1.0-1.0-60 A Near Flange Strain, Middle vs. Moment
Near Flange Strain, Middle (Microstrain)
Moment (ft*kips)


MRE1/2-1.0-1.0-60 A Far Flange Strain, Middle vs. Moment
Near Flange Strain, Middle (Microstrain)
Gage was damaged during testing
Moment (ft*kips)

Bolt Rupture at Far Flange

Weld Rupture at Far Flange



Overall View

MRE1/2-1.0-1.0-60 A AFTER TESTING

APPENDIX I

MRE1/2 – 1.0 – 1.0 – 60 B

RESULTS AND TEST DATA

SPECIMEN PROPERTIES & TEST SUMMARY

TEST NAME: MRE1/2-1.0-1.0-60 B
TEST DATE: April 26, 2004

BEAM DATA

DEPTH, d:	60.00	in
FLANGE WIDTH, b_f:	8.00	in
FLANGE THICKNESS, t_f:	0.63	in
WEB THICKNESS, t_w:	0.38	in
FLANGE YIELD STRESS, F_{yf}:	66.8	ksi
FLANGE ULTIMATE STRESS, F_{uf}:	90.5	ksi
WEB YIELD STRESS, F_{yw}:	66.0	ksi
WEB ULTIMATE STRESS, F_{uw}:	77.1	ksi

END-PLATE DATA

END-PLATE THICKNESS, t_p:	1.04	in
END-PLATE WIDTH, b_p:	8.03	in
END-PLATE LENGTH, L_p:	68.00	in
END-PLATE EXTENSION OUTSIDE FLANGE, p_{ext}:	3.99	in
OUTER PITCH, BOLT TO FLANGE, p_{fo}:	2.00	in
INNER PITCH, BOLT TO FLANGE, p_{fi}:	2.02	in
PITCH, BOLT TO BOLT, p_b:	3.00	in
GAGE, g:	4.04	in
END-PLATE YIELD STRESS, F_{yp}:	62.0	ksi
END-PLATE ULTIMATE STRESS, F_{up}:	86.2	ksi

BOLT DATA

BOLT DIAMETER, d_b:	1.00	in
BOLT LENGTH, L_b:	3.25	in
BOLT TYPE:	A490	
NOMINAL BOLT TENSILE STRESS (AISC J3.6), F_t:	113	ksi
NOMINAL BOLT TENSILE STRENGTH, P_t:	89	kips
BOLT PRETENSION, T_b:	64	kips

EXPERIMENTAL RESULTS

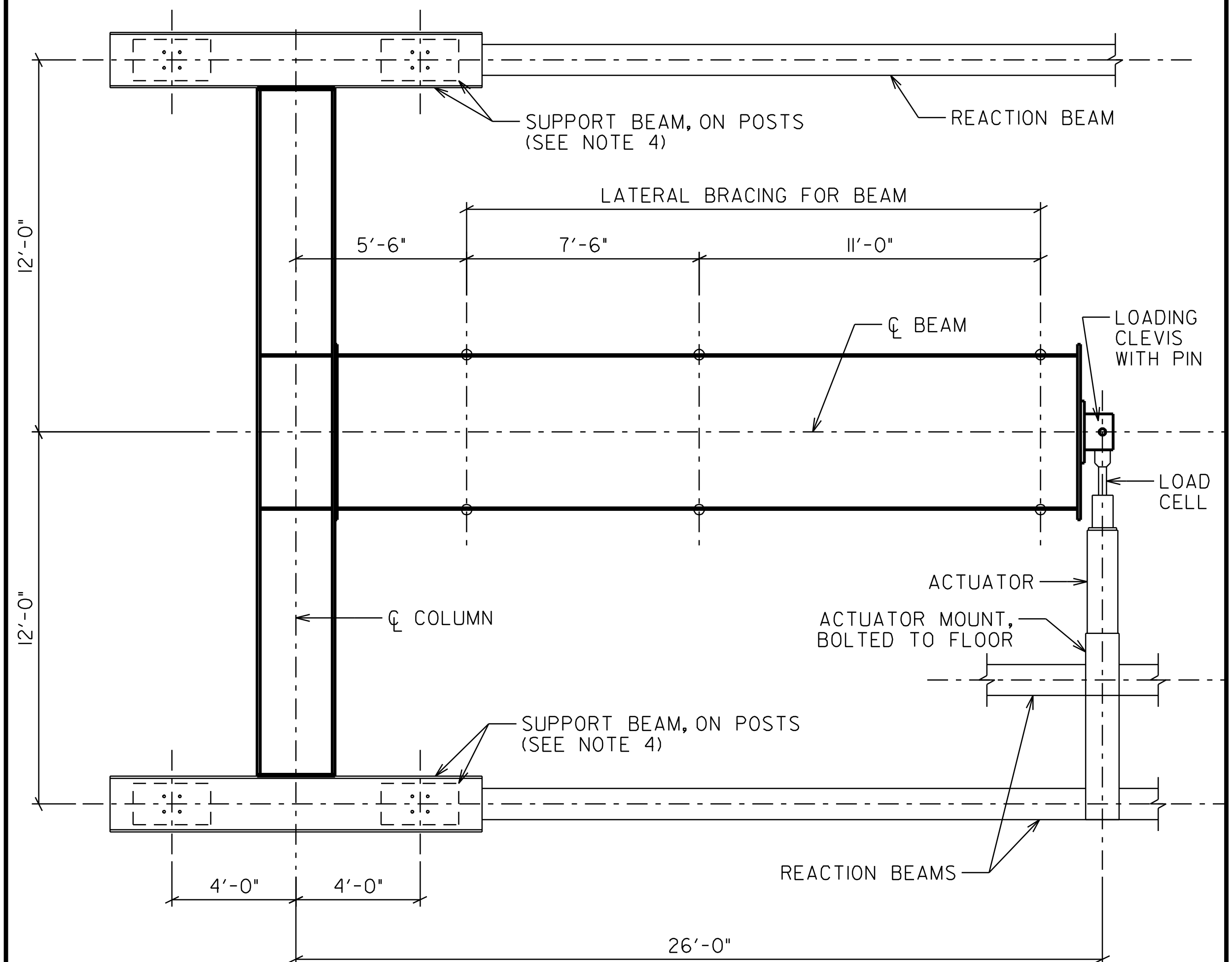
FAILURE MODE:	Bolt Tension Rupture	
MAXIMUM MOMENT AT COLUMN CENTERLINE, $M_{max,cc}$:	2866.1	ft-kips
MAXIMUM MOMENT AT FAYING SURFACE, $M_{max,fs}$:	2728.3	ft-kips
MAXIMUM INTERSTORY DRIFT ANGLE, θ_{max}:	0.021	rad *
MAXIMUM INTERSTORY DRIFT ANGLE AT 80% 0F $M_{max,cc}$, θ_{preq}:	0.016	rad

* Bolts ruptured on negative moment (near) side at 0.016 rad, then half-cycles performed on other side to 0.021 rad

CALCULATED STRENGTHS

BEAM MOMENT STRENGTH [1], M_b:	2661.0	ft-kips
BEAM PLASTIC STRENGTH [1], M_p:	3468.5	ft-kips
BEAM EXPECTED PLASTIC STRENGTH, M_{pe}:	3161.2	ft-kips
END-PLATE STRENGTH [1], M_{PL}:	2979.2	ft-kips
BOLT TENSION RUPTURE (w/o Prying, using F_t), M_{NP}:	2562.5	ft-kips

1. Measured material properties used.



MREI/2-1.0-1.0-60 B TEST SUBASSEMBLAGE PLAN

BEAM

WEB = $\frac{3}{8}$×58$\frac{3}{4}$
FLANGE = $\frac{5}{8}$×8
LENGTH = 288$\frac{1}{2}$ (INCLUDES ENDPLATES)

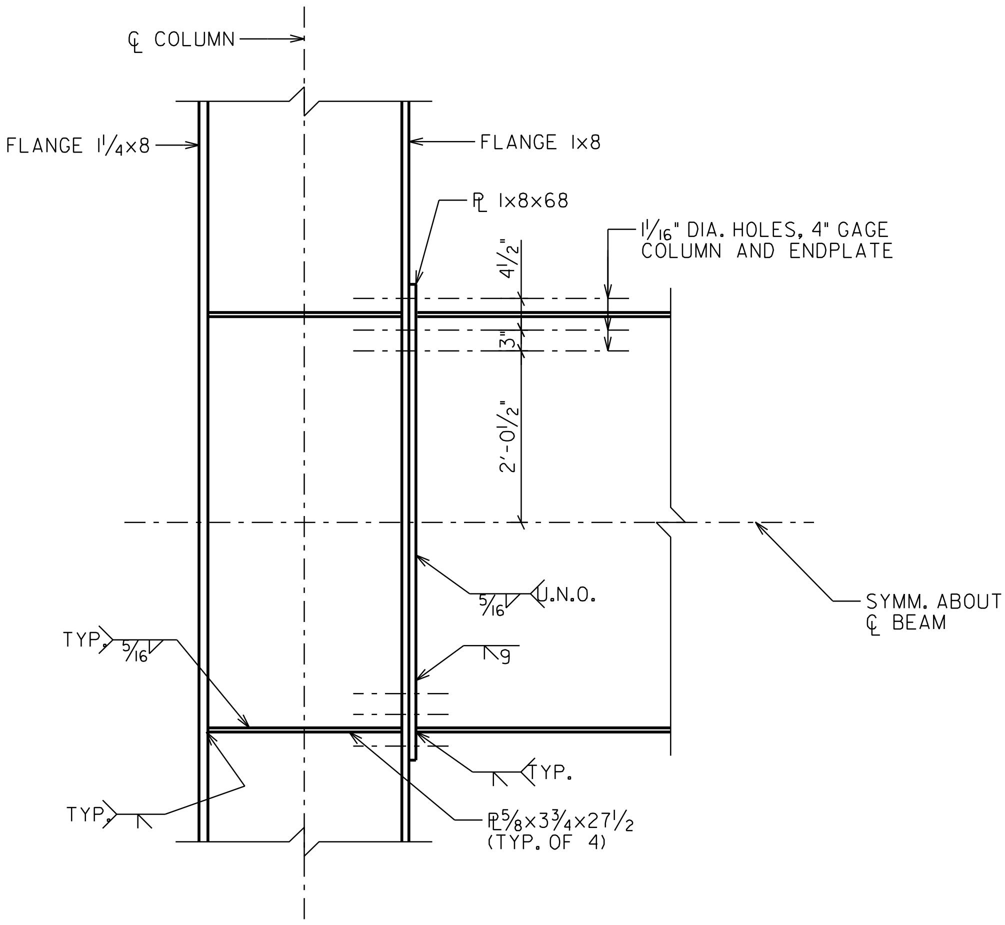
COLUMN

WEB = $\frac{1}{2}$×27$\frac{3}{4}$
FLANGES = 1$\frac{1}{4}$×8 & 1×8
LENGTH = 266$\frac{1}{2}$ (INCLUDES ENDPLATES)

NOTES

1. COLUMN, BEAM, AND CONNECTION STEEL, A572 Gr50.
2. ALL BOLTS, A490.
3. ALL WELDING, AWS D1.1, E70XX ELECTRODES.
4. COLUMN BOLTED TO SUPPORT BEAM. SUPPORT BEAM BOLTED POSTS. POSTS BOLTED TO REACTION BEAM.

SPECIMEN: MREI/2-1.0-1.0-60 B	DRAWN BY: SEB



MREI/2-1.0-1.0-60 B
CONNECTION DETAIL

SPECIMEN: MREI/2-1.0-1.0-60 B	DRAWN BY: SEB

SAC PROTOCOL LOADING HISTORY

Test Name: MRE1/2-1.0-1.0-60 B Test By: SEB Date: 4/26/2004

Load Step	Cycle	Max. Pos. Load (kips)	Max. Disp. (in)	Max. Neg. Load (kips)	Max. Disp. (in)	Comments
I (0.00375 rad)	1	31.2	1.156	-31.3	-1.170	
δ=1.163"	2	31.7	1.159	-31.5	-1.160	
	3	31.4	1.159	-31.8	-1.176	
	4	31.7	1.160	-31.4	-1.153	
	5	31.6	1.153	-31.8	-1.169	
	6	31.5	1.159	-31.4	-1.150	
		Permanent Set =	-0.087"			
II (0.005 rad)	1	41.2	1.552	-41.3	-1.550	
δ=1.55"	2	41.3	1.553	-41.8	-1.563	
	3	41.2	1.552	-41.5	-1.551	
	4	41.4	1.564	-41.6	-1.556	
	5	41.4	1.559	-41.4	-1.545	
	6	41.2	1.557	-41.5	-1.550	
		Permanent Set =	-0.095"			
III (0.0075 rad)	1	59.9	2.330	-61.9	-2.337	
δ=2.325"	2	60.2	2.339	-62.1	-2.329	
	3	60.1	2.280	-61.9	-2.313	
	4	60.0	2.331	-62.2	-2.326	
	5	60.0	2.329	-62.5	-2.341	
	6	59.8	2.325	-62.2	-2.326	
		Permanent Set =	-0.091"			
IV (0.01 rad)	1	78.4	3.097	-80.9	-3.108	
δ=3.1"	2	79.2	3.114	-80.5	-3.094	
	3	78.6	3.105	-80.8	-3.103	
	4	78.5	3.096	-80.7	-3.106	
		Permanent Set =	-0.154"			
V (0.015 rad)	1	100.8	4.643	-107.2	-4.654	Whitewash flaking on flanges
δ=4.65"	2	102.0	4.653	-107.2	-4.644	
		Permanent Set =	-0.457"			
VI (0.02 rad)	1	91.6	6.215	~ -110	~ -5	Bolt Tension Rupture
δ=6.2"	2	88.7	6.203			
		Permanent Set =	1.807"			
VII (0.03 rad)	1	~ 80	~ 6.4			Test stopped
δ=9.3"	2					
		Permanent Set =				
VIII (0.04 rad)	1					
	2					
		Permanent Set =				
IX (0.05 rad)	1					
	2					
		Permanent Set =				

CALCULATED STRENGTHS

TEST NAME: MRE1/2-1.0-1.0-60 B
TEST DATE: April 26, 2004

Specified Grade for all Steel = ASTM A572, Grade 50

MEASURED PROPERTIES

$d := 60 \cdot in$ $F_{yf} := 66.8 \cdot ksi$ $t_p := 1.04 \cdot in$ $p_{fo} := 2.00 \cdot in$ $p_b := 3.00 \cdot in$

$b_f := 8.00 \cdot in$ $F_{uf} := 90.5 \cdot ksi$ $b_p := 8.03 \cdot in$ $p_{fi} := 2.02 \cdot in$

$t_f := 0.63 \cdot in$ $F_{yw} := 66.0 \cdot ksi$ $L_p := 68.00 \cdot in$ $g := 4.04 \cdot in$

$t_w := 0.38 \cdot in$ $F_{uw} := 77.1 \cdot ksi$ $p_{ext} := 3.99 \cdot in$ $F_{yp} := 62.0 \cdot ksi$ $h := d - 2 \cdot t_f$

SPECIFIED PROPERTIES

$R_y := 1.1$ $P_t := 89 \cdot k$

BEAM PLASTIC STRENGTH, Mp

$$M_p := 2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right) \cdot F_{yf} + 2 \cdot t_w \cdot \frac{h}{2} \cdot \frac{h}{4} \cdot F_{yw} \qquad M_p = 3468.5 k \cdot ft$$

BEAM EXPECTED PLASTIC STRENGTH, Mpe

$$M_{pe} := 1.1 \cdot R_y \cdot \left[2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right) \cdot 50 \cdot ksi + 2 \cdot t_w \cdot \frac{h}{2} \cdot \frac{h}{4} \cdot 50 \cdot ksi\right] \qquad M_{pe} = 3161.2 k \cdot ft$$

BEAM MOMENT STRENGTH, Mb

$\frac{b_f}{2 \cdot t_f} = 6.3$ $\frac{65}{\sqrt{50}} = 9.2$ Flange is compact

$\frac{970}{\sqrt{50}} = 137.2$ $\frac{h}{t_w} = 154.6$ Web is slender, refer to Appendix G

Therefore, Mb < Mp

$$I_x := 2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right)^2 + \frac{1}{12} \cdot t_w \cdot h^3 \qquad I_x = 15301 in^4 \qquad S_x := \frac{I_x}{0.5 \cdot d} \qquad S_x = 510 in^3$$

$$a_r := \frac{h \cdot t_w}{b_f \cdot t_f} \qquad m := \frac{55}{55} \qquad R_e := \frac{12 + a_r \cdot \left(3 \cdot m - m^3\right)}{12 + 2 \cdot a_r} \qquad R_e = 1$$

BEAM MOMENT STRENGTH, Mb (CONT'D)

$I_T := \frac{1}{12} \cdot \left(\frac{0.5 \cdot d}{3}\right) \cdot t_w^3 + \frac{1}{12} \cdot t_f \cdot b_f^3$ $\quad I_T = 26.9 in^4$

$A_T := \left(\frac{0.5 \cdot d}{3}\right) \cdot t_w + t_f \cdot b_f$ $\quad A_T = 8.8 in^2$ $\quad r_T := \sqrt{\frac{I_T}{A_T}}$ $\quad r_T = 1.7 in$

Required L_b: $\quad L_b := 40.4 r_T$ $\quad L_b = 5.9 ft$ $\quad$ If the unbraced length is less than L_b: $\quad F_{cr} := F_{yf}$

$$R_{PG} := 1 - \left(\frac{a_r}{1200 + 300 \cdot a_r}\right) \cdot \left(\frac{h}{t_w} - 5.7 \cdot \sqrt{\frac{29000 ksi}{F_{cr}}}\right)$$

$M_b := S_x \cdot R_{PG} \cdot R_e \cdot F_{cr}$ $\quad M_b = 2661 k \cdot ft$

END-PLATE STRENGTH, Mpl

$d_0 := d - \frac{t_f}{2} + p_{fo}$ $\quad d_0 = 61.7 in$ $\quad h_0 := d_0 + \frac{t_f}{2}$ $\quad h_0 = 62 in$

$d_1 := d - \frac{t_f}{2} - t_f - p_{fi}$ $\quad d_1 = 57 in$ $\quad h_1 := d_1 + \frac{t_f}{2}$ $\quad h_1 = 57.4 in$

$d_2 := d_1 - p_b$ $\quad d_2 = 54 in$ $\quad h_2 := d_2 + \frac{t_f}{2}$ $\quad h_2 = 54.4 in$

$s := \frac{1}{2} \cdot \sqrt{b_p \cdot g}$ $\quad s = 2.85 in$

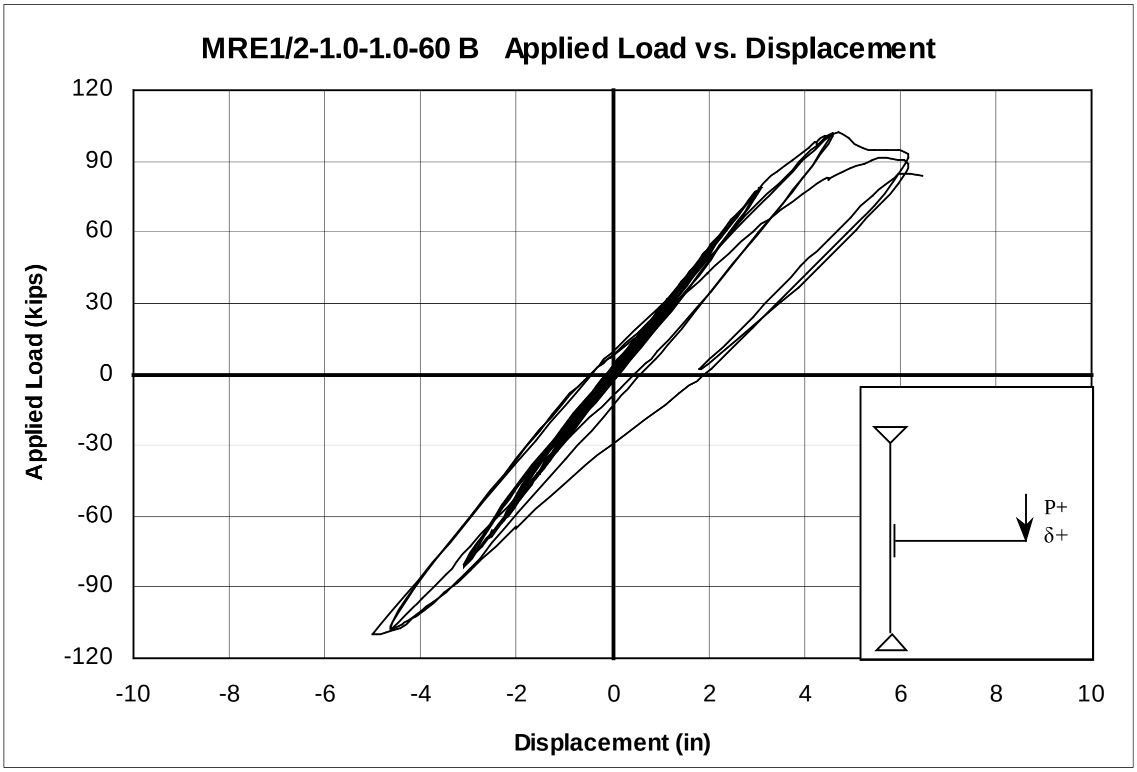
$$Y := \frac{b_p}{2} \cdot \left(\frac{h_1}{p_{fi}} + \frac{h_2}{s} + \frac{h_0}{p_{fo}} - \frac{1}{2}\right) + \frac{2}{g} \cdot \left[h_1 \cdot \left(p_{fi} + 0.75 \cdot p_b\right) + h_2 \cdot \left(s + 0.25 \cdot p_b\right)\right] + \frac{g}{2}$$

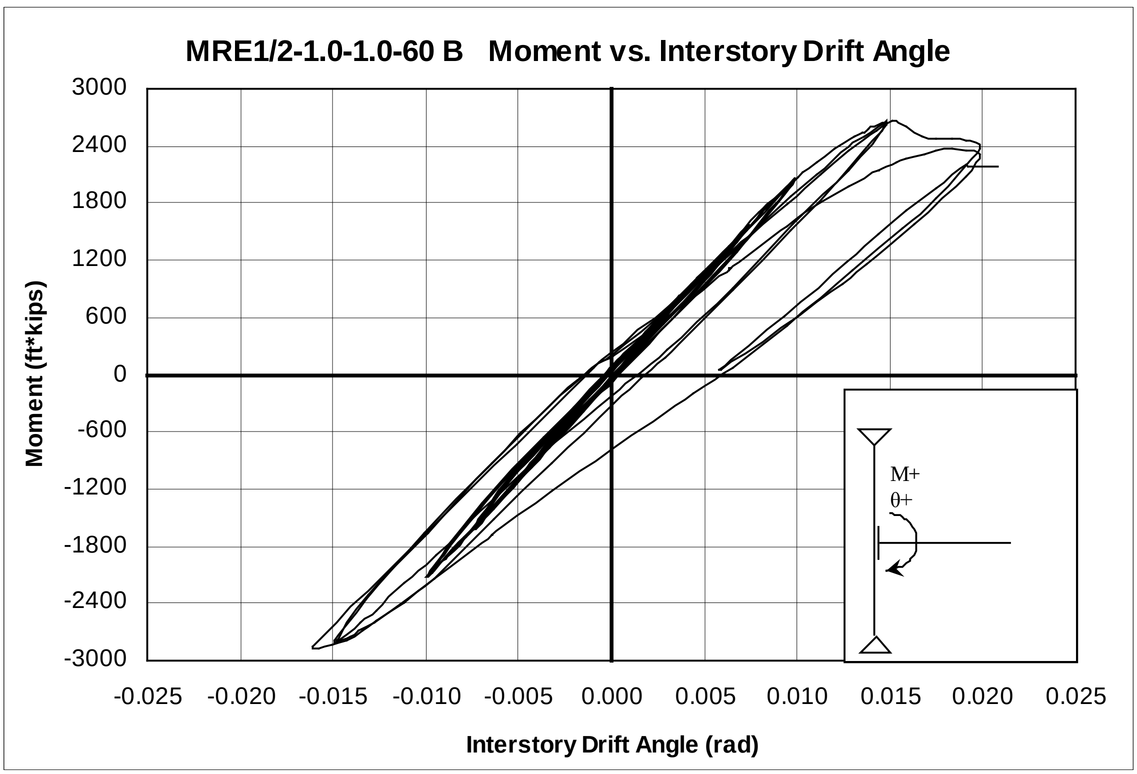
$Y = 533.1 in$

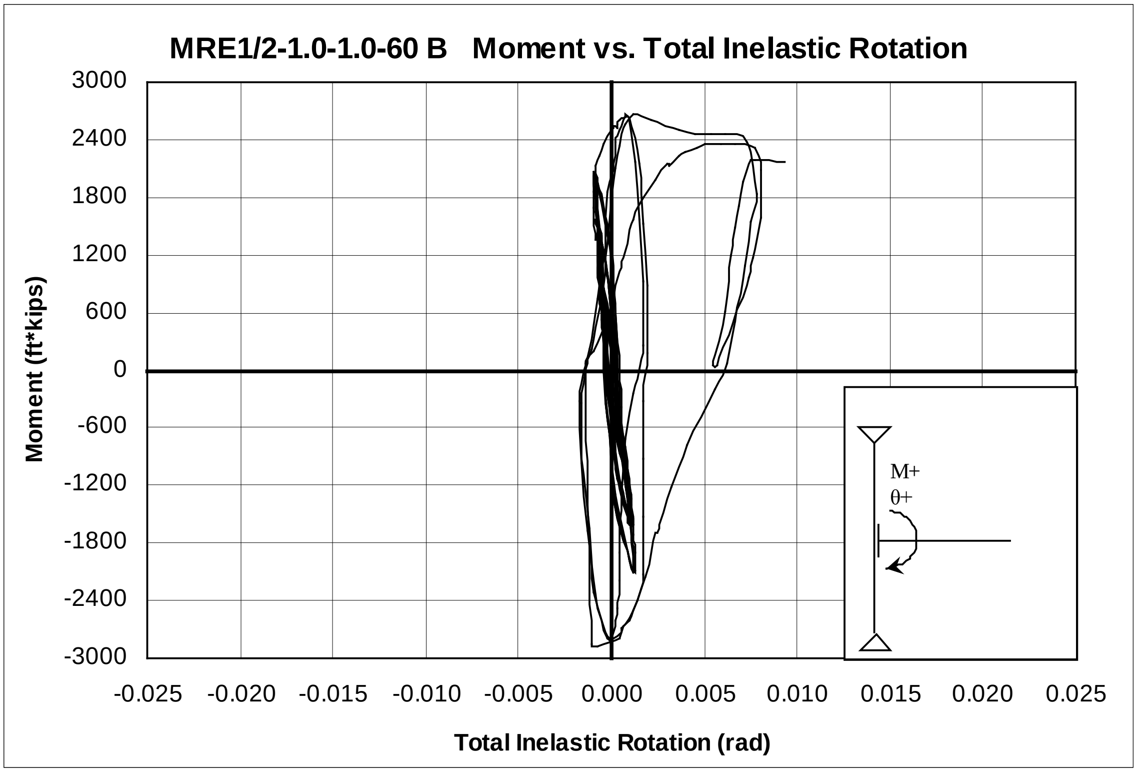
$M_{PL} := t_p^2 \cdot F_{yp} \cdot Y$ $\quad M_{PL} = 2979.2 k \cdot ft$

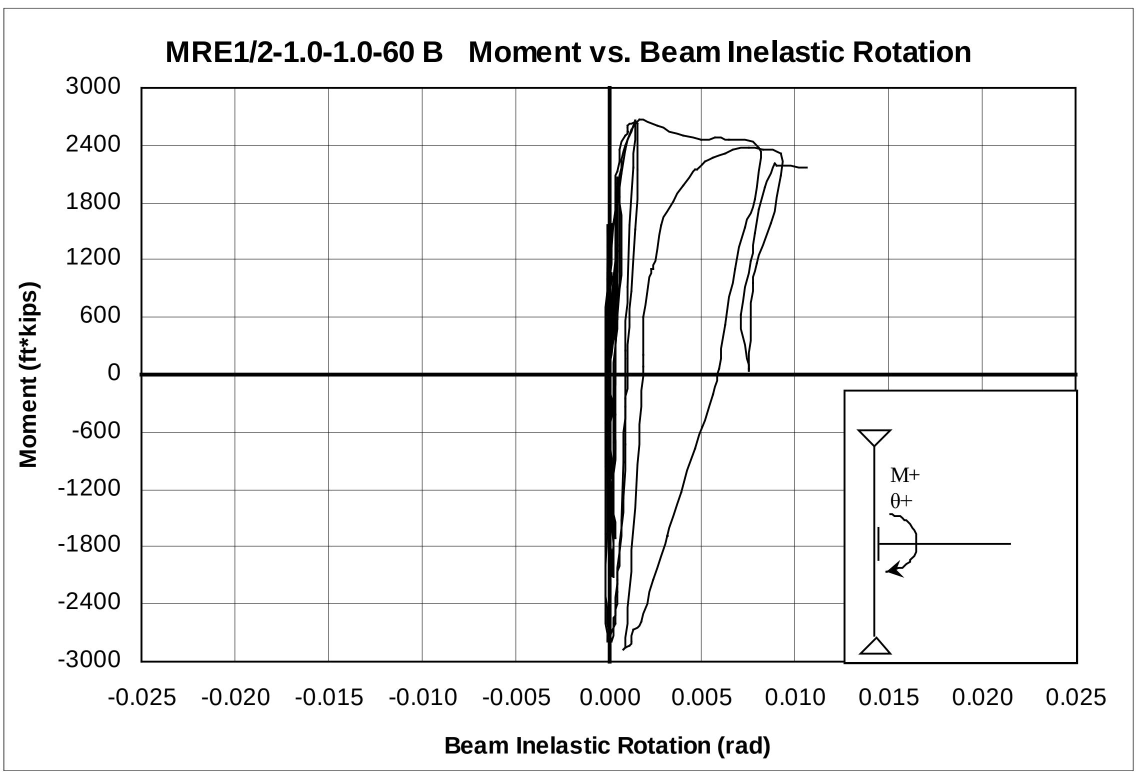
BOLT STRENGTH WITHOUT PRYING, Mnp

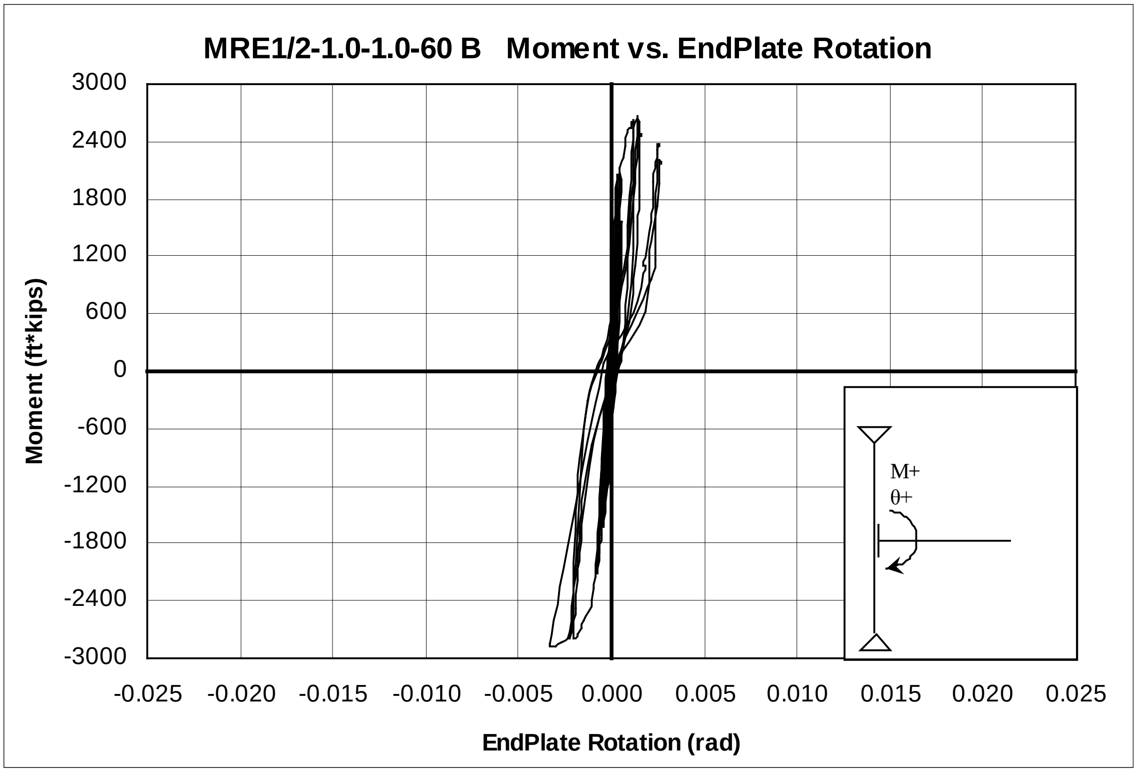
$M_{NP} := 2 \cdot P_t \cdot \left(d_0 + d_1 + d_2\right)$ $\quad M_{NP} = 2562.5 k \cdot ft$

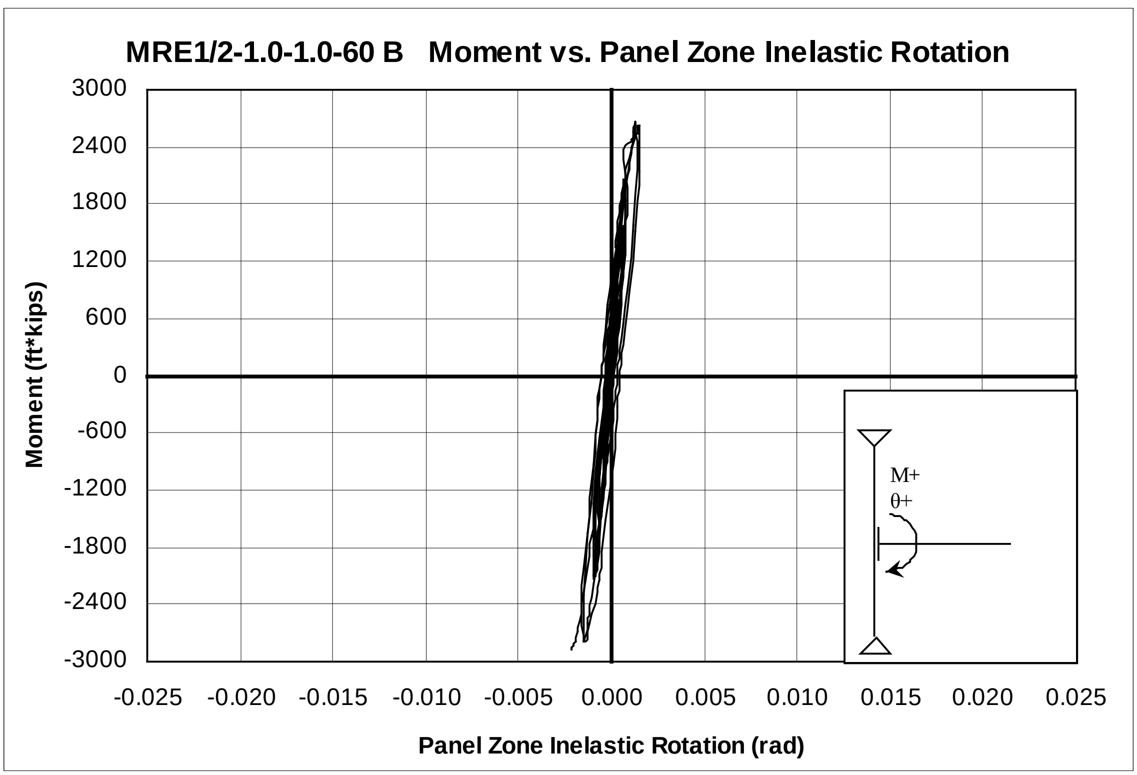

MRE1/2-1.0-1.0-60 B Applied Load vs. Displacement
Applied Load (kips)
120
90
60
30
0
-30
-60
-90
-120
-10
-8
-6
-4
-2
0
2
4
6
8
10
Displacement (in)
P+
δ+

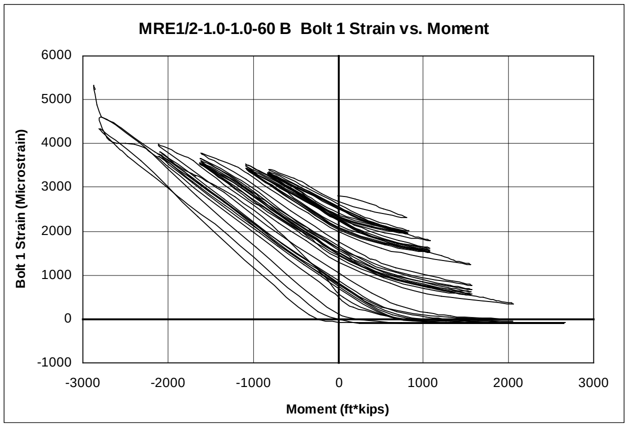

MRE1/2-1.0-1.0-60 B Moment vs. Interstory Drift Angle
Moment (ft*kips)
3000
2400
1800
1200
600
0
-600
-1200
-1800
-2400
-3000
-0.025
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.010
0.015
0.020
0.025
Interstory Drift Angle (rad)
M+
θ+

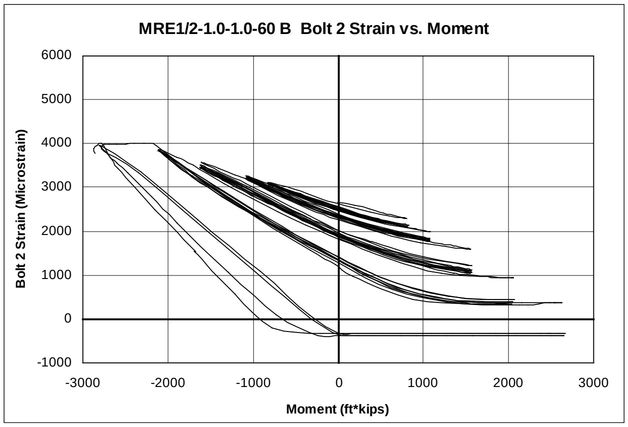

MRE1/2-1.0-1.0-60 B   Moment vs. Total Inelastic Rotation
Moment (ft*kips)
3000
2400
1800
1200
600
0
-600
-1200
-1800
-2400
-3000
-0.025
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.010
0.015
0.020
0.025
Total Inelastic Rotation (rad)
M+
θ+

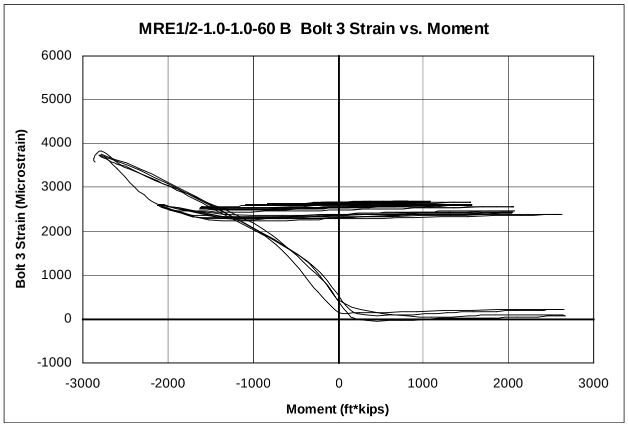

MRE1/2-1.0-1.0-60 B   Moment vs. Beam Inelastic Rotation
Moment (ft*kips)
3000
2400
1800
1200
600
0
-600
-1200
-1800
-2400
-3000
-0.025
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.010
0.015
0.020
0.025
Beam Inelastic Rotation (rad)
M+
θ+

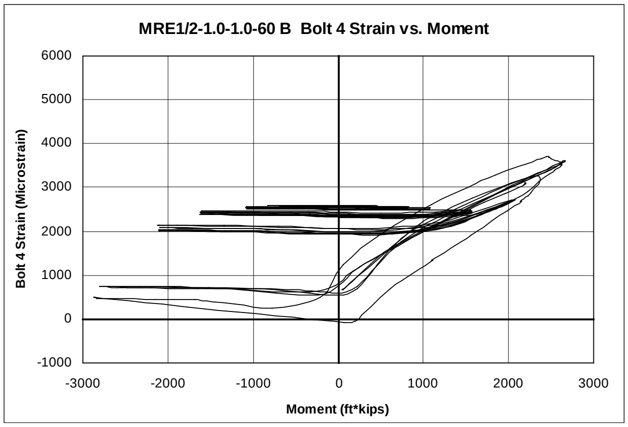

MRE1/2-1.0-1.0-60 B Moment vs. EndPlate Rotation
Moment (ft*kips)
3000
2400
1800
1200
600
0
-600
-1200
-1800
-2400
-3000
-0.025 -0.020 -0.015 -0.010 -0.005 0.000 0.005 0.010 0.015 0.020 0.025
EndPlate Rotation (rad)
M+
θ+

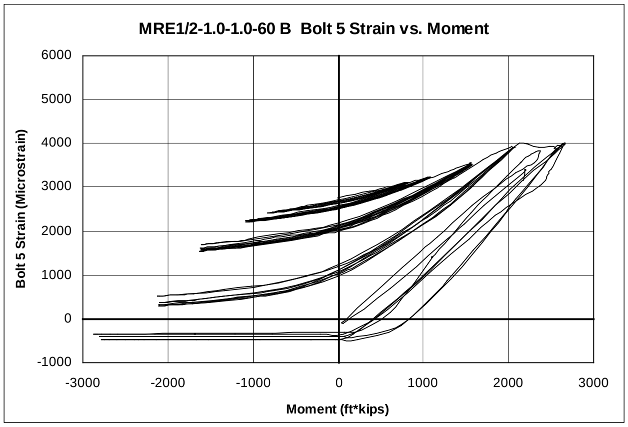

MRE1/2-1.0-1.0-60 B Moment vs. Panel Zone Inelastic Rotation
Moment (ft*kips)
3000
2400
1800
1200
600
0
-600
-1200
-1800
-2400
-3000
-0.025 -0.020 -0.015 -0.010 -0.005 0.000 0.005 0.010 0.015 0.020 0.025
Panel Zone Inelastic Rotation (rad)
M+
θ+

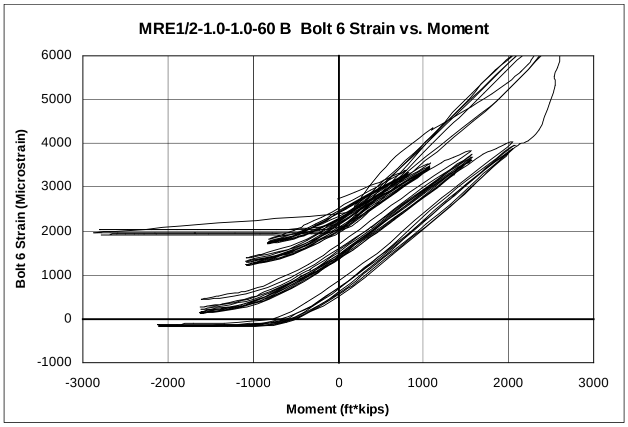

MRE1/2-1.0-1.0-60 B Bolt 1 Strain vs. Moment
Bolt 1 Strain (Microstrain)
Moment (ft*kips)
6000
5000
4000
3000
2000
1000
0
-1000
-3000
-2000
-1000
0
1000
2000
3000

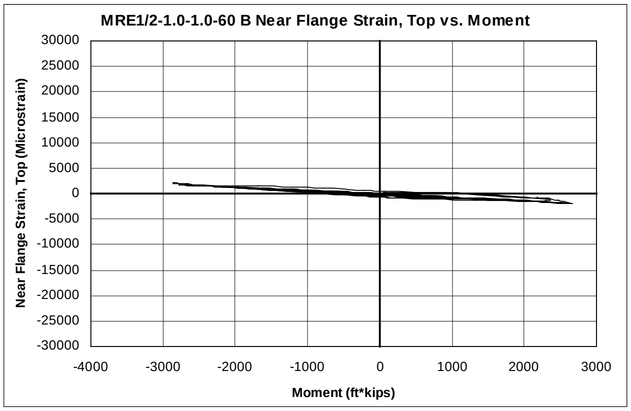

MRE1/2-1.0-1.0-60 B Bolt 2 Strain vs. Moment
Bolt 2 Strain (Microstrain)
Moment (ft*kips)
6000
5000
4000
3000
2000
1000
0
-1000
-3000
-2000
-1000
0
1000
2000
3000

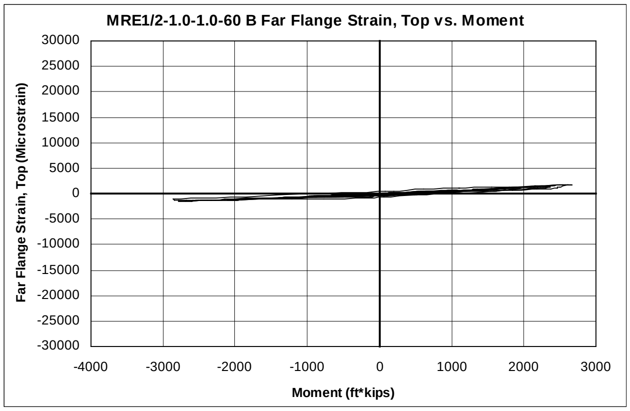

MRE1/2-1.0-1.0-60 B Bolt 3 Strain vs. Moment
Bolt 3 Strain (Microstrain)
Moment (ft*kips)
6000
5000
4000
3000
2000
1000
0
-1000
-3000
-2000
-1000
0
1000
2000
3000

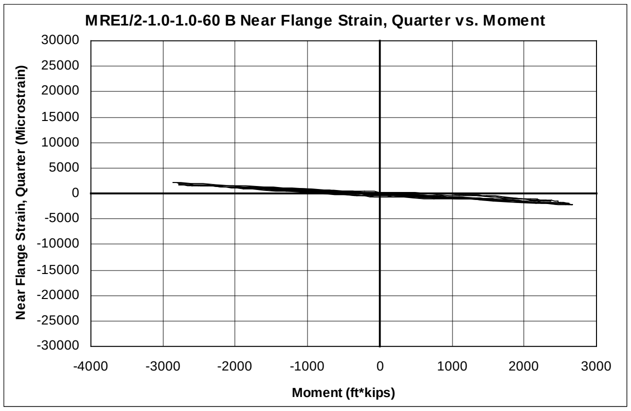

MRE1/2-1.0-1.0-60 B Bolt 4 Strain vs. Moment
Bolt 4 Strain (Microstrain)
Moment (ft*kips)
6000
5000
4000
3000
2000
1000
0
-1000
-3000
-2000
-1000
0
1000
2000
3000

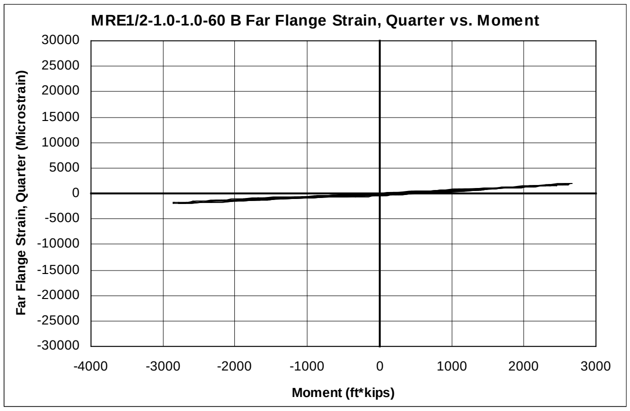

MRE1/2-1.0-1.0-60 B Bolt 5 Strain vs. Moment
Bolt 5 Strain (Microstrain)
Moment (ft*kips)
6000
5000
4000
3000
2000
1000
0
-1000
-3000
-2000
-1000
0
1000
2000
3000

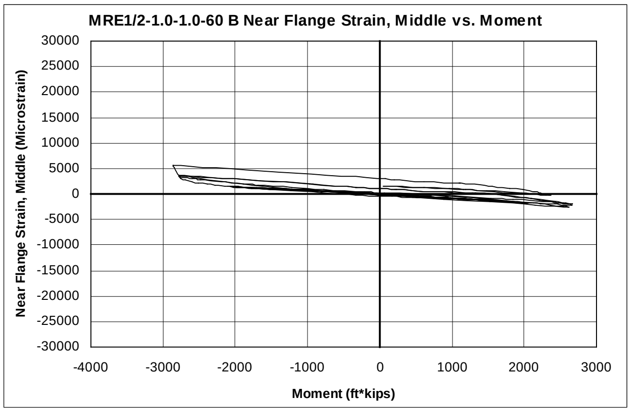

MRE1/2-1.0-1.0-60 B Bolt 6 Strain vs. Moment
Bolt 6 Strain (Microstrain)
Moment (ft*kips)
6000
5000
4000
3000
2000
1000
0
-1000
-3000
-2000
-1000
0
1000
2000
3000

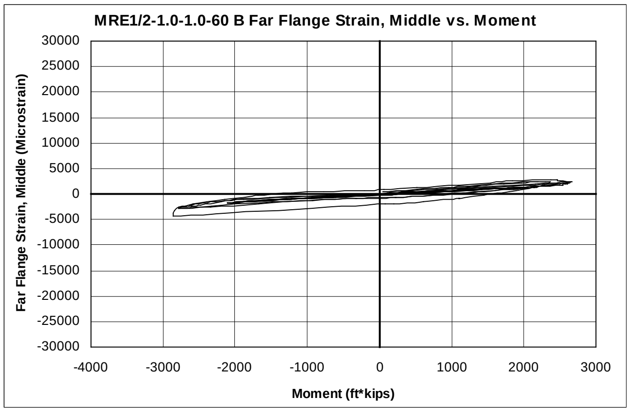

MRE1/2-1.0-1.0-60 B Near Flange Strain, Top vs. Moment
Near Flange Strain, Top (Microstrain)
30000
25000
20000
15000
10000
5000
0
-5000
-10000
-15000
-20000
-25000
-30000
-4000
-3000
-2000
-1000
0
1000
2000
3000
Moment (ft*kips)


MRE1/2-1.0-1.0-60 B Far Flange Strain, Top vs. Moment
Far Flange Strain, Top (Microstrain)
30000
25000
20000
15000
10000
5000
0
-5000
-10000
-15000
-20000
-25000
-30000
-4000
-3000
-2000
-1000
0
1000
2000
3000
Moment (ft*kips)

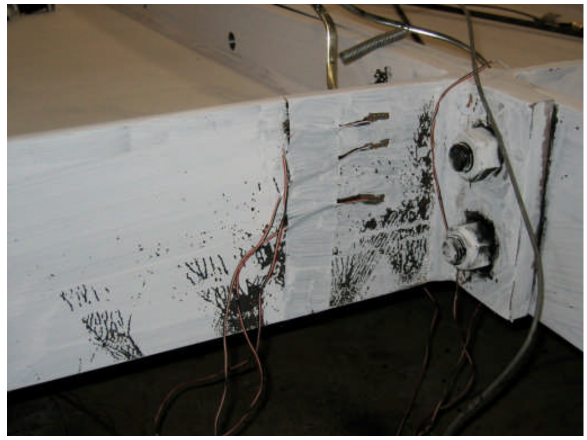

MRE1/2-1.0-1.0-60 B Near Flange Strain, Quarter vs. Moment
Near Flange Strain, Quarter (Microstrain)
30000
25000
20000
15000
10000
5000
0
-5000
-10000
-15000
-20000
-25000
-30000
-4000
-3000
-2000
-1000
0
1000
2000
3000
Moment (ft*kips)


MRE1/2-1.0-1.0-60 B Far Flange Strain, Quarter vs. Moment
Far Flange Strain, Quarter (Microstrain)
30000
25000
20000
15000
10000
5000
0
-5000
-10000
-15000
-20000
-25000
-30000
-4000
-3000
-2000
-1000
0
1000
2000
3000
Moment (ft*kips)

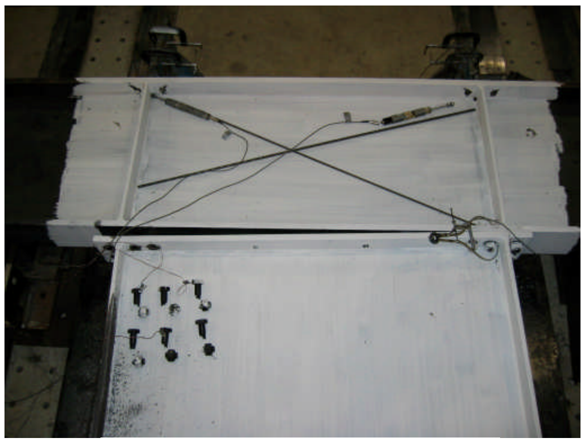

MRE1/2-1.0-1.0-60 B Near Flange Strain, Middle vs. Moment
Near Flange Strain, Middle (Microstrain)
30000
25000
20000
15000
10000
5000
0
-5000
-10000
-15000
-20000
-25000
-30000
-4000
-3000
-2000
-1000
0
1000
2000
3000
Moment (ft*kips)


MRE1/2-1.0-1.0-60 B Far Flange Strain, Middle vs. Moment
Far Flange Strain, Middle (Microstrain)
30000
25000
20000
15000
10000
5000
0
-5000
-10000
-15000
-20000
-25000
-30000
-4000
-3000
-2000
-1000
0
1000
2000
3000
Moment (ft*kips)

Near Flange



Far Flange



Top View

MRE1/2-1.0-1.0-60 B AFTER TESTING

APPENDIX J

MRE1/3S – 1.25 – 1.25 – 72 A

RESULTS AND TEST DATA

SPECIMEN PROPERTIES & TEST SUMMARY

TEST NAME: MRE1/3S-1.25-1.25-72 A
TEST DATE: March 12, 2004

BEAM DATA

DEPTH, d:	72.40	in
FLANGE WIDTH, b_f:	11.90	in
FLANGE THICKNESS, t_f:	1.00	in
WEB THICKNESS, t_w:	0.50	in
FLANGE YIELD STRESS, F_{yf}:	55.6	ksi
FLANGE ULTIMATE STRESS, F_{uf}:	78.8	ksi
WEB YIELD STRESS, F_{yw}:	51.8	ksi
WEB ULTIMATE STRESS, F_{uw}:	74.1	ksi

END-PLATE DATA

END-PLATE THICKNESS, t_p:	1.27	in
END-PLATE WIDTH, b_p:	11.95	in
END-PLATE LENGTH, L_p:	80.42	in
END-PLATE EXTENSION OUTSIDE FLANGE, p_{ext}:	4.01	in
OUTER PITCH, BOLT TO FLANGE, p_{fo}:	2.00	in
INNER PITCH, BOLT TO FLANGE, p_{fi}:	1.99	in
PITCH, BOLT TO BOLT, p_b:	3.00	in
GAGE, g:	6.05	in
END-PLATE YIELD STRESS, F_{yp}:	53.9	ksi
END-PLATE ULTIMATE STRESS, F_{up}:	78.3	ksi

BOLT DATA

BOLT DIAMETER, d_b:	1.25	in
BOLT LENGTH, L_b:	5.00	in
BOLT TYPE:	A490	
NOMINAL BOLT TENSILE STRESS (AISC J3.6), F_t:	113	ksi
NOMINAL BOLT TENSILE STRENGTH, P_t:	139	kips
BOLT PRETENSION, T_b:	102	kips

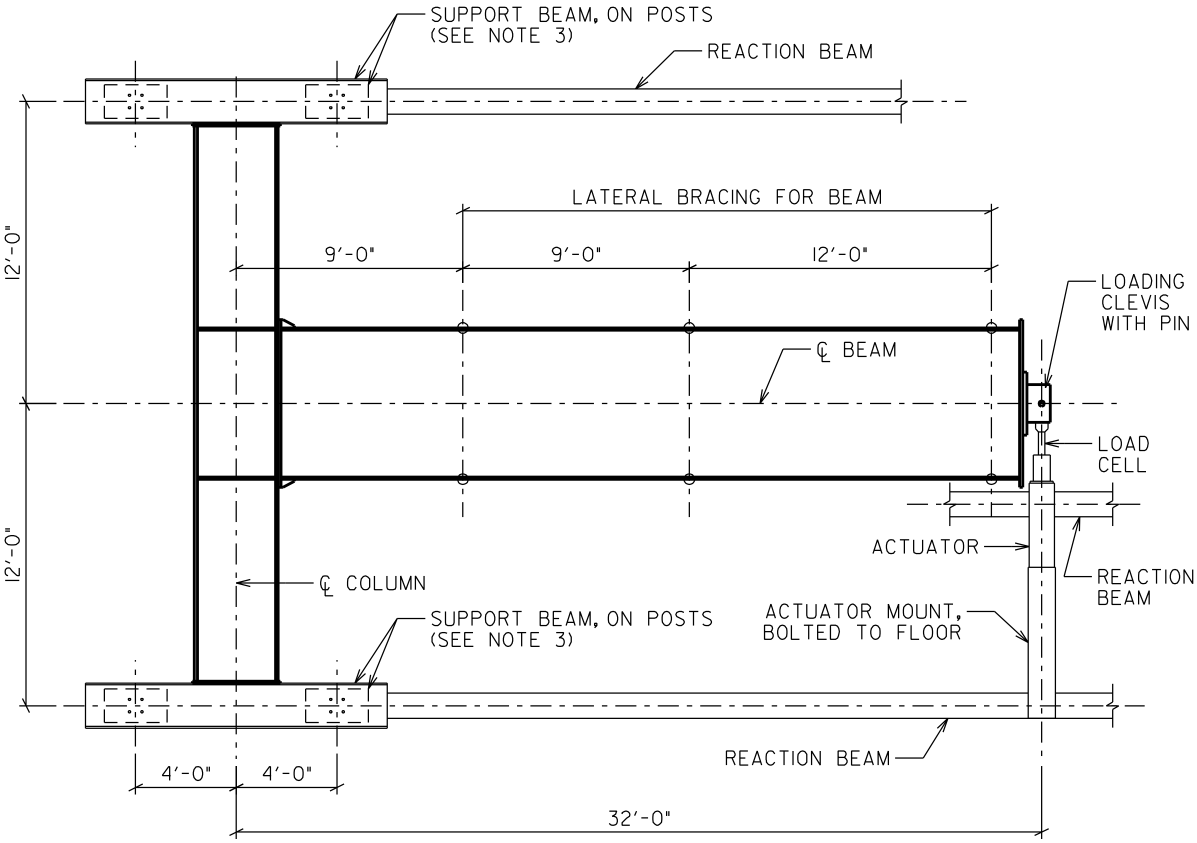
EXPERIMENTAL RESULTS

FAILURE MODE:	Bolt Tension Rupture	
MAXIMUM MOMENT AT COLUMN CENTERLINE, $M_{max,cc}$:	7553.5	ft-kips
MAXIMUM MOMENT AT FAYING SURFACE, $M_{max,fs}$:	7160.1	ft-kips
MAXIMUM INTERSTORY DRIFT ANGLE, θ_{max}:	0.014	rad
MAXIMUM INTERSTORY DRIFT ANGLE AT 80% 0F $M_{max,cc}$, θ_{preq}:	0.014	rad

CALCULATED STRENGTHS

BEAM MOMENT STRENGTH [1], M_b:	5656.7	ft-kips
BEAM PLASTIC STRENGTH [1], M_p:	6611.0	ft-kips
BEAM EXPECTED PLASTIC STRENGTH, M_{pe}:	7835.5	ft-kips
END-PLATE STRENGTH [1], M_{PL}:	6833.1	ft-kips
BOLT TENSION RUPTURE (w/o Prying, using F_t), M_{NP}:	6292.8	ft-kips

1. Measured material properties used.



MREI/3S-1.25-1.25-72 A TEST SUBASSEMBLAGE PLAN

BEAM

WEB = 1/2x70, A572 Grade 50
FLANGE = 1x12, A572 Grade 55
LENGTH = 355 (INCLUDES ENDPLATES)

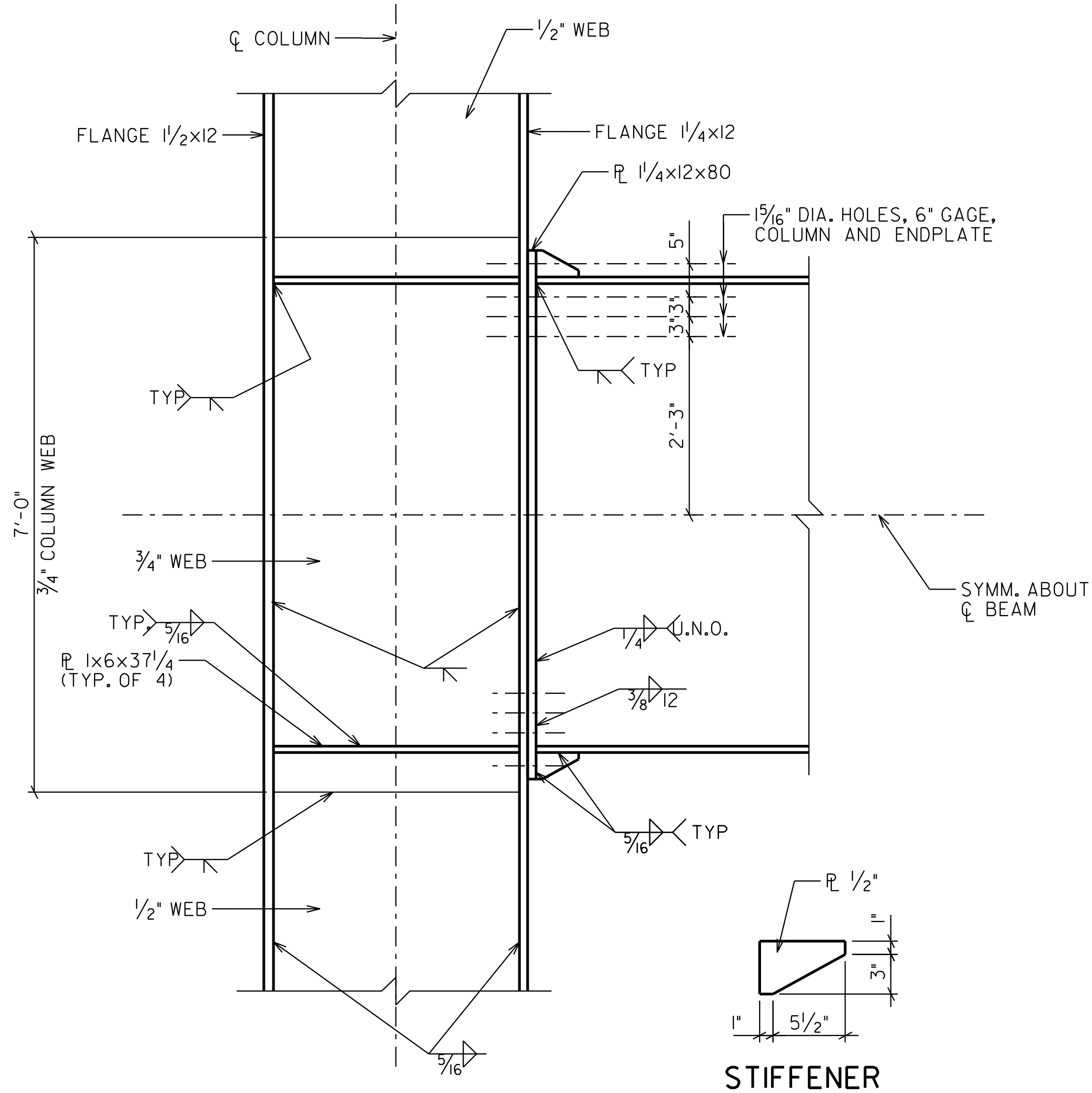
COLUMN

WEB = 1/2x37 1/4, A572 Grade 50
PANEL ZONE WEB = 3/4x37 1/4, A572 Grade 50
FLANGES = 1 1/4x12 & 1 1/2x12, A572 Grade 55
LENGTH = 266 1/2 (INCLUDES ENDPLATES)

NOTES

1. ALL BOLTS, A490.
2. ALL WELDING, AWS D1.1, E70XX ELECTRODES.
3. COLUMN BOLTED TO SUPPORT BEAM. SUPPORT BEAM BOLTED TO POSTS. POSTS BOLTED TO REACTION BEAM.

SPECIMEN: MREI/3S-1.25-1.25-72 A	DRAWN BY: SEB



MREI/3S-1.25-1.25-72 A
CONNECTION DETAIL

SPECIMEN: MREI/3S-1.25-1.25-72 A | DRAWN BY: SEB

SAC PROTOCOL LOADING HISTORY

Test Name: MRE1/3S-1.25-1.25-72 A   Test By: SEB   Date: 3/12/2004

Load Step	Cycle	Max. Pos. Load (kips)	Max. Disp. (in)	Max. Neg. Load (kips)	Max. Disp. (in)	Comments
I (0.00375 rad)	1	66.9	1.440	-65.8	-1.440	
δ=1.44"	2	68.4	1.438	-65.6	-1.436	
	3	69.6	1.444	-63.8	-1.442	
	4	68.9	1.447	-64.1	-1.436	
	5	69.8	1.451	-63.6	-1.436	
	6	69.9	1.451	-63.4	-1.441	
		Permanent Set =	-0.110"			
II (0.005 rad)	1	88.9	1.924	-84.5	-1.920	
δ=1.92"	2	88.3	1.920	-82.3	-1.932	
	3	85.6	1.922	-81.3	-1.929	
	4	87.4	1.924	-81.3	-1.922	
	5	87.5	1.922	-81.3	-1.929	
	6	88.0	1.942	-81.3	-1.921	
		Permanent Set =	-0.189"			
III (0.0075 rad)	1	138.8	2.872	-130.4	-2.880	
δ=2.88"	2	139.0	2.878	-130.1	-2.882	
	3	138.8	2.874	-129.6	-2.883	
	4	138.9	2.889	-130.6	-2.887	
	5	138.0	2.886	-129.1	-2.879	
	6	138.6	2.880	-129.6	-2.890	
		Permanent Set =	-0.205"			
IV (0.01 rad)	1	167.2	3.810	-166.3	-3.833	Whitewash flaking, beam flange
δ=3.84"	2	163.3	3.812	-172.8	-3.842	
	3	159.9	3.837	-172.0	-3.826	
	4	157.4	3.843	-173.0	-3.819	
		Permanent Set =	0.098"			
V (0.015 rad)	1	~ 175	~ 5	-236.0	-5.764	Single bolt, tension rupture
δ=5.76"	2					
		Permanent Set =				
VI (0.02 rad)	1					
	2					
		Permanent Set =				
VII (0.03 rad)	1					
	2					
		Permanent Set =				
VIII (0.04 rad)	1					
	2					
		Permanent Set =				
IX (0.05 rad)	1					
	2					
		Permanent Set =				

CALCULATED STRENGTHS

TEST NAME: MRE1/3S-1.25-1.25-72 A
TEST DATE: March 12, 2004

Specified Grade for Beam Web Steel = ASTM A572, Grade 50
Specified Grade for all other Steel = ASTM A572, Grade 55

MEASURED PROPERTIES

$d := 72.4 \cdot in$ $\quad F_{yf} := 55.6 \cdot ksi$ $\quad t_p := 1.27 \cdot in$ $\quad p_{fo} := 2.00 \cdot in$ $\quad p_b := 3.00 \cdot in$

$b_f := 11.9 \cdot in$ $\quad F_{uf} := 78.8 \cdot ksi$ $\quad b_p := 11.95 \cdot in$ $\quad p_{fi} := 1.99 \cdot in$

$t_f := 1.00 \cdot in$ $\quad F_{yw} := 51.8 \cdot ksi$ $\quad L_p := 80.42 \cdot in$ $\quad g := 6.05 \cdot in$

$t_w := 0.50 \cdot in$ $\quad F_{uw} := 74.1 \cdot ksi$ $\quad p_{ext} := 4.01 \cdot in$ $\quad F_{yp} := 53.9 \cdot ksi$ $\quad h := d - 2 \cdot t_f$

SPECIFIED PROPERTIES

$R_y := 1.1$ $\quad P_t := 139 \cdot k$

BEAM PLASTIC STRENGTH, Mp

$$M_p := 2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right) \cdot F_{yf} + 2 \cdot t_w \cdot \frac{h}{2} \cdot \frac{h}{4} \cdot F_{yw} \qquad M_p = 6611 k \cdot ft$$

BEAM EXPECTED PLASTIC STRENGTH, Mpe

$$M_{pe} := 1.1 \cdot R_y \cdot \left[2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right) \cdot 55 \cdot ksi + \left(2 \cdot t_w \cdot \frac{h}{2}\right) \cdot \frac{h}{4} \cdot 50 \cdot ksi\right] \qquad M_{pe} = 7835.5 k \cdot ft$$

BEAM MOMENT STRENGTH, Mb

$\frac{b_f}{2 \cdot t_f} = 6$ $\qquad \frac{65}{\sqrt{55}} = 8.8$ $\qquad$ Flange is compact

$\frac{970}{\sqrt{50}} = 137.2$ $\qquad \frac{h}{t_w} = 140.8$ $\qquad$ Web is slender, refer to Appendix G

Therefore, Mb < Mp

$$I_x := 2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right)^2 + \frac{1}{12} \cdot t_w \cdot h^3 \qquad I_x = 44871 in^4 \qquad S_x := \frac{I_x}{0.5 \cdot d} \qquad S_x = 1239.5 in^3$$

$$a_r := \frac{h \cdot t_w}{b_f \cdot t_f} \qquad m := \frac{55}{55} \qquad R_e := \frac{12 + a_r \cdot \left(3 \cdot m - m^3\right)}{12 + 2 \cdot a_r} \qquad R_e = 1$$

BEAM MOMENT STRENGTH, Mb (CONT'D)

$I_T := \frac{1}{12}\cdot\left(\frac{0.5\cdot d}{3}\right)\cdot t_w^3 + \frac{1}{12}\cdot t_f\cdot b_f^3 \qquad I_T = 140.6in^4$

$A_T := \left(\frac{0.5\cdot d}{3}\right)\cdot t_w + t_f\cdot b_f \qquad A_T = 17.9in^2 \qquad r_T := \sqrt{\frac{I_T}{A_T}} \qquad r_T = 2.8in$

Required L_b: $L_b := 40.4\cdot r_T \qquad L_b = 9.4ft$ If the unbraced length is less than L_b: $F_{cr} := F_{yf}$

$$R_{PG} := 1 - \left(\frac{a_r}{1200 + 300\cdot a_r}\right)\cdot\left(\frac{h}{t_w} - 5.7\cdot\sqrt{\frac{29000ksi}{F_{cr}}}\right)$$

$M_b := S_x\cdot R_{PG}\cdot R_e\cdot F_{cr} \qquad M_b = 5656.7k\cdot ft$

END-PLATE STRENGTH, Mpl

$d_0 := d - \frac{t_f}{2} + p_{fo} \qquad d_0 = 73.9in \qquad h_0 := d_0 + \frac{t_f}{2} \qquad h_0 = 74.4in$

$d_1 := d - \frac{t_f}{2} - t_f - p_{fi} \qquad d_1 = 68.9in \qquad h_1 := d_1 + \frac{t_f}{2} \qquad h_1 = 69.4in$

$d_2 := d_1 - p_b \qquad d_2 = 65.9in \qquad h_2 := d_2 + \frac{t_f}{2} \qquad h_2 = 66.4in$

$d_3 := d_2 - p_b \qquad d_3 = 62.9in \qquad h_3 := d_3 + \frac{t_f}{2} \qquad h_3 = 63.4in$

$s := \frac{1}{2}\cdot\sqrt{b_p\cdot g} \qquad s = 4.25in \qquad d_e := p_{ext} - p_{fo} \qquad d_e = 2in$

$d_e < s$, therefore Case 2 governs

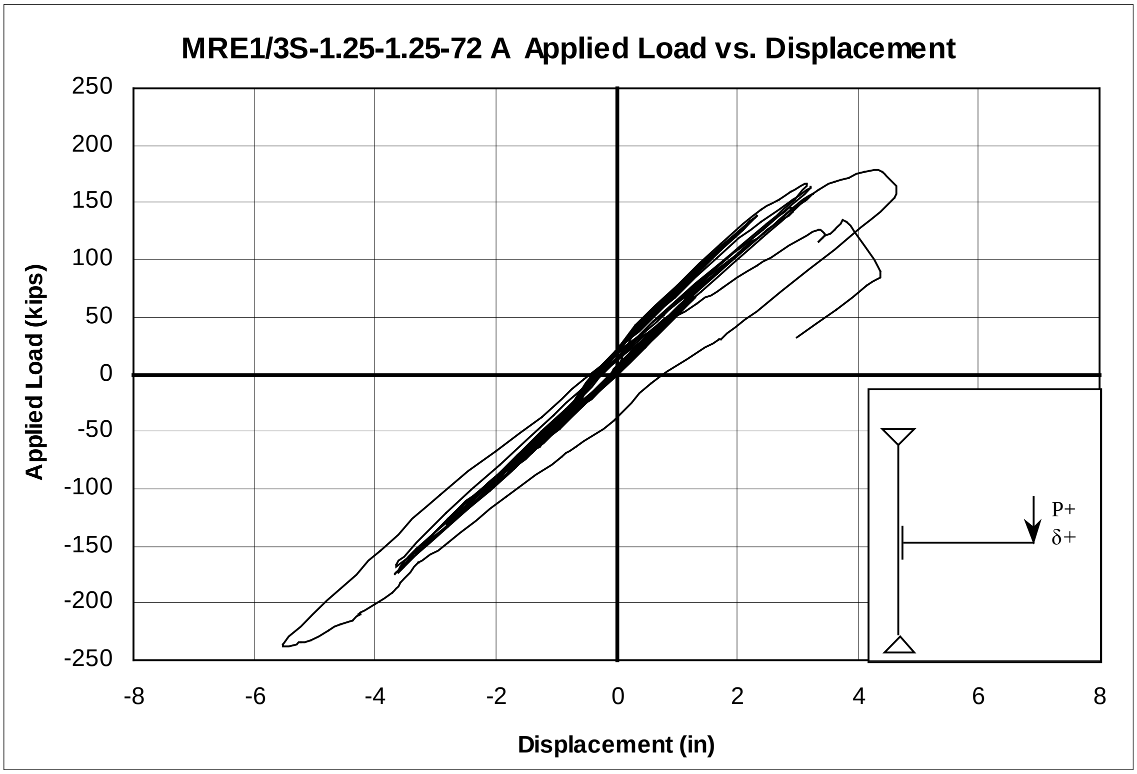
$$Y := \frac{b_p}{2}\cdot\left[\frac{h_1}{p_{fi}} + \frac{h_3}{s} + h_0\cdot\left(\frac{1}{p_{fo}} + \frac{1}{2\cdot s}\right)\right] + \frac{2}{g}\cdot\left[h_1\cdot\left(p_{fi} + 1.5\cdot p_b\right) + h_3\cdot\left(s + 0.5\cdot p_b\right) + h_0\cdot\left(d_e + p_{fo}\right)\right] + \frac{g}{2}$$

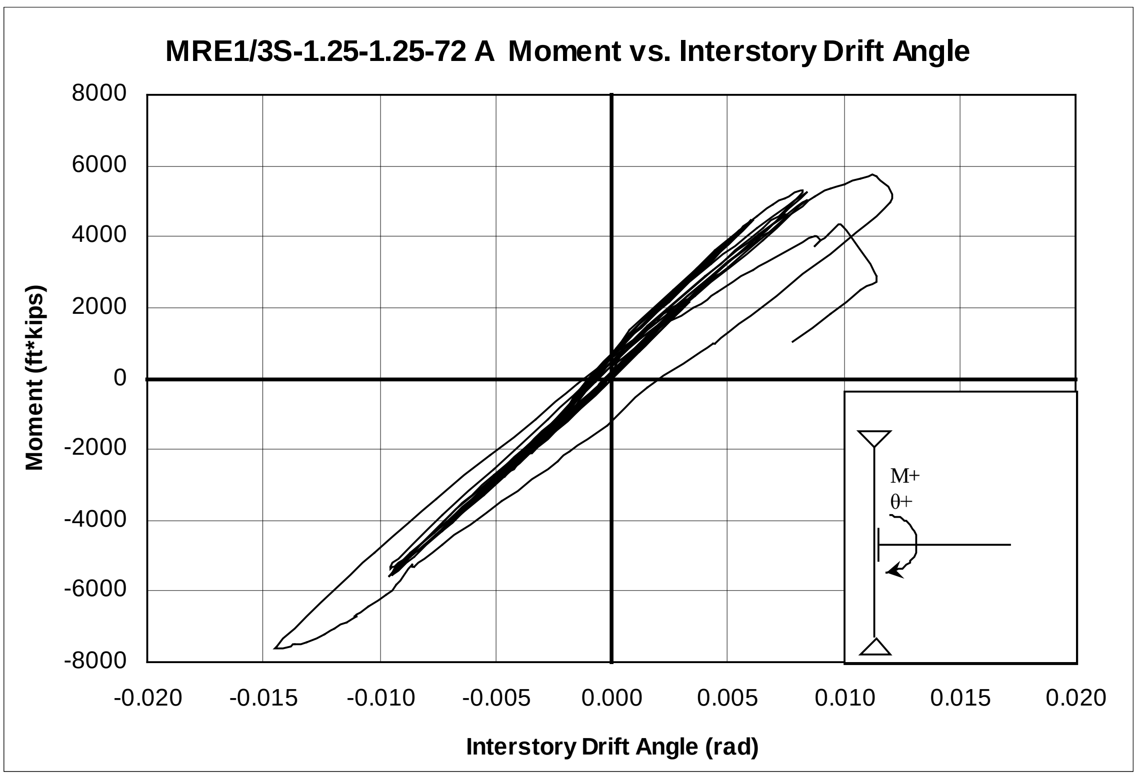
$Y = 943.2in$

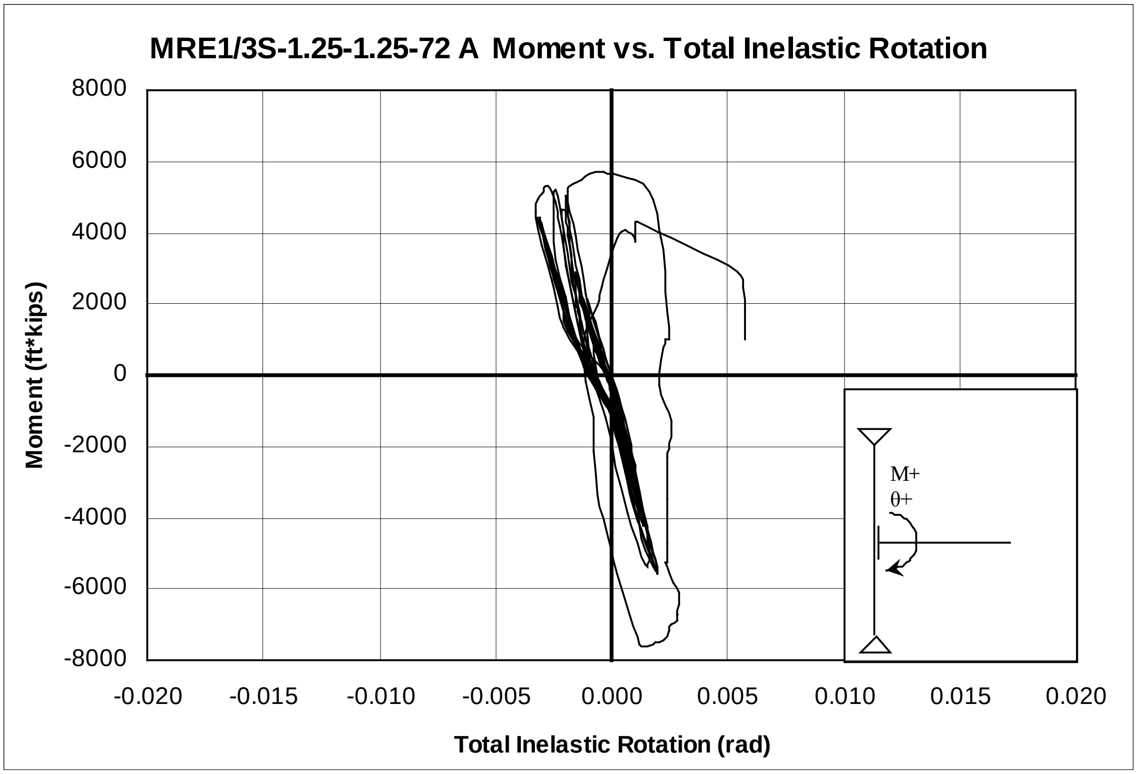
$M_{PL} := t_p^2\cdot F_{yp}\cdot Y \qquad M_{PL} = 6833.1k\cdot ft$

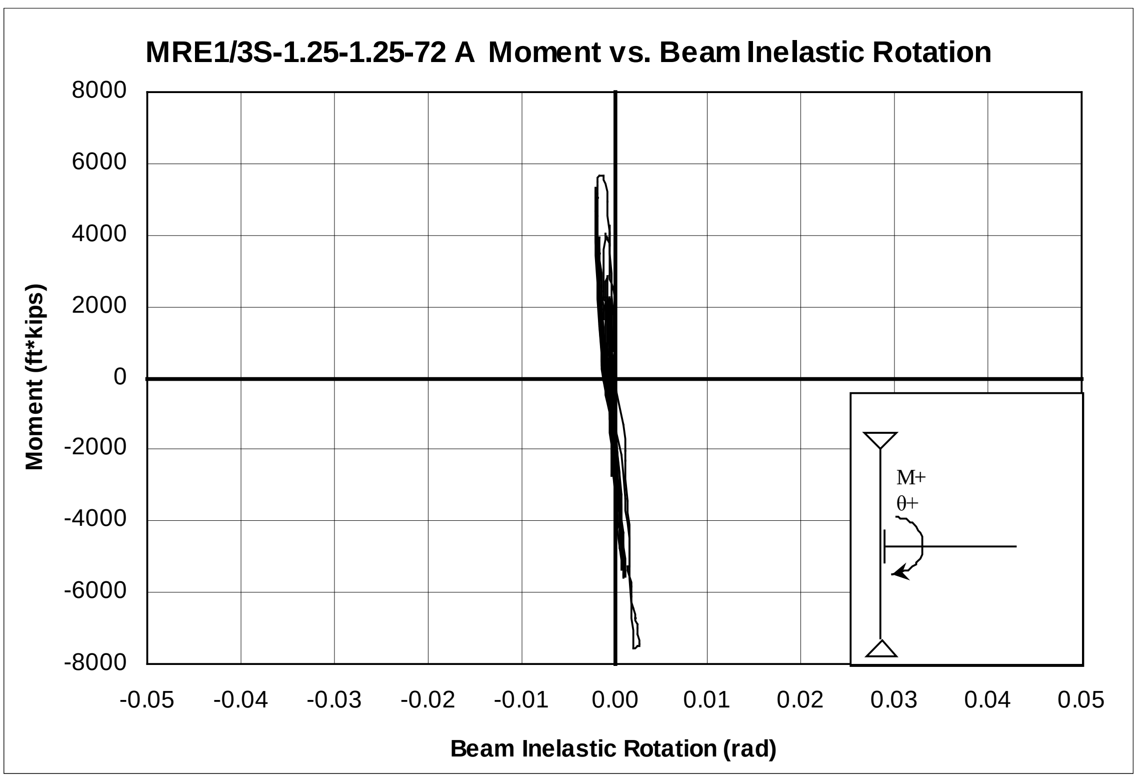
BOLT STRENGTH WITHOUT PRYING, Mnp

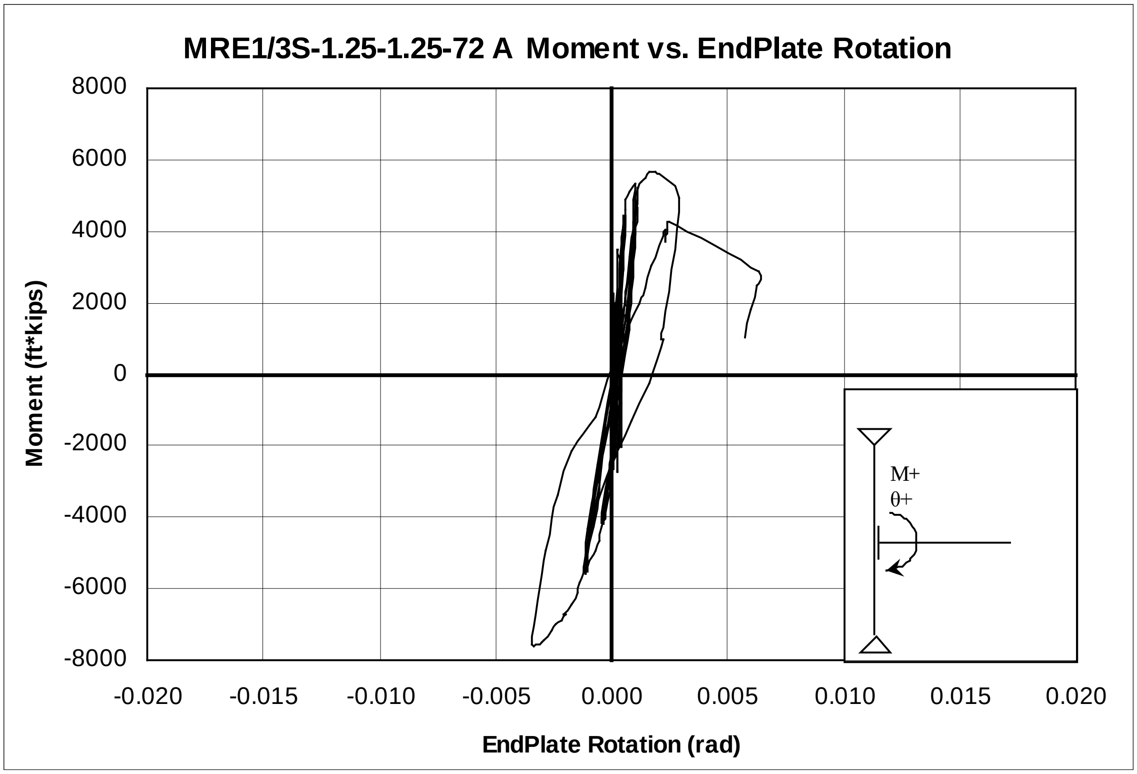
$M_{NP} := 2\cdot P_t\cdot\left(d_0 + d_1 + d_2 + d_3\right) \qquad M_{NP} = 6292.8k\cdot ft$

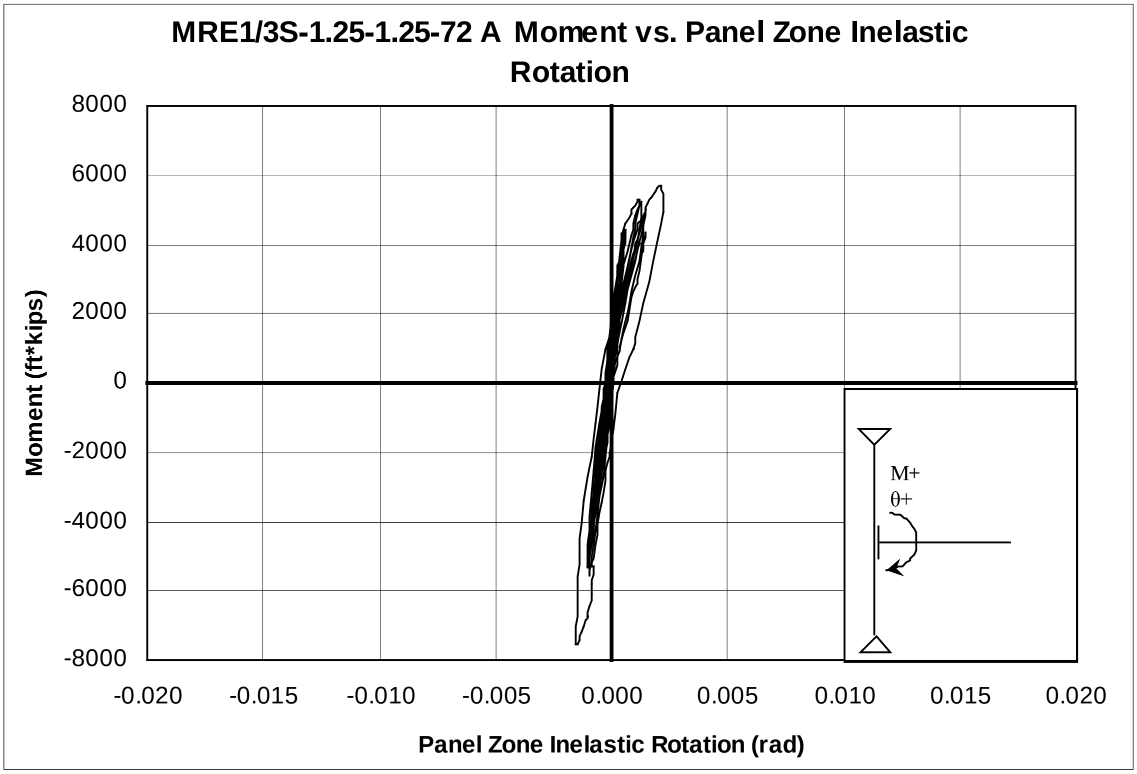

MRE1/3S-1.25-1.25-72 A Applied Load vs. Displacement
Applied Load (kips)
250
200
150
100
50
0
-50
-100
-150
-200
-250
-8
-6
-4
-2
0
2
4
6
8
Displacement (in)
P+
δ+

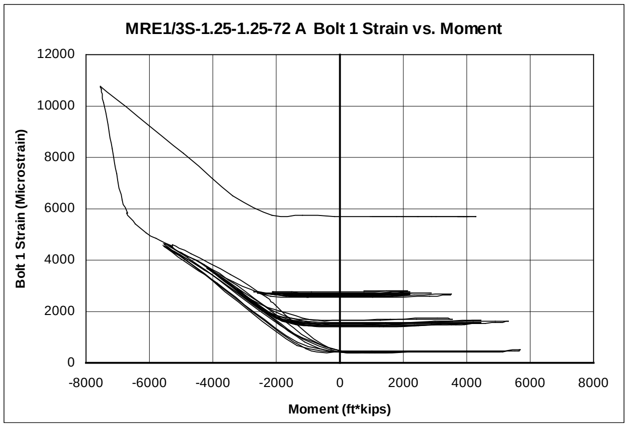

MRE1/3S-1.25-1.25-72 A Moment vs. Interstory Drift Angle
Moment (ft*kips)
8000
6000
4000
2000
0
-2000
-4000
-6000
-8000
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.010
0.015
0.020
Interstory Drift Angle (rad)
M+
θ+

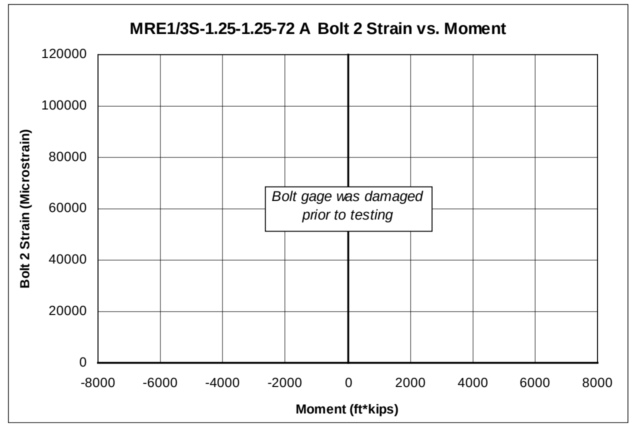

MRE1/3S-1.25-1.25-72 A Moment vs. Total Inelastic Rotation
Moment (ft*kips)
8000
6000
4000
2000
0
-2000
-4000
-6000
-8000
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.010
0.015
0.020
Total Inelastic Rotation (rad)
M+
θ+

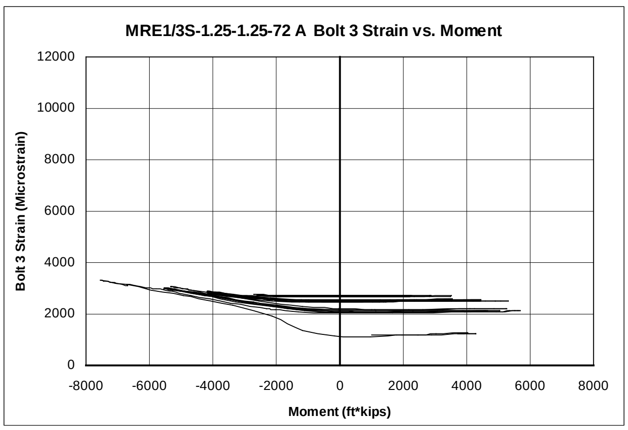

MRE1/3S-1.25-1.25-72 A Moment vs. Beam Inelastic Rotation
Moment (ft*kips)
8000
6000
4000
2000
0
-2000
-4000
-6000
-8000
-0.05
-0.04
-0.03
-0.02
-0.01
0.00
0.01
0.02
0.03
0.04
0.05
Beam Inelastic Rotation (rad)
M+
θ+

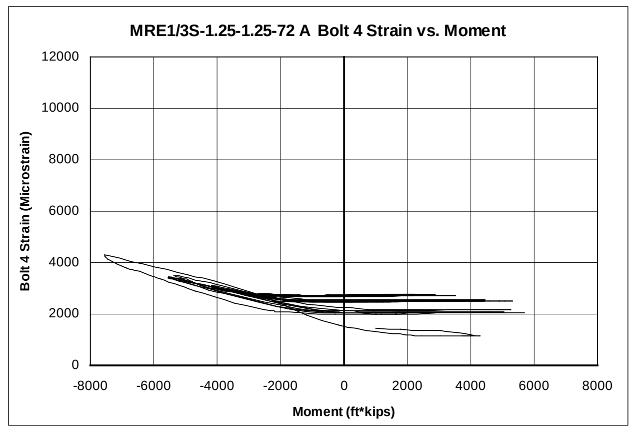

MRE1/3S-1.25-1.25-72 A Moment vs. EndPlate Rotation
8000
6000
4000
2000
0
-2000
-4000
-6000
-8000
Moment (ft*kips)
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.010
0.015
0.020
EndPlate Rotation (rad)
M+
θ+

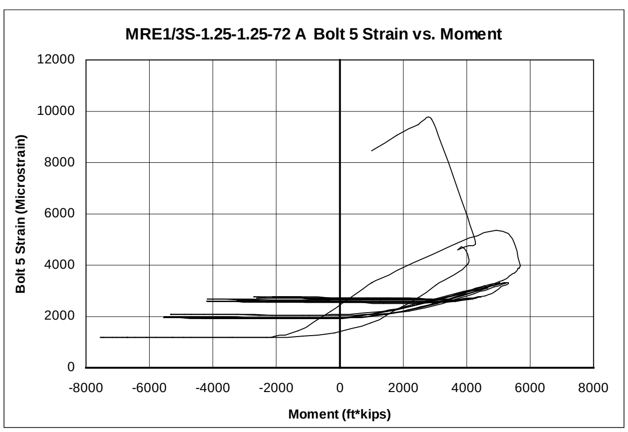

MRE1/3S-1.25-1.25-72 A Moment vs. Panel Zone Inelastic Rotation
8000
6000
4000
2000
0
-2000
-4000
-6000
-8000
Moment (ft*kips)
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.010
0.015
0.020
Panel Zone Inelastic Rotation (rad)
M+
θ+

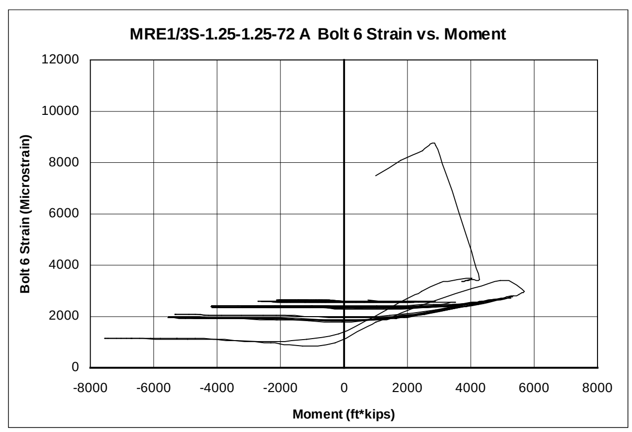

MRE1/3S-1.25-1.25-72 A Bolt 1 Strain vs. Moment
Bolt 1 Strain (Microstrain)
0
2000
4000
6000
8000
10000
12000
-8000
-6000
-4000
-2000
0
2000
4000
6000
8000
Moment (ft*kips)

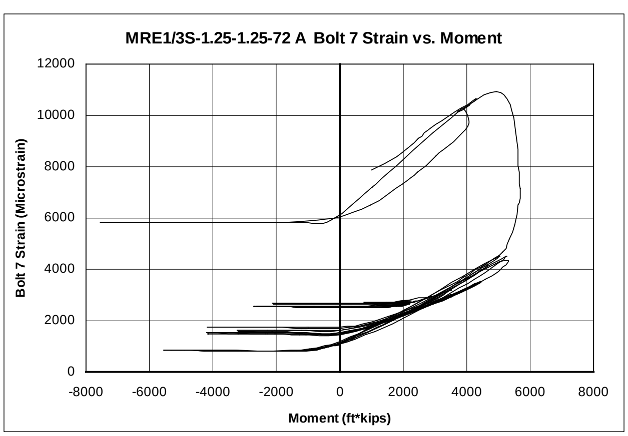

MRE1/3S-1.25-1.25-72 A Bolt 2 Strain vs. Moment
Bolt 2 Strain (Microstrain)
0
20000
40000
60000
80000
100000
120000
Bolt gage was damaged prior to testing
-8000
-6000
-4000
-2000
0
2000
4000
6000
8000
Moment (ft*kips)

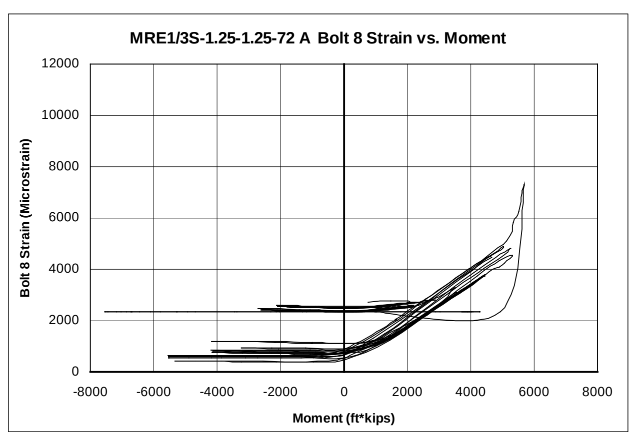

MRE1/3S-1.25-1.25-72 A Bolt 3 Strain vs. Moment
Bolt 3 Strain (Microstrain)
0
2000
4000
6000
8000
10000
12000
-8000
-6000
-4000
-2000
0
2000
4000
6000
8000
Moment (ft*kips)

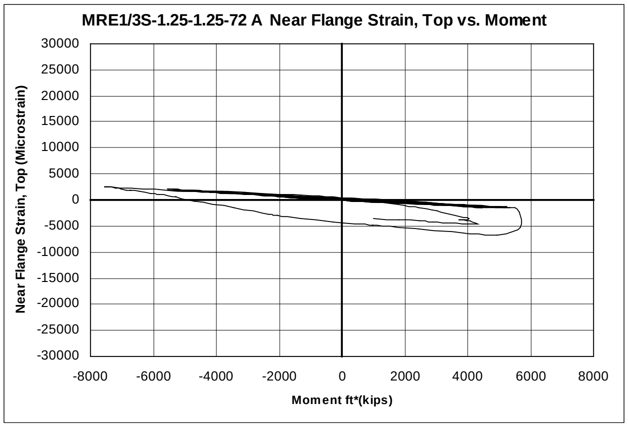

MRE1/3S-1.25-1.25-72 A Bolt 4 Strain vs. Moment
Bolt 4 Strain (Microstrain)
0
2000
4000
6000
8000
10000
12000
-8000
-6000
-4000
-2000
0
2000
4000
6000
8000
Moment (ft*kips)

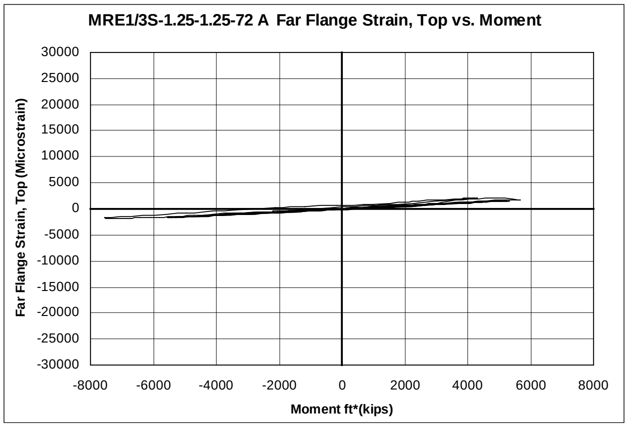

MRE1/3S-1.25-1.25-72 A Bolt 5 Strain vs. Moment
Bolt 5 Strain (Microstrain)
0
2000
4000
6000
8000
10000
12000
-8000
-6000
-4000
-2000
0
2000
4000
6000
8000
Moment (ft*kips)

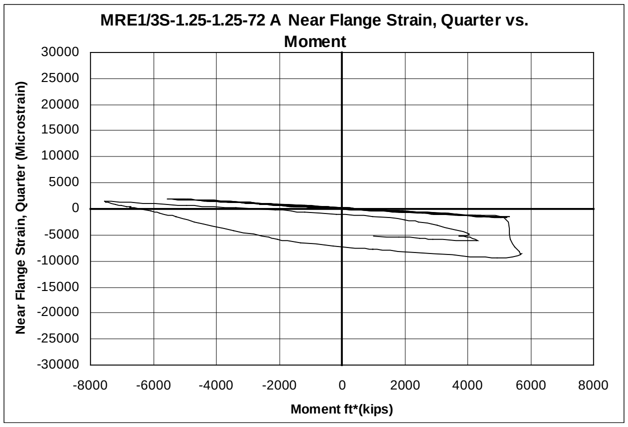

MRE1/3S-1.25-1.25-72 A Bolt 6 Strain vs. Moment
Bolt 6 Strain (Microstrain)
0
2000
4000
6000
8000
10000
12000
-8000
-6000
-4000
-2000
0
2000
4000
6000
8000
Moment (ft*kips)

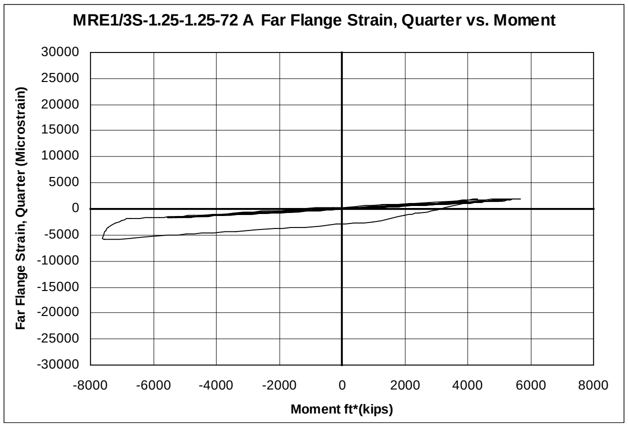

MRE1/3S-1.25-1.25-72 A Bolt 7 Strain vs. Moment
Bolt 7 Strain (Microstrain)
0
2000
4000
6000
8000
10000
12000
-8000
-6000
-4000
-2000
0
2000
4000
6000
8000
Moment (ft*kips)

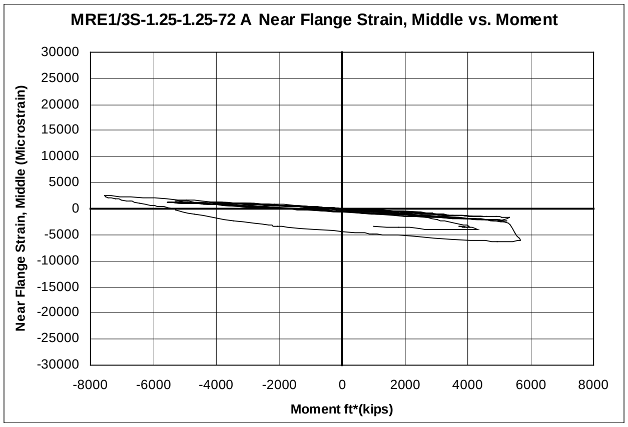

MRE1/3S-1.25-1.25-72 A Bolt 8 Strain vs. Moment
Bolt 8 Strain (Microstrain)
0
2000
4000
6000
8000
10000
12000
-8000
-6000
-4000
-2000
0
2000
4000
6000
8000
Moment (ft*kips)

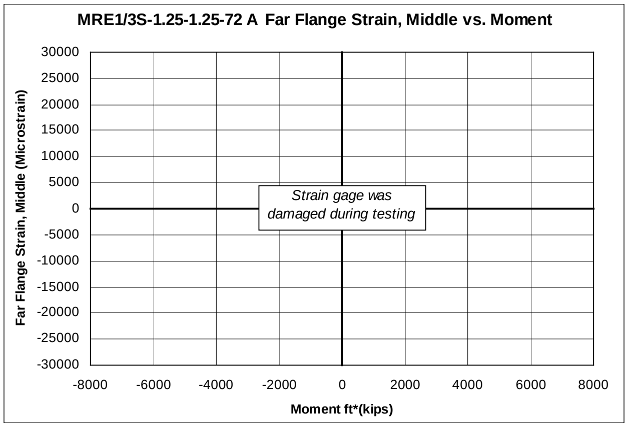

MRE1/3S-1.25-1.25-72 A Near Flange Strain, Top vs. Moment
Near Flange Strain, Top (Microstrain)
Moment ft*(kips)

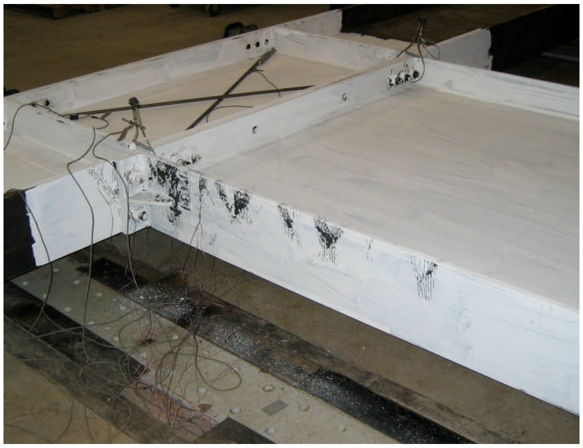

MRE1/3S-1.25-1.25-72 A Far Flange Strain, Top vs. Moment
Far Flange Strain, Top (Microstrain)
Moment ft*(kips)


MRE1/3S-1.25-1.25-72 A Near Flange Strain, Quarter vs. Moment
Near Flange Strain, Quarter (Microstrain)
Moment ft*(kips)

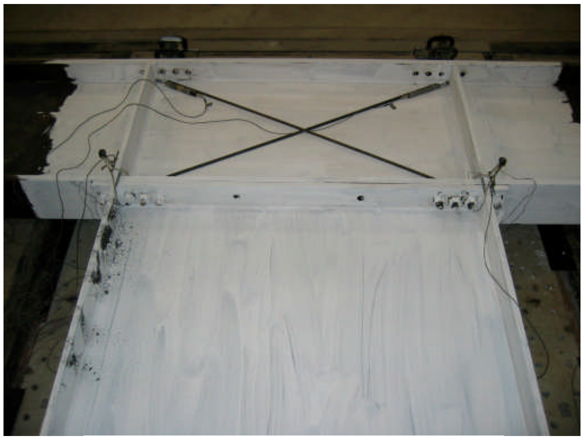

MRE1/3S-1.25-1.25-72 A Far Flange Strain, Quarter vs. Moment
Far Flange Strain, Quarter (Microstrain)
Moment ft*(kips)


MRE1/3S-1.25-1.25-72 A Near Flange Strain, Middle vs. Moment
Near Flange Strain, Middle (Microstrain)
Moment ft*(kips)


MRE1/3S-1.25-1.25-72 A Far Flange Strain, Middle vs. Moment
Far Flange Strain, Middle (Microstrain)
Strain gage was damaged during testing
Moment ft*(kips)


Near Flange


Far Flange at Bolt Rupture


Top View

MRE1/3S-1.25-1.25-72 A AFTER TESTING

APPENDIX K

MRE1/3S – 1.25 – 1.25 – 72 B

RESULTS AND TEST DATA

SPECIMEN PROPERTIES & TEST SUMMARY

TEST NAME: MRE1/3S-1.25-1.25-72 B
TEST DATE: June 18, 2004

BEAM DATA

DEPTH, d:	72.10	in
FLANGE WIDTH, b_f:	11.98	in
FLANGE THICKNESS, t_f:	1.01	in
WEB THICKNESS, t_w:	0.50	in
FLANGE YIELD STRESS, F_{yf}:	57.6	ksi
FLANGE ULTIMATE STRESS, F_{uf}:	83.5	ksi
WEB YIELD STRESS, F_{yw}:	72.3	ksi
WEB ULTIMATE STRESS, F_{uw}:	82.3	ksi

END-PLATE DATA

END-PLATE THICKNESS, t_p:	1.23	in
END-PLATE WIDTH, b_p:	12.00	in
END-PLATE LENGTH, L_p:	80.04	in
END-PLATE EXTENSION OUTSIDE FLANGE, p_{ext}:	3.97	in
OUTER PITCH, BOLT TO FLANGE, p_{fo}:	2.00	in
INNER PITCH, BOLT TO FLANGE, p_{fi}:	2.03	in
PITCH, BOLT TO BOLT, p_b:	3.00	in
GAGE, g:	6.00	in
END-PLATE YIELD STRESS, F_{yp}:	53.8	ksi
END-PLATE ULTIMATE STRESS, F_{up}:	79.8	ksi

BOLT DATA

BOLT DIAMETER, d_b:	1.25	in
BOLT LENGTH, L_b:	4.50	in
BOLT TYPE:	A490	
NOMINAL BOLT TENSILE STRESS (AISC J3.6), F_t:	113	ksi
NOMINAL BOLT TENSILE STRENGTH, P_t:	139	kips
BOLT PRETENSION, T_b:	102	kips

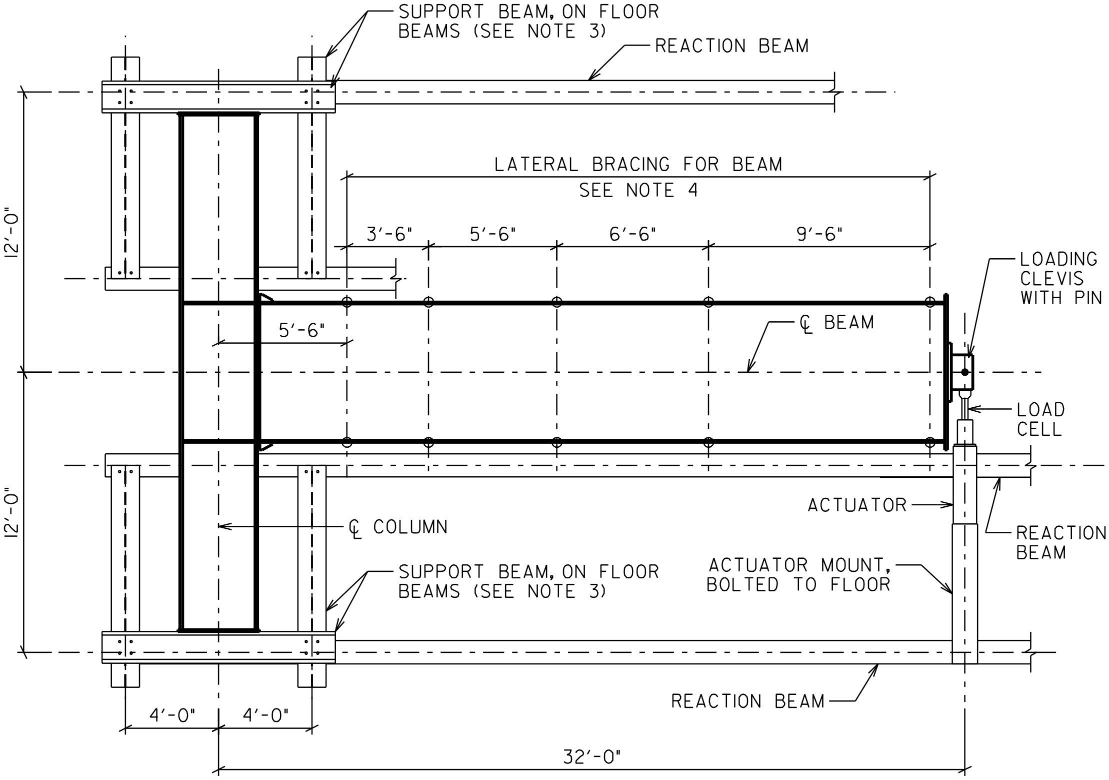
EXPERIMENTAL RESULTS

FAILURE MODE:	Bolt Tension Rupture & Bolt Thread Stripping	
MAXIMUM MOMENT AT COLUMN CENTERLINE, $M_{max,cc}$:	5975.8	ft-kips
MAXIMUM MOMENT AT FAYING SURFACE, $M_{max,fs}$:	5664.6	ft-kips
MAXIMUM INTERSTORY DRIFT ANGLE, θ_{max}:	0.013	rad
MAXIMUM INTERSTORY DRIFT ANGLE AT 80% 0F $M_{max,cc}$, θ_{preq}:	0.013	rad

CALCULATED STRENGTHS

BEAM MOMENT STRENGTH [1], M_b:	5877.8	ft-kips
BEAM PLASTIC STRENGTH [1], M_p:	7827.6	ft-kips
BEAM EXPECTED PLASTIC STRENGTH, M_{pe}:	7865.5	ft-kips
END-PLATE STRENGTH [1], M_{PL}:	6376.4	ft-kips
BOLT TENSION RUPTURE (w/o Prying, using F_t), M_{NP}:	6261.0	ft-kips

1. Measured material properties used.



MREI/3S-1.25-1.25-72 B TEST SUBASSEMBLAGE PLAN

BEAM

WEB = $\frac{1}{2}$×70, A572 Grade 50
FLANGE = 1×12, A572 Grade 55
LENGTH = 355 (INCLUDES ENDPLATES)

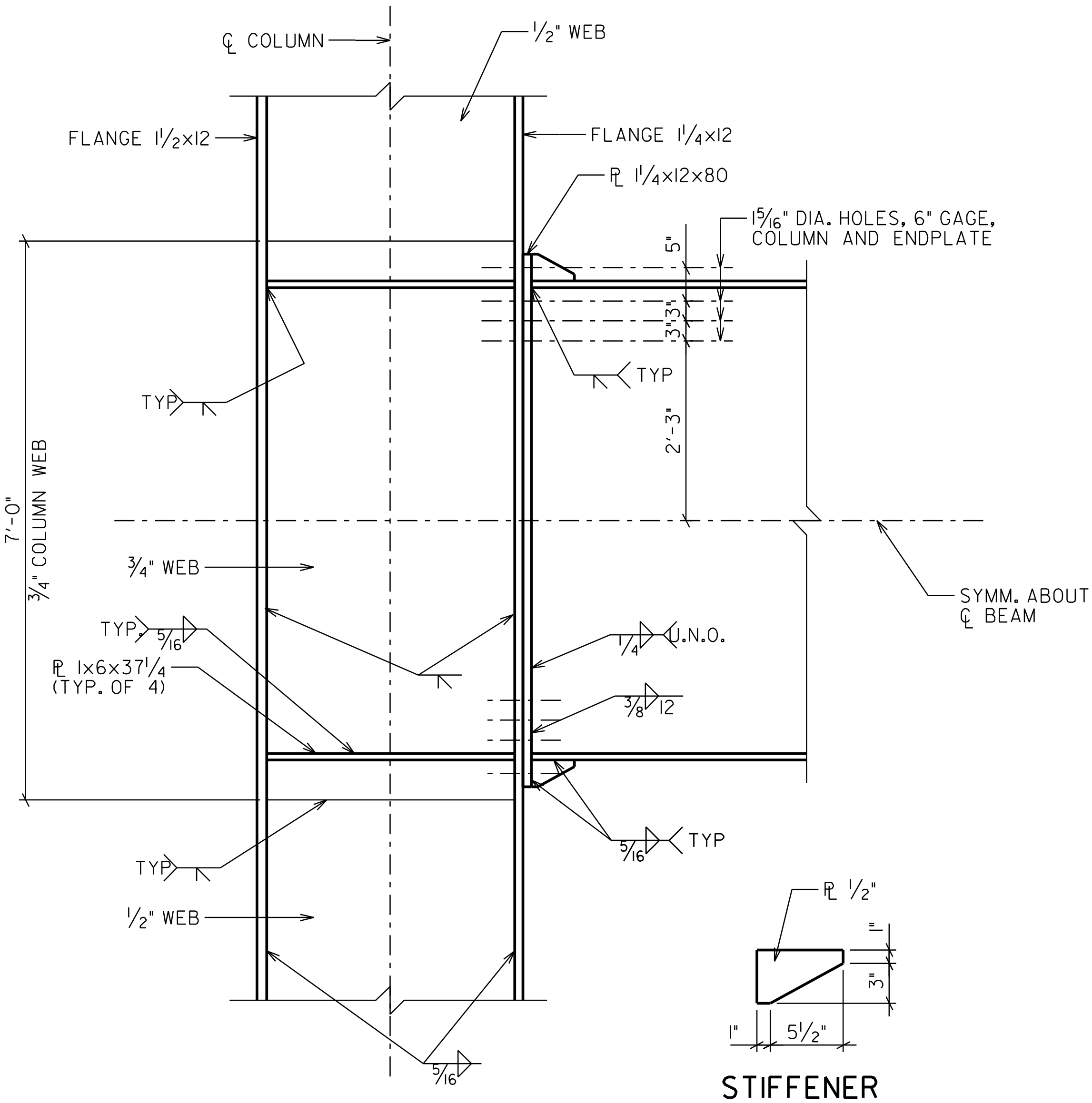
COLUMN

WEB = $\frac{1}{2}$×37$\frac{1}{4}$, A572 Grade 50
PANEL ZONE WEB = $\frac{3}{4}$×37$\frac{1}{4}$, A572 Grade 50
FLANGES = 1$\frac{1}{4}$×12 & 1$\frac{1}{2}$×12, A572 Grade 55
LENGTH = 266$\frac{1}{2}$ (INCLUDES ENDPLATES)

NOTES

1. ALL BOLTS, A490.
2. ALL WELDING, AWS D1.1, E70XX ELECTRODES.
3. COLUMN BOLTED TO SUPPORT BEAM. SUPPORT BEAM BOLTED TO FLOOR BEAMS. FLOOR BEAMS BOLTED TO REACTION BEAM.
4. COUPONS CUT FROM BEAM FLANGES PRIOR TO TESTING. EXTRA BRACING REQUIRED. BEAM STRENGTH AT CONNECTION NOT AFFECTED.

SPECIMEN: MREI/3S-1.25-1.25-72 B	DRAWN BY: SEB



MREI/3S-1.25-1.25-72 B CONNECTION DETAIL

SPECIMEN: MREI/3S-1.25-1.25-72 B	DRAWN BY: SEB

SAC PROTOCOL LOADING HISTORY

Test Name: MRE1/3S-1.25-1.25-72 B   Test By: SEB   Date: 6/18/2004

Load Step	Cycle	Max. Pos. Load (kips)	Max. Disp. (in)	Max. Neg. Load (kips)	Max. Disp. (in)	Comments
I (0.00375 rad)	1	67.5	1.437	-69.0	-1.442	
δ=1.44"	2	69.1	1.442	-69.4	-1.440	
	3	69.5	1.438	-69.6	-1.442	
	4	69.0	1.441	-69.7	-1.436	
	5	69.8	1.448	-69.9	-1.439	
	6	70.1	1.454	-70.3	-1.440	
		Permanent Set =	-0.009"			
II (0.005 rad)	1	92.3	1.933	-83.0	-1.930	← Bolt banging jostled
δ=1.92"	2	97.0	1.920	-89.0	-1.920	potentiometer. Tightened bolts
	3	96.8	1.925	-88.8	-1.925	on support framing, and
	4	96.1	1.923	-89.0	-1.925	rezeroed test
	5	95.9	1.917	-89.3	-1.933	
	6	96.3	1.926	-89.3	-1.930	
		Permanent Set =	-0.095"			
III (0.0075 rad)	1	137.8	2.877	-132.7	-2.874	
δ=2.88"	2	137.9	2.877	-131.2	-2.877	
	3	138.2	2.875	-131.6	-2.887	
	4	138.5	2.876	-130.8	-2.879	
	5	138.8	2.878	-130.8	-2.885	
	6	139.0	2.883	-130.9	-2.879	Whitewash flaking, beam flange
		Permanent Set =	-0.124"			
IV (0.01 rad)	1	168.7	3.848	-166.2	-3.842	
δ=3.84"	2	170.0	3.848	-166.0	-3.843	
	3	170.2	3.863	-167.0	-3.868	
	4	171.3	3.876	-166.0	-3.843	
		Permanent Set =	-0.135"			
V (0.015 rad)	1	~ 190	~ 5	~ -180	~ -4.5	Single bolt, tension rupture,
δ=5.76"	2					and bolt thread stripping
		Permanent Set =				
VI (0.02 rad)	1					
	2					
		Permanent Set =				
VII (0.03 rad)	1					
	2					
		Permanent Set =				
VIII (0.04 rad)	1					
	2					
		Permanent Set =				
IX (0.05 rad)	1					
	2					
		Permanent Set =				

CALCULATED STRENGTHS

TEST NAME: MRE1/3S-1.25-1.25-72 B
TEST DATE: June 18, 2004

Specified Grade for Beam Web Steel = ASTM A572, Grade 50
Specified Grade for all other Steel = ASTM A572, Grade 55

MEASURED PROPERTIES

$d := 72.1 \cdot in$ $\quad F_{yf} := 57.6 \cdot ksi$ $\quad t_p := 1.23 \cdot in$ $\quad p_{fo} := 2.00 \cdot in$ $\quad p_b := 3.00 \cdot in$

$b_f := 11.98 \cdot in$ $\quad F_{uf} := 83.5 \cdot ksi$ $\quad b_p := 12.00 \cdot in$ $\quad p_{fi} := 2.03 \cdot in$

$t_f := 1.01 \cdot in$ $\quad F_{yw} := 72.3 \cdot ksi$ $\quad L_p := 80.04 \cdot in$ $\quad g := 6.00 \cdot in$

$t_w := 0.50 \cdot in$ $\quad F_{uw} := 82.3 \cdot ksi$ $\quad p_{ext} := 3.97 \cdot in$ $\quad F_{yp} := 53.8 \cdot ksi$ $\quad h := d - 2 \cdot t_f$

SPECIFIED PROPERTIES

$R_y := 1.1$ $\quad P_t := 139 \cdot k$

BEAM PLASTIC STRENGTH, Mp

$$M_p := 2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right) \cdot F_{yf} + 2 \cdot t_w \cdot \frac{h}{2} \cdot \frac{h}{4} \cdot F_{yw} \qquad M_p = 7827.6 \, k \cdot ft$$

BEAM EXPECTED PLASTIC STRENGTH, Mpe

$$M_{pe} := 1.1 R_y \cdot \left[2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right) \cdot 55 \cdot ksi + 2 \cdot t_w \cdot \frac{h}{2} \cdot \frac{h}{4} \cdot 50 \cdot ksi\right] \qquad M_{pe} = 7865.5 \, k \cdot ft$$

BEAM MOMENT STRENGTH, Mb

$\frac{b_f}{2 \cdot t_f} = 5.9$ $\quad \frac{65}{\sqrt{55}} = 8.8$ $\quad$ Flange is compact

$\frac{970}{\sqrt{50}} = 137.2$ $\quad \frac{h}{t_w} = 140.2$ $\quad$ Web is slender, refer to Appendix G

Therefore, Mb< Mp

$$I_x := 2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right)^2 + \frac{1}{12} \cdot t_w \cdot h^3 \qquad I_x = 44916 \, in^4 \qquad S_x := \frac{I_x}{0.5 \cdot d} \qquad S_x = 1245.9 \, in^3$$

$$a_r := \frac{h \cdot t_w}{b_f \cdot t_f} \qquad m := \frac{55}{55} \qquad R_e := \frac{12 + a_r \cdot \left(3 \cdot m - m^3\right)}{12 + 2 \cdot a_r} \qquad R_e = 1$$

BEAM MOMENT STRENGTH, Mb (CONT'D)

$I_T := \frac{1}{12}\cdot\left(\frac{0.5\cdot d}{3}\right)\cdot t_w^{\ 3} + \frac{1}{12}\cdot t_f\cdot b_f^{\ 3}$ $\quad I_T = 144.8\,in^4$

$A_T := \left(\frac{0.5\cdot d}{3}\right)\cdot t_w + t_f\cdot b_f$ $\quad A_T = 18.1\,in^2$ $\quad r_T := \sqrt{\frac{I_T}{A_T}}$ $\quad r_T = 2.8\,in$

Required L_b: $\quad L_b := 40.4\cdot r_T$ $\quad L_b = 9.5\,ft$ $\quad$ If the unbraced length is less than L_b: $\quad F_{cr} := F_{yf}$

$$R_{PG} := 1 - \left(\frac{a_r}{1200 + 300\cdot a_r}\right)\cdot\left(\frac{h}{t_w} - 5.7\cdot\sqrt{\frac{29000\,ksi}{F_{cr}}}\right)$$

$M_b := S_x\cdot R_{PG}\cdot R_e\cdot F_{cr}$ $\quad M_b = 5877.8\,k\cdot ft$

END-PLATE STRENGTH, Mpl

$d_0 := d - \frac{t_f}{2} + p_{fo}$ $\quad d_0 = 73.6\,in$ $\quad h_0 := d_0 + \frac{t_f}{2}$ $\quad h_0 = 74.1\,in$

$d_1 := d - \frac{t_f}{2} - t_f - p_{fi}$ $\quad d_1 = 68.6\,in$ $\quad h_1 := d_1 + \frac{t_f}{2}$ $\quad h_1 = 69.1\,in$

$d_2 := d_1 - p_b$ $\quad d_2 = 65.6\,in$ $\quad h_2 := d_2 + \frac{t_f}{2}$ $\quad h_2 = 66.1\,in$

$d_3 := d_2 - p_b$ $\quad d_3 = 62.6\,in$ $\quad h_3 := d_3 + \frac{t_f}{2}$ $\quad h_3 = 63.1\,in$

$s := \frac{1}{2}\cdot\sqrt{b_p\cdot g}$ $\quad s = 4.24\,in$ $\quad d_e := p_{ext} - p_{fo}$ $\quad d_e = 2\,in$

$d_e < s$, therefore Case 2 governs

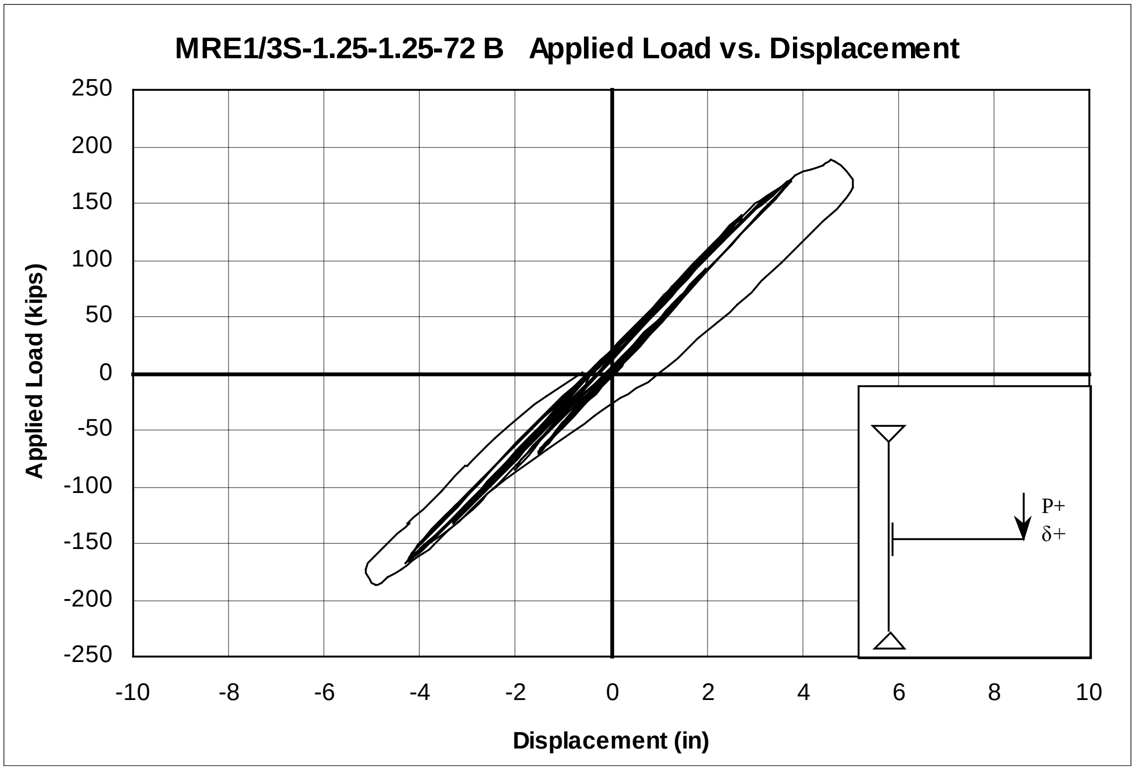
$$Y := \frac{b_p}{2}\cdot\left[\frac{h_1}{p_{fi}} + \frac{h_3}{s} + h_0\cdot\left(\frac{1}{p_{fo}} + \frac{1}{2\cdot s}\right)\right] + \frac{2}{g}\cdot\left[h_1\cdot\left(p_{fi} + 1.5\cdot p_b\right) + h_3\cdot\left(s + 0.5\cdot p_b\right) + h_0\cdot\left(d_e + p_{fo}\right)\right] + \frac{g}{2}$$

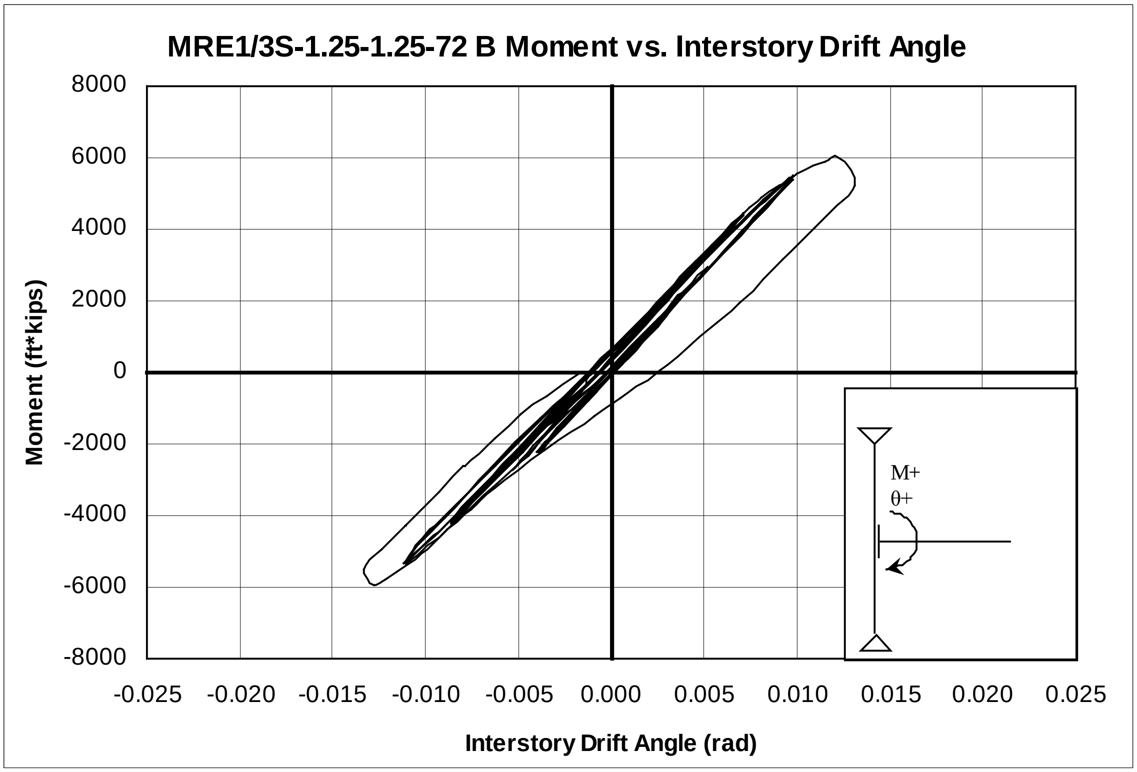
$Y = 940.1\,in$

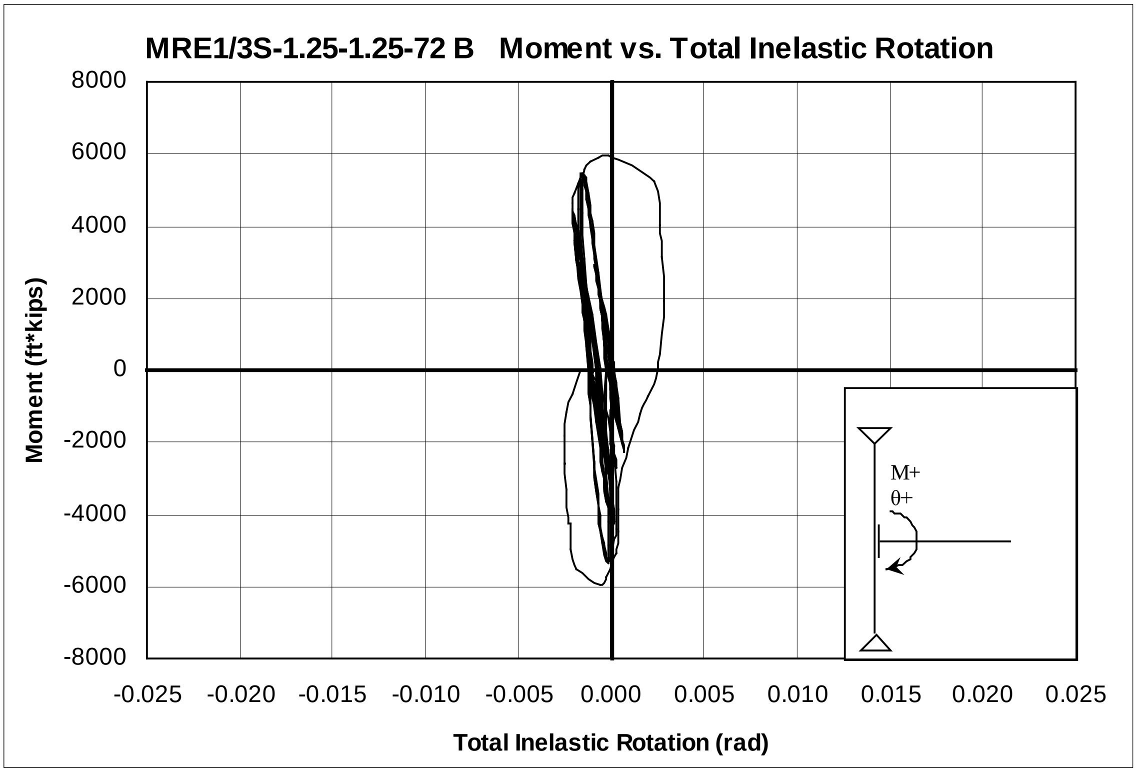
$M_{PL} := t_p^{\ 2}\cdot F_{yp}\cdot Y$ $\quad M_{PL} = 6376.4\,k\cdot ft$

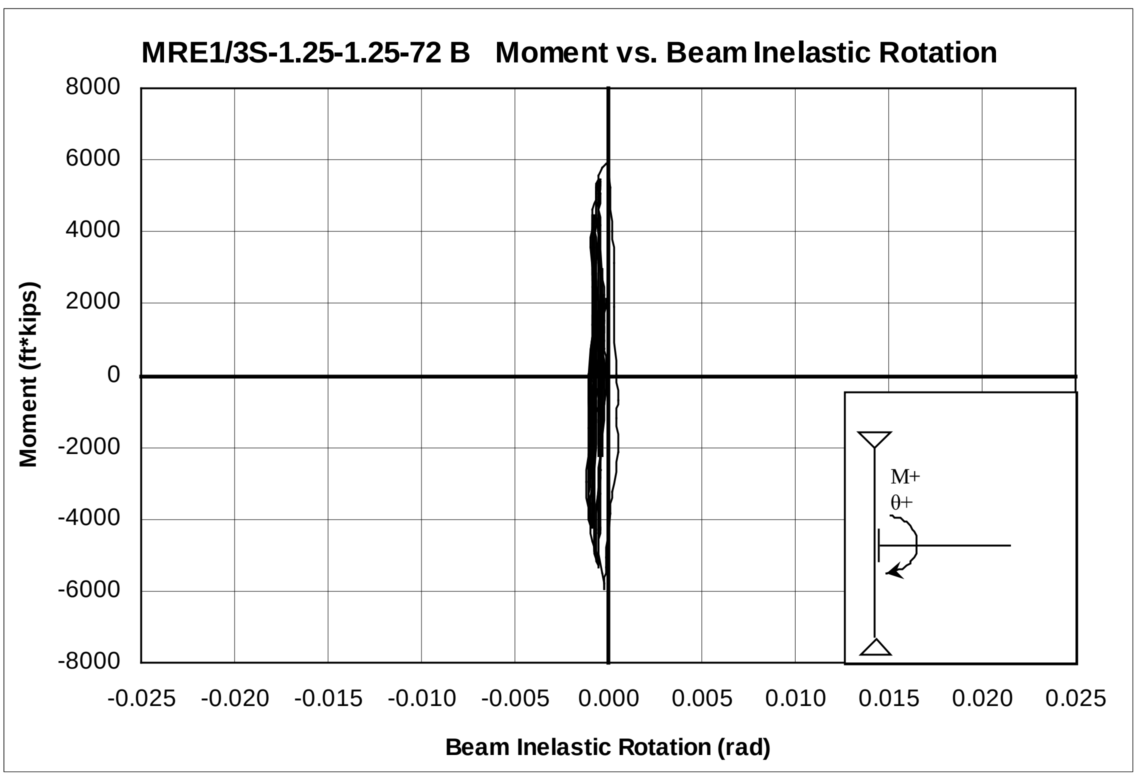
BOLT STRENGTH WITHOUT PRYING, Mnp

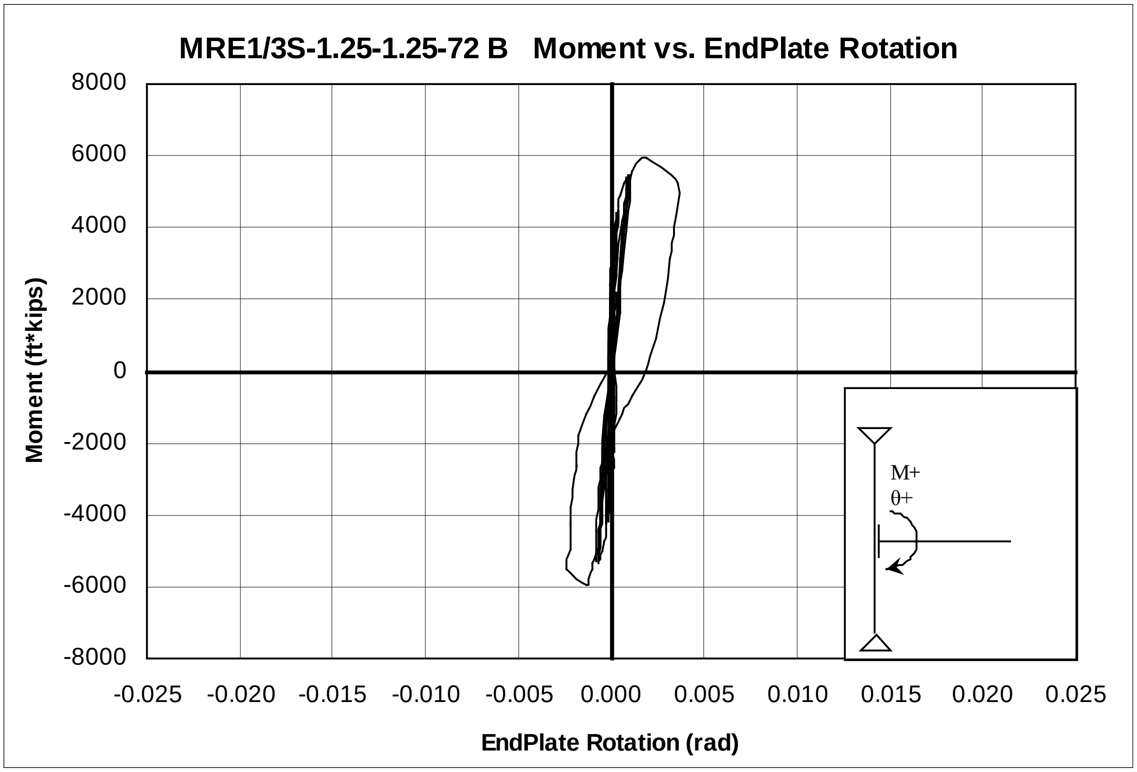
$M_{NP} := 2\cdot P_t\cdot\left(d_0 + d_1 + d_2 + d_3\right)$ $\quad M_{NP} = 6261\,k\cdot ft$

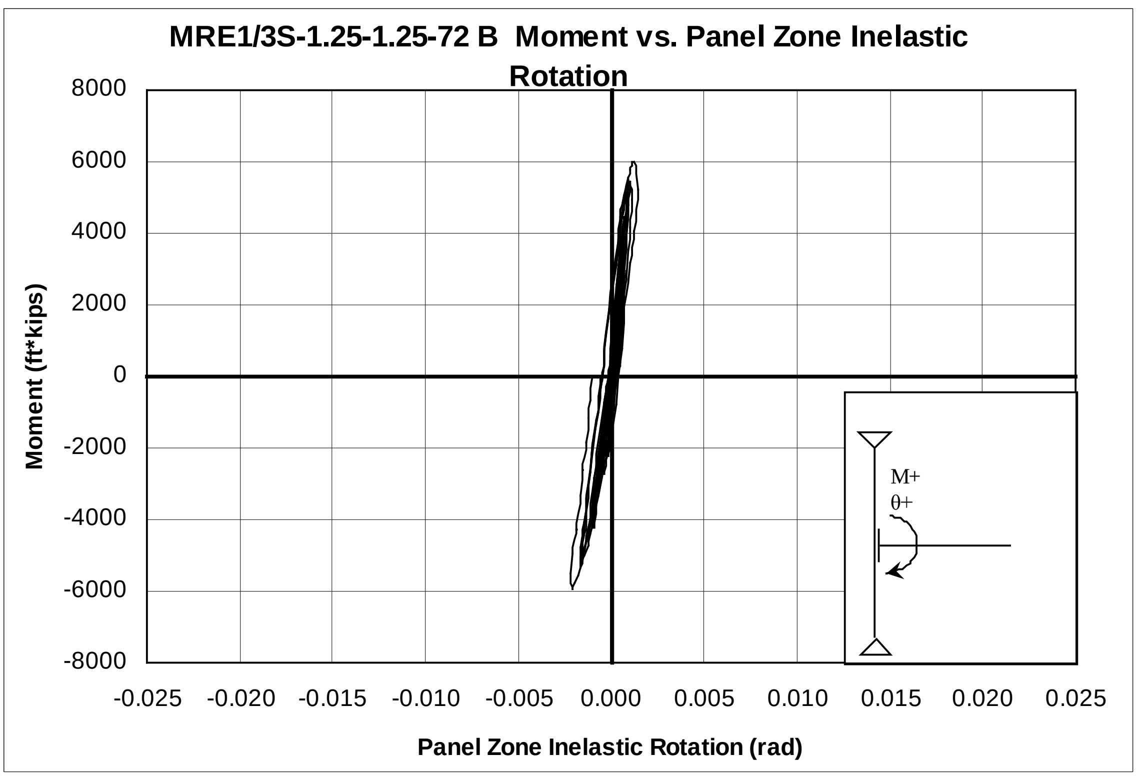

MRE1/3S-1.25-1.25-72 B Applied Load vs. Displacement
Applied Load (kips)
Displacement (in)
250
200
150
100
50
0
-50
-100
-150
-200
-250
-10
-8
-6
-4
-2
0
2
4
6
8
10
P+
δ+

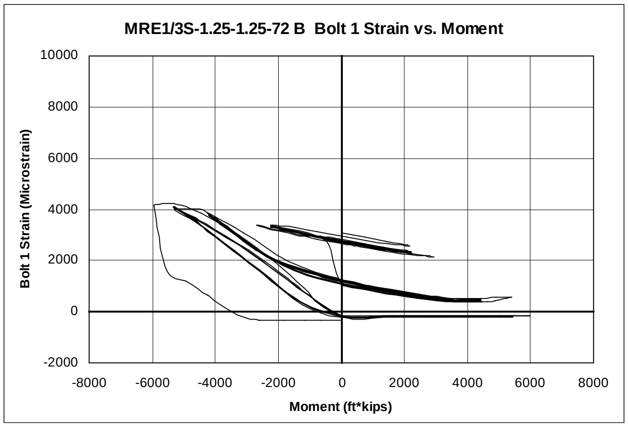

MRE1/3S-1.25-1.25-72 B Moment vs. Interstory Drift Angle
Moment (ft*kips)
Interstory Drift Angle (rad)
8000
6000
4000
2000
0
-2000
-4000
-6000
-8000
-0.025
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.010
0.015
0.020
0.025
M+
θ+

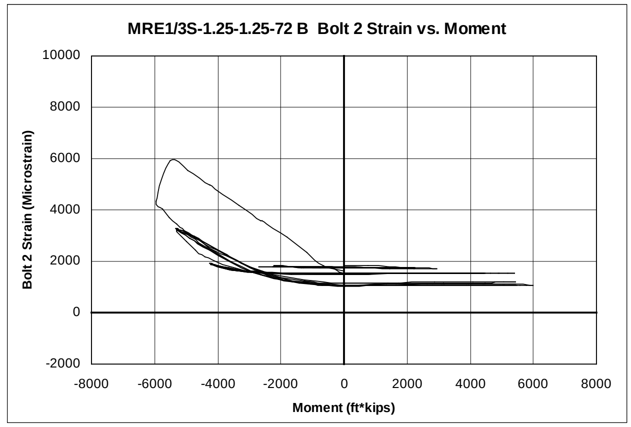

MRE1/3S-1.25-1.25-72 B Moment vs. Total Inelastic Rotation
Moment (ft*kips)
8000
6000
4000
2000
0
-2000
-4000
-6000
-8000
-0.025 -0.020 -0.015 -0.010 -0.005 0.000 0.005 0.010 0.015 0.020 0.025
Total Inelastic Rotation (rad)
M+
θ+

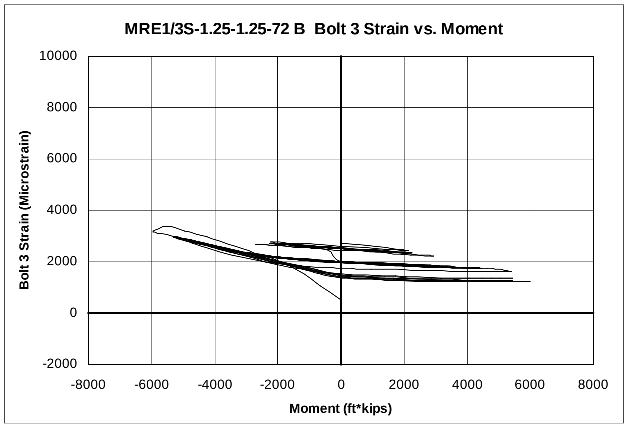

MRE1/3S-1.25-1.25-72 B Moment vs. Beam Inelastic Rotation
Moment (ft*kips)
8000
6000
4000
2000
0
-2000
-4000
-6000
-8000
-0.025 -0.020 -0.015 -0.010 -0.005 0.000 0.005 0.010 0.015 0.020 0.025
Beam Inelastic Rotation (rad)
M+
θ+

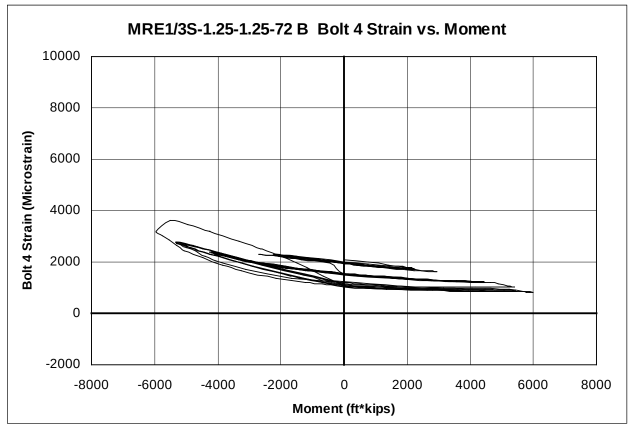

MRE1/3S-1.25-1.25-72 B Moment vs. EndPlate Rotation
8000
6000
4000
2000
0
-2000
-4000
-6000
-8000
Moment (ft*kips)
-0.025
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.010
0.015
0.020
0.025
EndPlate Rotation (rad)
M+
θ+

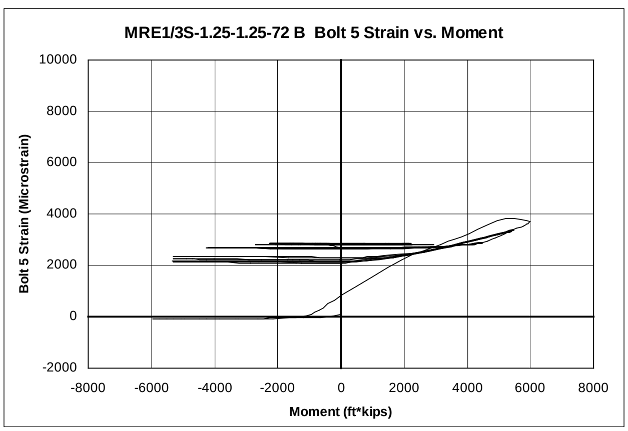

MRE1/3S-1.25-1.25-72 B Moment vs. Panel Zone Inelastic Rotation
8000
6000
4000
2000
0
-2000
-4000
-6000
-8000
Moment (ft*kips)
-0.025
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.010
0.015
0.020
0.025
Panel Zone Inelastic Rotation (rad)
M+
θ+

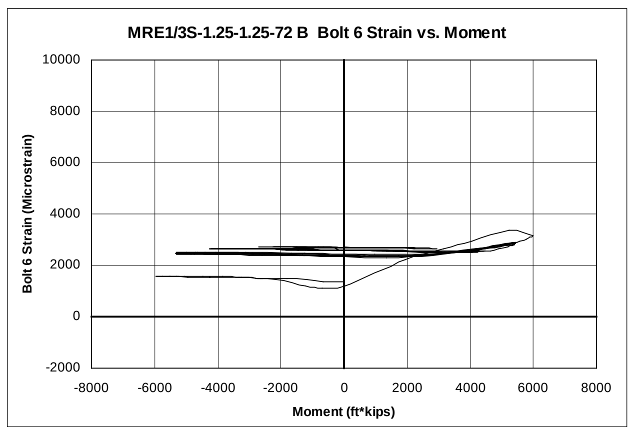

MRE1/3S-1.25-1.25-72 B Bolt 1 Strain vs. Moment
Bolt 1 Strain (Microstrain)
10000
8000
6000
4000
2000
0
-2000
-8000
-6000
-4000
-2000
0
2000
4000
6000
8000
Moment (ft*kips)

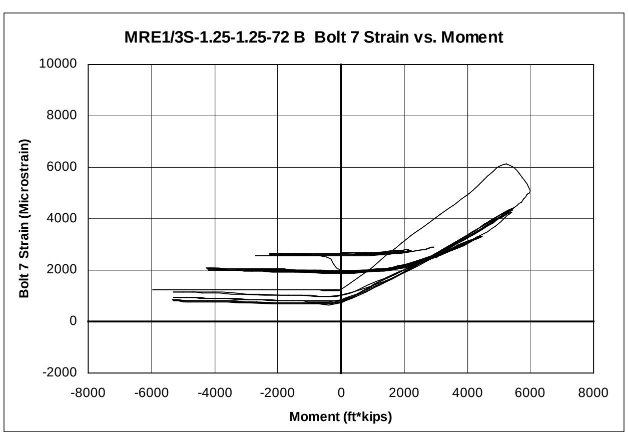

MRE1/3S-1.25-1.25-72 B Bolt 2 Strain vs. Moment
Bolt 2 Strain (Microstrain)
10000
8000
6000
4000
2000
0
-2000
-8000
-6000
-4000
-2000
0
2000
4000
6000
8000
Moment (ft*kips)

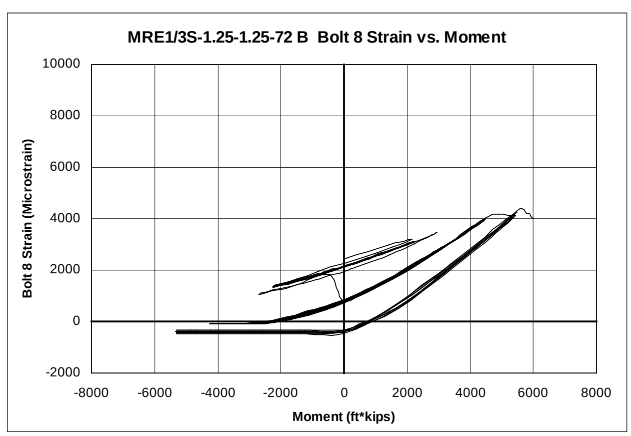

MRE1/3S-1.25-1.25-72 B Bolt 3 Strain vs. Moment
Bolt 3 Strain (Microstrain)
10000
8000
6000
4000
2000
0
-2000
-8000
-6000
-4000
-2000
0
2000
4000
6000
8000
Moment (ft*kips)

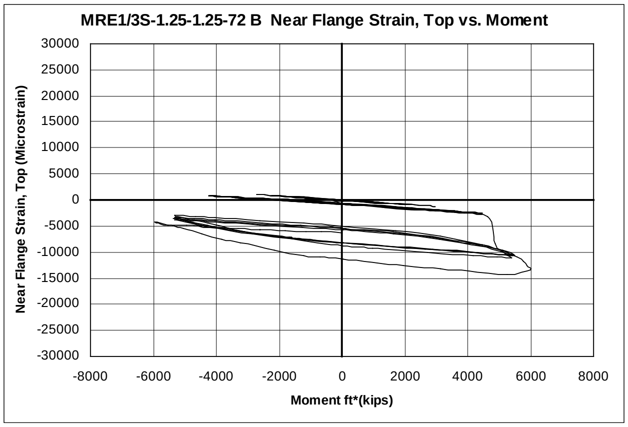

MRE1/3S-1.25-1.25-72 B Bolt 4 Strain vs. Moment
Bolt 4 Strain (Microstrain)
10000
8000
6000
4000
2000
0
-2000
-8000
-6000
-4000
-2000
0
2000
4000
6000
8000
Moment (ft*kips)

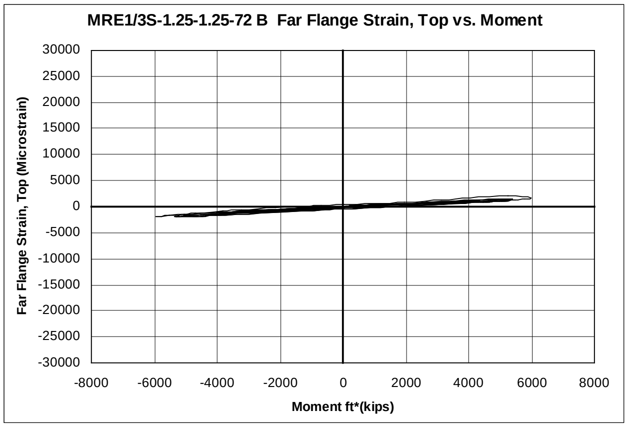

MRE1/3S-1.25-1.25-72 B Bolt 5 Strain vs. Moment
Bolt 5 Strain (Microstrain)
10000
8000
6000
4000
2000
0
-2000
-8000
-6000
-4000
-2000
0
2000
4000
6000
8000
Moment (ft*kips)

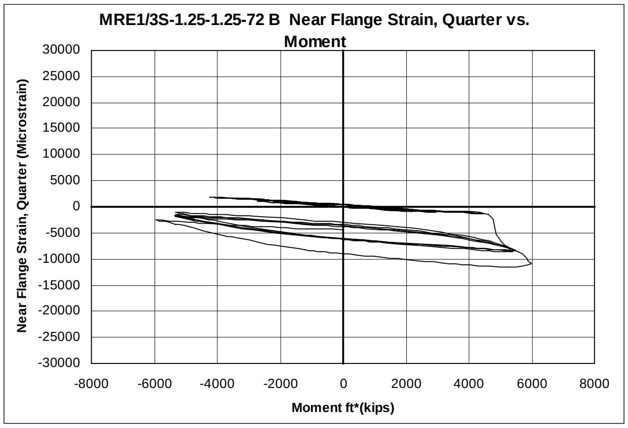

MRE1/3S-1.25-1.25-72 B Bolt 6 Strain vs. Moment
Bolt 6 Strain (Microstrain)
10000
8000
6000
4000
2000
0
-2000
-8000
-6000
-4000
-2000
0
2000
4000
6000
8000
Moment (ft*kips)

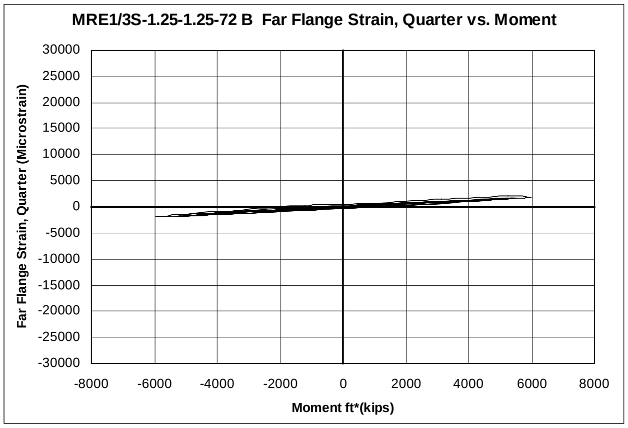

MRE1/3S-1.25-1.25-72 B Bolt 7 Strain vs. Moment
Bolt 7 Strain (Microstrain)
10000
8000
6000
4000
2000
0
-2000
-8000
-6000
-4000
-2000
0
2000
4000
6000
8000
Moment (ft*kips)

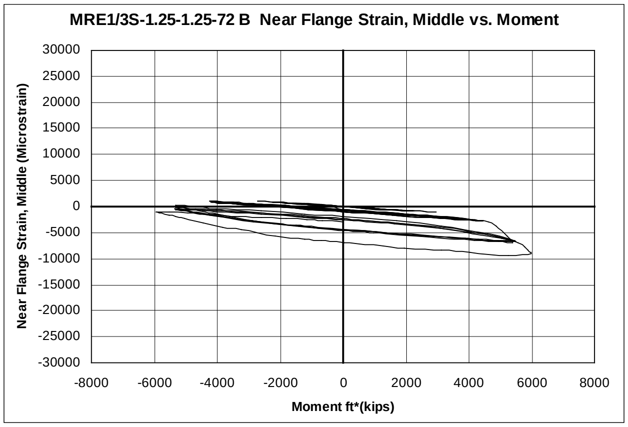

MRE1/3S-1.25-1.25-72 B Bolt 8 Strain vs. Moment
Bolt 8 Strain (Microstrain)
10000
8000
6000
4000
2000
0
-2000
-8000
-6000
-4000
-2000
0
2000
4000
6000
8000
Moment (ft*kips)

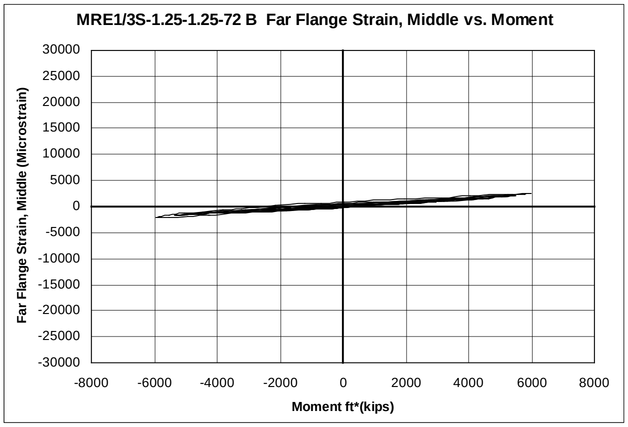

MRE1/3S-1.25-1.25-72 B Near Flange Strain, Top vs. Moment
Near Flange Strain, Top (Microstrain)
Moment ft*(kips)

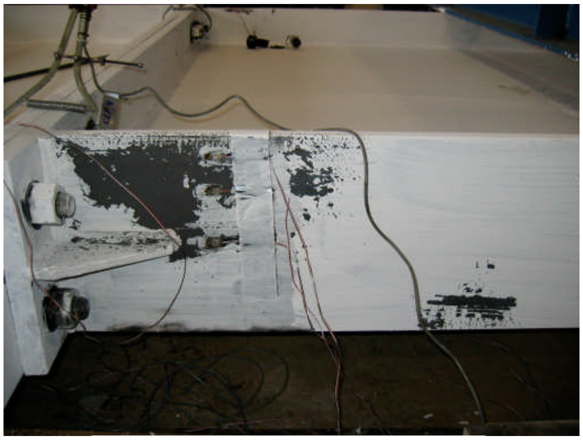

MRE1/3S-1.25-1.25-72 B Far Flange Strain, Top vs. Moment
Far Flange Strain, Top (Microstrain)
Moment ft*(kips)

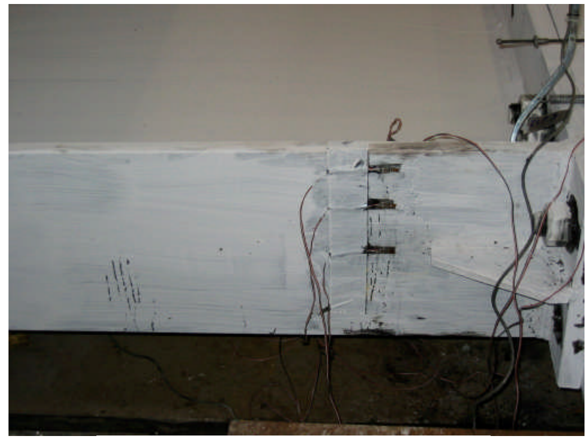

MRE1/3S-1.25-1.25-72 B Near Flange Strain, Quarter vs. Moment
Near Flange Strain, Quarter (Microstrain)
Moment ft*(kips)


MRE1/3S-1.25-1.25-72 B Far Flange Strain, Quarter vs. Moment
Far Flange Strain, Quarter (Microstrain)
Moment ft*(kips)


MRE1/3S-1.25-1.25-72 B Near Flange Strain, Middle vs. Moment
Near Flange Strain, Middle (Microstrain)
Moment ft*(kips)


MRE1/3S-1.25-1.25-72 B Far Flange Strain, Middle vs. Moment
Far Flange Strain, Middle (Microstrain)
Moment ft*(kips)


Near Flange


Far Flange


Far Flange with Ruptured Bolt

MRE1/3S-1.25-1.25-72 B AFTER TESTING

APPENDIX L

MRE1/3 – 1.25 – 1.5 – 72

RESULTS AND TEST DATA

SPECIMEN PROPERTIES & TEST SUMMARY

TEST NAME: MRE1/3-1.25-1.5-72
TEST DATE: June 11, 2004

BEAM DATA

DEPTH, d:	72.10	in
FLANGE WIDTH, b_f:	11.98	in
FLANGE THICKNESS, t_f:	1.01	in
WEB THICKNESS, t_w:	0.50	in
FLANGE YIELD STRESS, F_{yf}:	57.6	ksi
FLANGE ULTIMATE STRESS, F_{uf}:	83.5	ksi
WEB YIELD STRESS, F_{yw}:	72.3	ksi
WEB ULTIMATE STRESS, F_{uw}:	82.3	ksi

END-PLATE DATA

END-PLATE THICKNESS, t_p:	1.51	in
END-PLATE WIDTH, b_p:	12.02	in
END-PLATE LENGTH, L_p:	80.06	in
END-PLATE EXTENSION OUTSIDE FLANGE, p_{ext}:	3.98	in
OUTER PITCH, BOLT TO FLANGE, p_{fo}:	2.01	in
INNER PITCH, BOLT TO FLANGE, p_{fi}:	1.99	in
PITCH, BOLT TO BOLT, p_b:	3.00	in
GAGE, g:	6.01	in
END-PLATE YIELD STRESS, F_{yp}:	52.9	ksi
END-PLATE ULTIMATE STRESS, F_{up}:	74.9	ksi

BOLT DATA

BOLT DIAMETER, d_b:	1.25	in
BOLT LENGTH, L_b:	5.00	in
BOLT TYPE:	A490	
NOMINAL BOLT TENSILE STRESS (AISC J3.6), F_t:	113	ksi
NOMINAL BOLT TENSILE STRENGTH, P_t:	139	kips
BOLT PRETENSION, T_b:	102	kips

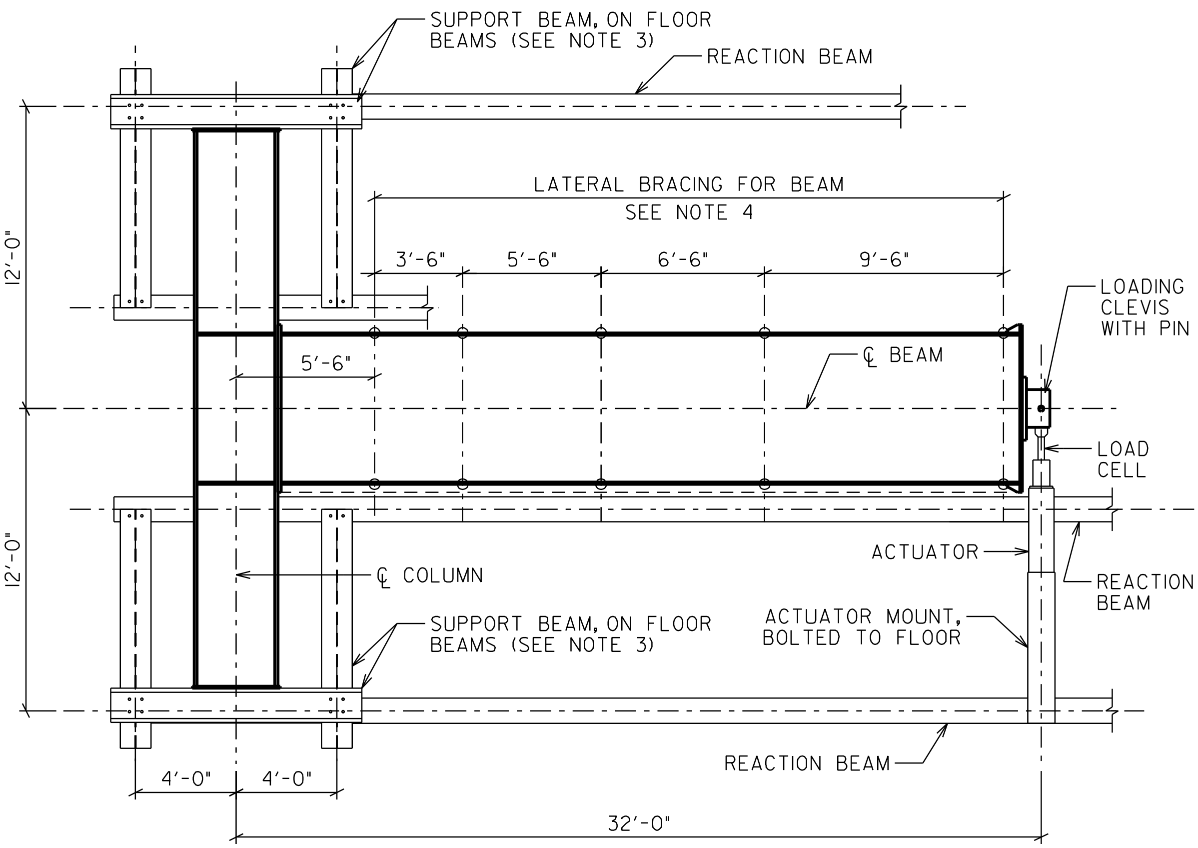
EXPERIMENTAL RESULTS

FAILURE MODE:	Bolt Thread Stripping	
MAXIMUM MOMENT AT COLUMN CENTERLINE, $M_{max,cc}$:	6688.1	ft-kips
MAXIMUM MOMENT AT FAYING SURFACE, $M_{max,fs}$:	6339.8	ft-kips
MAXIMUM INTERSTORY DRIFT ANGLE, θ_{max}:	0.016	rad
MAXIMUM INTERSTORY DRIFT ANGLE AT 80% 0F $M_{max,cc}$, θ_{preq}:	0.016	rad

CALCULATED STRENGTHS

BEAM MOMENT STRENGTH [1], M_n:	5877.8	ft-kips
BEAM PLASTIC STRENGTH [1], M_p:	7827.6	ft-kips
BEAM EXPECTED PLASTIC STRENGTH, M_{pe}:	7865.5	ft-kips
END-PLATE STRENGTH [1], M_{PL}:	7935.5	ft-kips
BOLT TENSION RUPTURE (w/o Prying, using F_t), M_{NP}:	6264.0	ft-kips

1. Measured material properties used.



MREI/3-1.25-1.5-72 TEST SUBASSEMBLAGE PLAN

BEAM

WEB = 1/2×70, A572 Grade 50
FLANGE = 1×12, A572 Grade 55
LENGTH = 355 (INCLUDES ENDPLATES)

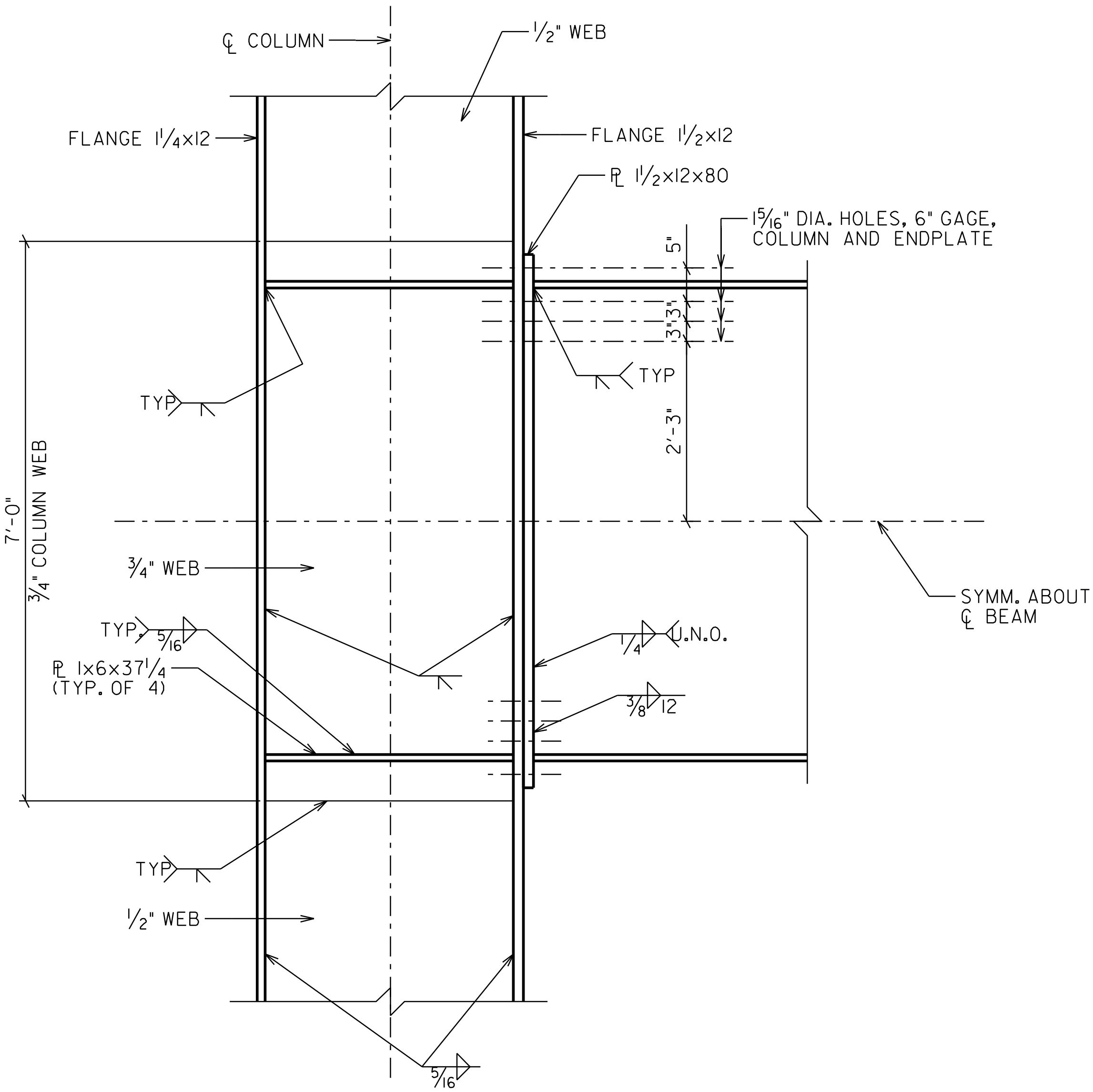
COLUMN

WEB = 1/2×37 1/4, A572 Grade 50
PANEL ZONE WEB = 3/4×37 1/4, A572 Grade 50
FLANGES = 1 1/4×12 & 1 1/2×12, A572 Grade 55
LENGTH = 266 1/2 (INCLUDES ENDPLATES)

NOTES

1. ALL BOLTS, A490.
2. ALL WELDING, AWS D1.1, E70XX ELECTRODES.
3. COLUMN BOLTED TO SUPPORT BEAM. SUPPORT BEAM BOLTED TO FLOOR BEAMS. FLOOR BEAMS BOLTED TO REACTION BEAM.
4. COUPONS CUT FROM BEAM FLANGES PRIOR TO TESTING. EXTRA BRACING REQUIRED. BEAM STRENGTH AT CONNECTION NOT AFFECTED.

SPECIMEN: MREI/3-1.25-1.5-72	DRAWN BY: SEB



MREI/3-1.25-1.5-72
CONNECTION DETAIL

SPECIMEN: MREI/3-1.25-1.5-72	DRAWN BY: SEB

SAC PROTOCOL LOADING HISTORY

Test Name: MRE1/3-1.25-1.5-72 Test By: SEB Date: 6/11/2004

Load Step	Cycle	Max. Pos. Load (kips)	Max. Disp. (in)	Max. Neg. Load (kips)	Max. Disp. (in)	Comments
I (0.00375 rad)	1	68.6	1.445	-71.7	-1.426	
δ=1.44"	2	69.2	1.437	-72.0	-1.427	
	3	69.2	1.451	-72.4	-1.431	
	4	69.6	1.447	-72.3	-1.439	
	5	69.7	1.449	-72.4	-1.435	
	6	69.6	1.456	-72.6	-1.440	
		Permanent Set =	-0.018"			
II (0.005 rad)	1	91.4	1.933	-96.1	-1.921	
δ=1.92"	2	91.9	1.925	-96.2	-1.920	
	3	92.6	1.947	-96.1	-1.923	
	4	91.4	1.919	-96.5	-1.924	
	5	91.9	1.924	-96.4	-1.920	
	6	91.0	1.920	-95.9	-1.920	
		Permanent Set =	-0.037"			
III (0.0075 rad)	1	134.5	2.878	-137.0	-2.877	
δ=2.88"	2	137.3	2.890	-138.0	-2.880	
	3	137.9	2.894	-137.9	-2.872	
	4	137.9	2.893	-138.4	-2.882	
	5	137.1	2.877	-138.5	-2.885	Whitewash flaking, col flange,
	6	138.2	2.906	-138.1	-2.874	adjacent to endplate
		Permanent Set =	-0.031"			
IV (0.01 rad)	1	172.9	3.842	-171.2	-3.838	Whitewash flaking, col flange,
δ=3.84"	2	173.8	3.847	-172.7	-3.836	opposite from connection
	3	173.5	3.842	-172.7	-3.848	
	4	174.0	3.838	-171.7	-3.837	
		Permanent Set =	-0.219"			
V (0.015 rad)	1	206.0	5.767	-204.0	-5.762	Whitewash flaking, bm flange
δ=5.76"	2	206.0	5.766	-208.0	-5.763	
		Permanent Set =	-0.7"			
VI (0.02 rad)	1	~ 210	~ 6.2			Threads stripped on four
δ=7.68"	2					outside bolts
		Permanent Set =				
VII (0.03 rad)	1					
	2					
		Permanent Set =				
VIII (0.04 rad)	1					
	2					
		Permanent Set =				
IX (0.05 rad)	1					
	2					
		Permanent Set =				

CALCULATED STRENGTHS

TEST NAME: MRE1/3-1.25-1.5-72
TEST DATE: June 11, 2004

Specified Grade for Beam Web Steel = ASTM A572, Grade 50
Specified Grade for all other Steel = ASTM A572, Grade 55

MEASURED PROPERTIES

$d := 72.1 \cdot in$	$F_{yf} := 57.6 \cdot ksi$	$t_p := 1.51 \cdot in$	$p_{fo} := 2.01 \cdot in$	$p_b := 3.00 \cdot in$
$b_f := 11.98 \cdot in$	$F_{uf} := 83.5 \cdot ksi$	$b_p := 12.02 \cdot in$	$p_{fi} := 1.99 \cdot in$	
$t_f := 1.01 \cdot in$	$F_{yw} := 72.3 \cdot ksi$	$L_p := 80.06 \cdot in$	$g := 6.01 \cdot in$	
$t_w := 0.50 \cdot in$	$F_{uw} := 82.3 \cdot ksi$	$p_{ext} := 3.98 \cdot in$	$F_{yp} := 52.9 \cdot ksi$	$h := d - 2 \cdot t_f$

SPECIFIED PROPERTIES

$R_y := 1.1$ $\quad P_t := 139 \cdot k$

BEAM PLASTIC STRENGTH, Mp

$$M_p := 2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right) \cdot F_{yf} + 2 \cdot t_w \cdot \frac{h}{2} \cdot \frac{h}{4} \cdot F_{yw} \qquad M_p = 7827.6 k \cdot ft$$

BEAM EXPECTED PLASTIC STRENGTH, Mpe

$$M_{pe} := 1.1 R_y \cdot \left[2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right) \cdot 55 \cdot ksi + 2 \cdot t_w \cdot \frac{h}{2} \cdot \frac{h}{4} \cdot 50 \cdot ksi\right] \qquad M_{pe} = 7865.5 k \cdot ft$$

BEAM MOMENT STRENGTH, Mb

$\frac{b_f}{2 \cdot t_f} = 5.9$ $\quad \frac{65}{\sqrt{55}} = 8.8$ $\quad$ Flange is compact

$\frac{970}{\sqrt{50}} = 137.2$ $\quad \frac{h}{t_w} = 140.2$ $\quad$ Web is slender, refer to Appendix G

Therefore, Mb< Mp

$$I_x := 2 \cdot t_f \cdot b_f \cdot \left(\frac{d}{2} - \frac{t_f}{2}\right)^2 + \frac{1}{12} \cdot t_w \cdot h^3 \qquad I_x = 44916 in^4 \qquad S_x := \frac{I_x}{0.5 \cdot d} \qquad S_x = 1245.9 in^3$$

$$a_r := \frac{h \cdot t_w}{b_f \cdot t_f} \qquad m := \frac{55}{55} \qquad R_e := \frac{12 + a_r \cdot \left(3 \cdot m - m^3\right)}{12 + 2 \cdot a_r} \qquad R_e = 1$$

BEAM MOMENT STRENGTH, Mb (CONT'D)

$$I_T := \frac{1}{12}\cdot\left(\frac{0.5\cdot d}{3}\right)\cdot t_w^{3} + \frac{1}{12}\cdot t_f\cdot b_f^{3} \qquad I_T = 144.8in^4$$

$$A_T := \left(\frac{0.5\cdot d}{3}\right)\cdot t_w + t_f\cdot b_f \qquad A_T = 18.1in^2 \qquad r_T := \sqrt{\frac{I_T}{A_T}} \qquad r_T = 2.8in$$

Required L_b: $L_b := 40.4\,r_T$ $\quad L_b = 9.5ft$ $\quad$ If the unbraced length is less than L_b: $F_{cr} := F_{yf}$

$$R_{PG} := 1 - \left(\frac{a_r}{1200 + 300\cdot a_r}\right)\cdot\left(\frac{h}{t_w} - 5.7\cdot\sqrt{\frac{29000ksi}{F_{cr}}}\right)$$

$$M_b := S_x\cdot R_{PG}\cdot R_e\cdot F_{cr} \qquad M_b = 5877.8k\cdot ft$$

END-PLATE STRENGTH, Mpl

$$d_0 := d - \frac{t_f}{2} + p_{fo} \qquad d_0 = 73.6in \qquad h_0 := d_0 + \frac{t_f}{2} \qquad h_0 = 74.1in$$

$$d_1 := d - \frac{t_f}{2} - t_f - p_{fi} \qquad d_1 = 68.6in \qquad h_1 := d_1 + \frac{t_f}{2} \qquad h_1 = 69.1in$$

$$d_2 := d_1 - p_b \qquad d_2 = 65.6in \qquad h_2 := d_2 + \frac{t_f}{2} \qquad h_2 = 66.1in$$

$$d_3 := d_2 - p_b \qquad d_3 = 62.6in \qquad h_3 := d_3 + \frac{t_f}{2} \qquad h_3 = 63.1in$$

$$s := \frac{1}{2}\cdot\sqrt{b_p\cdot g} \qquad s = 4.25in$$

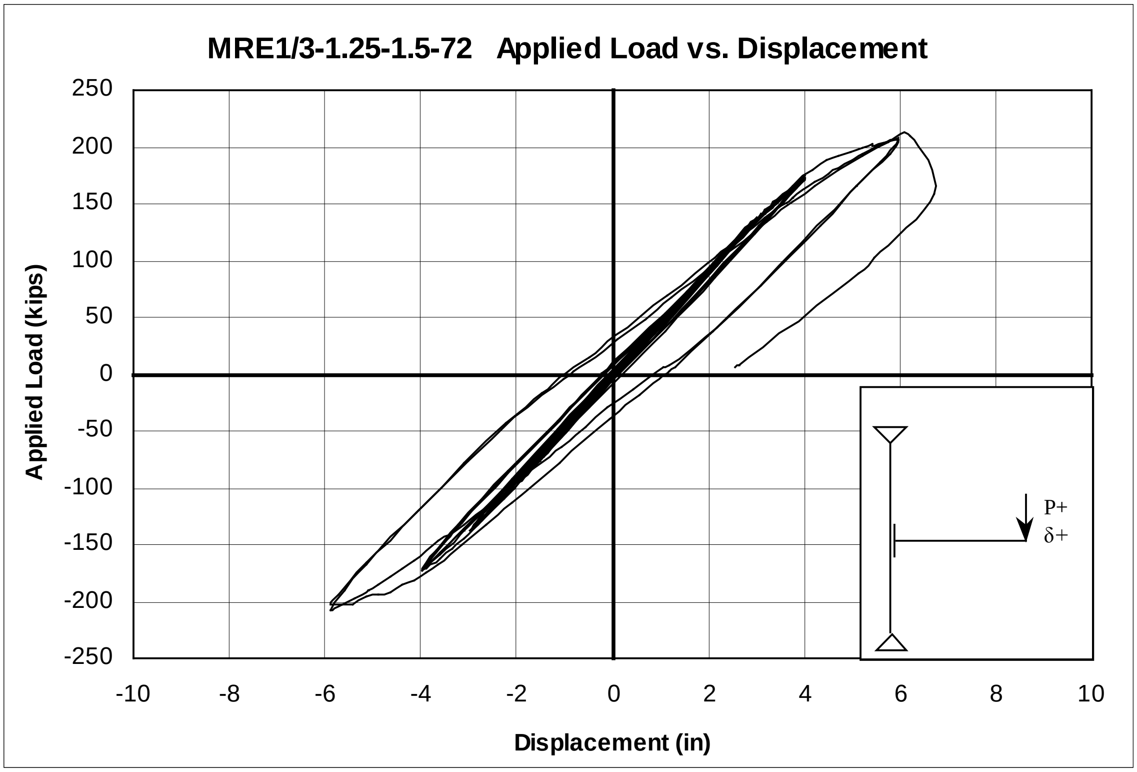
$$Y := \frac{b_p}{2}\cdot\left(\frac{h_1}{p_{fi}} + \frac{h_3}{s} + \frac{h_0}{p_{fo}} - \frac{1}{2}\right) + \frac{2}{g}\cdot\left[h_1\cdot\left(p_{fi} + 1.5\cdot p_b\right) + h_3\cdot\left(s + 0.5\cdot p_b\right)\right] + \frac{g}{2}$$

$$Y = 789.5in$$

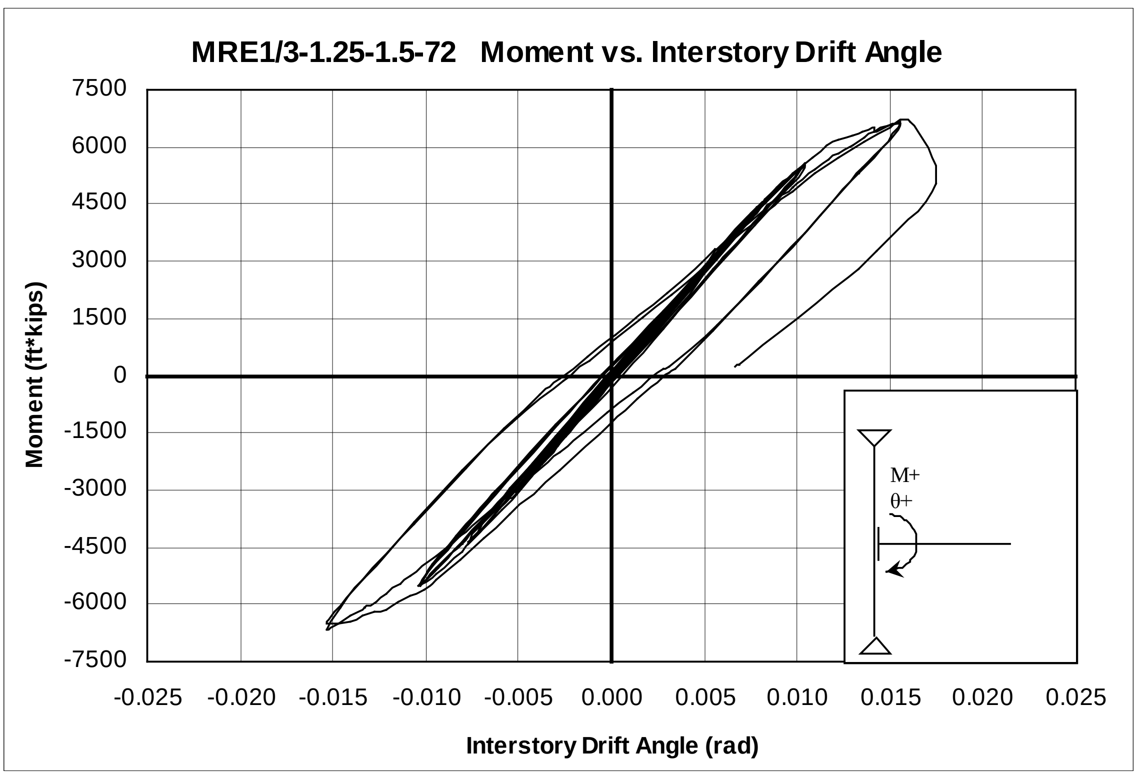
$$M_{PL} := t_p^{2}\cdot F_{yp}\cdot Y \qquad M_{PL} = 7935.5k\cdot ft$$

BOLT STRENGTH WITHOUT PRYING, Mnp

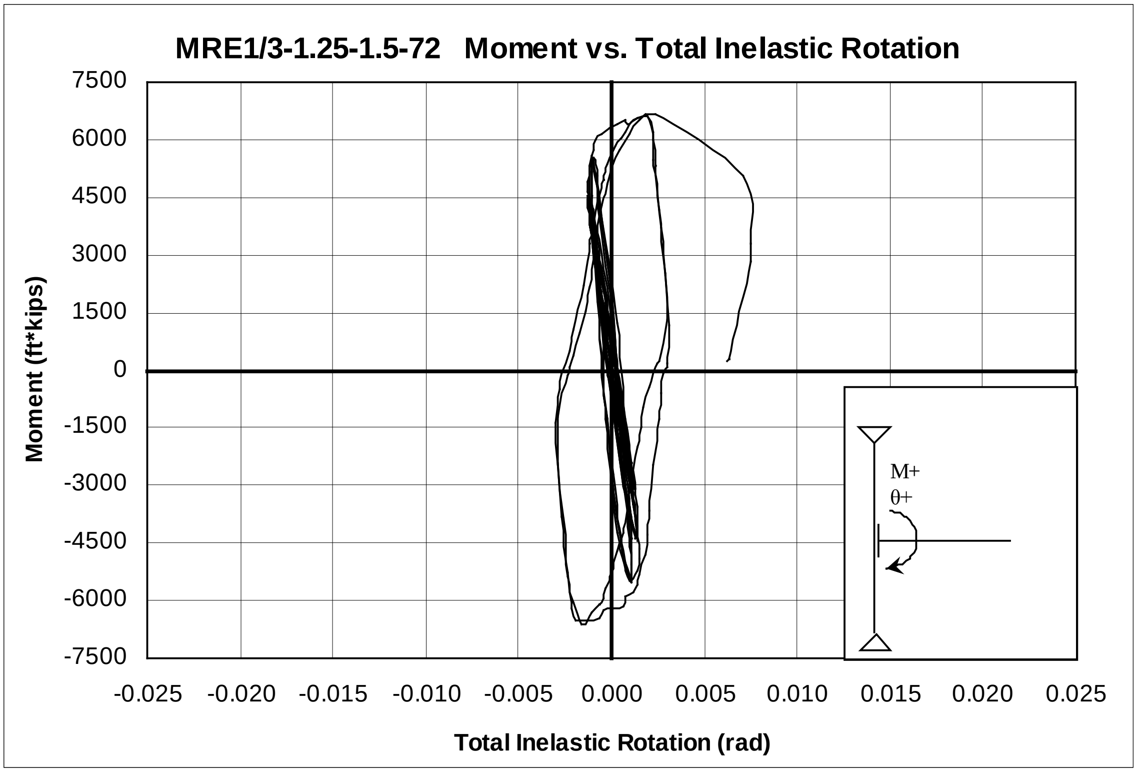
$$M_{NP} := 2\cdot P_t\cdot\left(d_0 + d_1 + d_2 + d_3\right) \qquad M_{NP} = 6264k\cdot ft$$

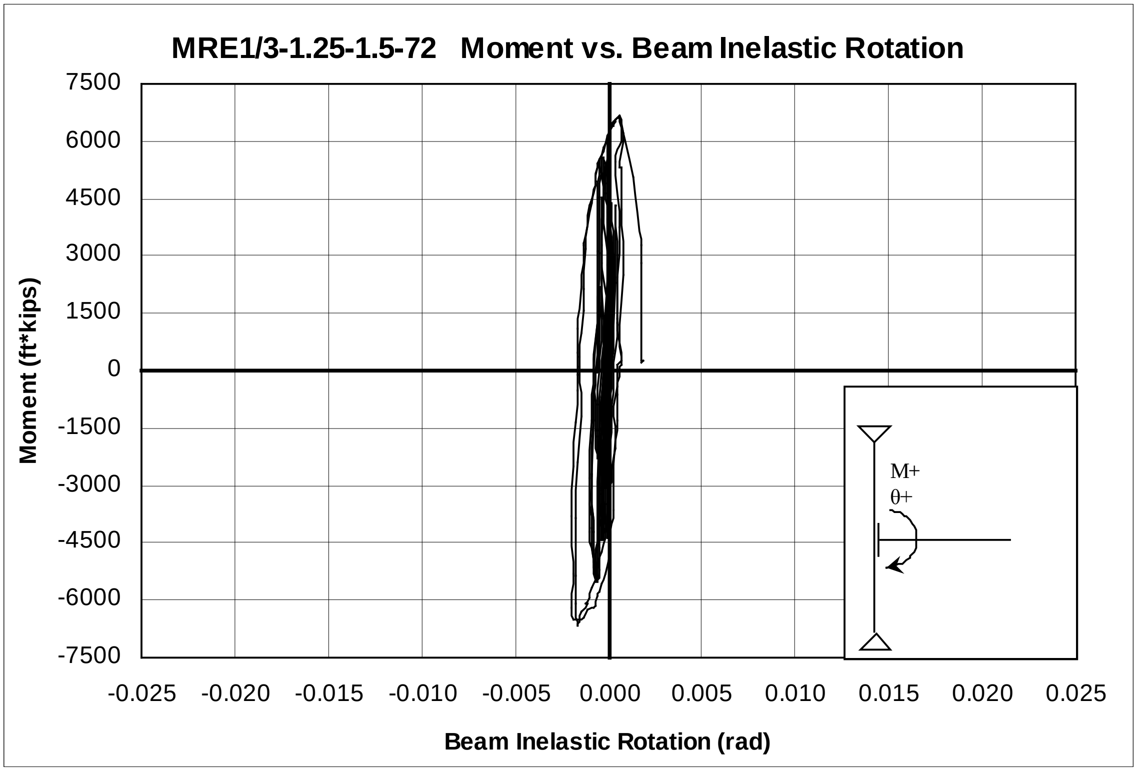


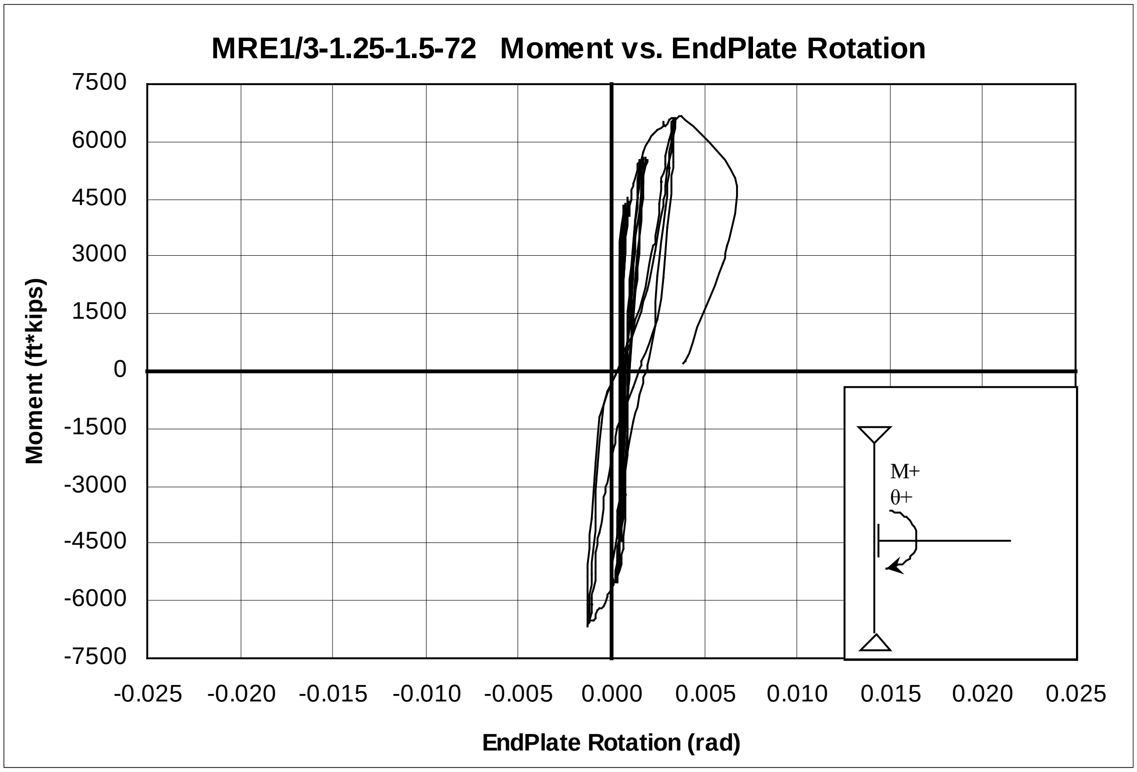
MRE1/3-1.25-1.5-72 Applied Load vs. Displacement
Applied Load (kips)
Displacement (in)
250
200
150
100
50
0
-50
-100
-150
-200
-250
-10
-8
-6
-4
-2
0
2
4
6
8
10
P+
δ+

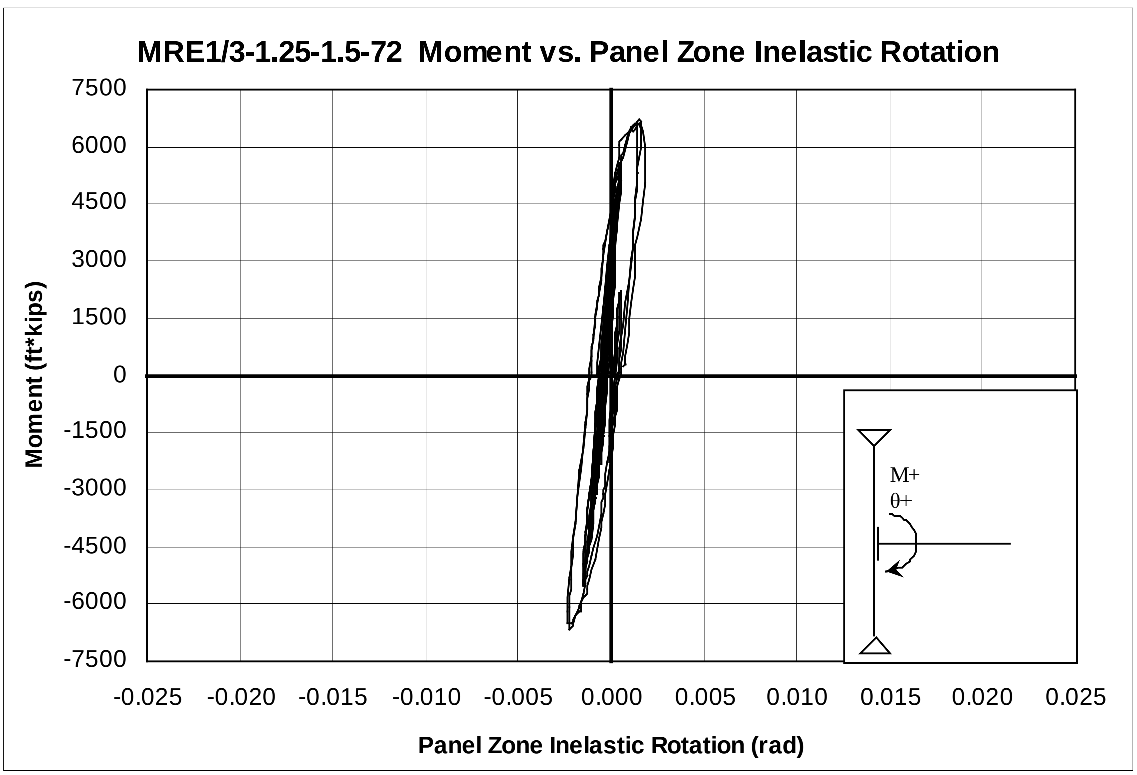


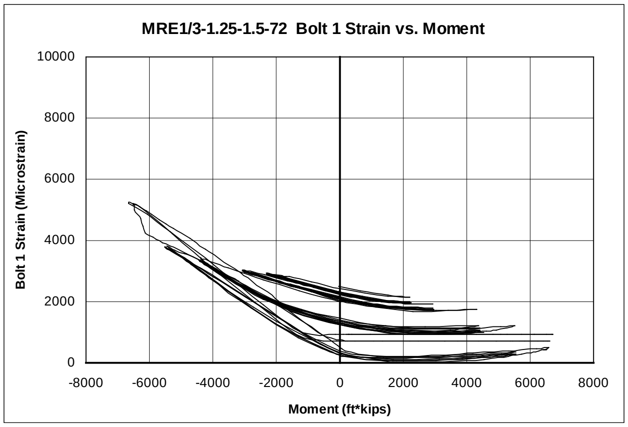
MRE1/3-1.25-1.5-72 Moment vs. Interstory Drift Angle
Moment (ft*kips)
Interstory Drift Angle (rad)
7500
6000
4500
3000
1500
0
-1500
-3000
-4500
-6000
-7500
-0.025
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.010
0.015
0.020
0.025
M+
θ+

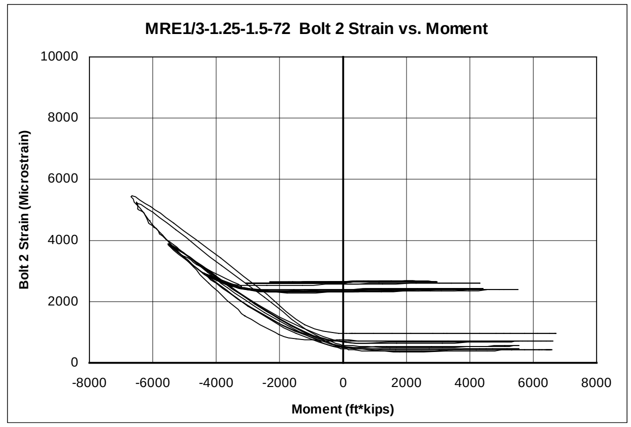

MRE1/3-1.25-1.5-72 Moment vs. Total Inelastic Rotation
Moment (ft*kips)
7500
6000
4500
3000
1500
0
-1500
-3000
-4500
-6000
-7500
-0.025
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.010
0.015
0.020
0.025
Total Inelastic Rotation (rad)
M+
θ+

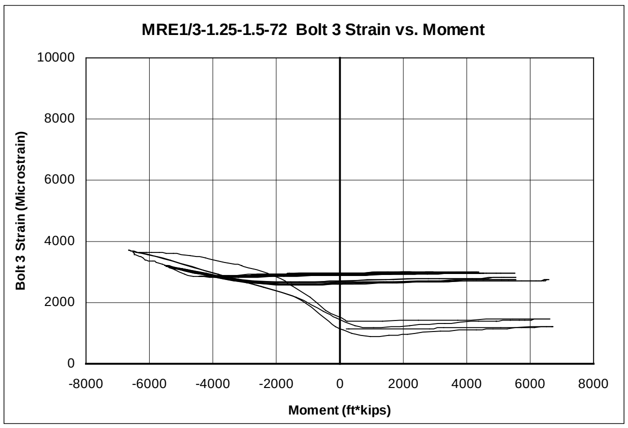

MRE1/3-1.25-1.5-72 Moment vs. Beam Inelastic Rotation
Moment (ft*kips)
7500
6000
4500
3000
1500
0
-1500
-3000
-4500
-6000
-7500
-0.025
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.010
0.015
0.020
0.025
Beam Inelastic Rotation (rad)
M+
θ+

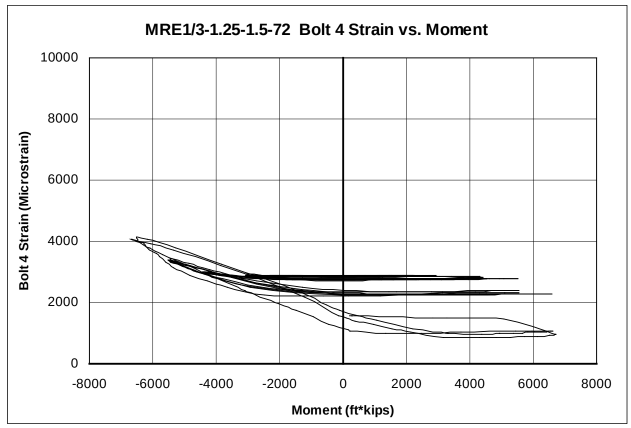

MRE1/3-1.25-1.5-72 Moment vs. EndPlate Rotation
Moment (ft*kips)
7500
6000
4500
3000
1500
0
-1500
-3000
-4500
-6000
-7500
-0.025
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.010
0.015
0.020
0.025
EndPlate Rotation (rad)
M+
θ+

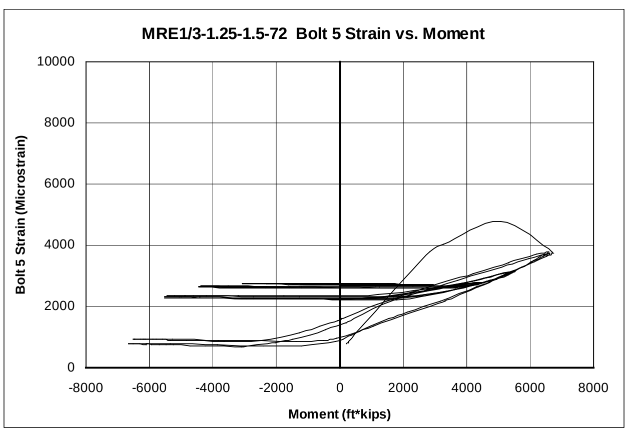

MRE1/3-1.25-1.5-72 Moment vs. Panel Zone Inelastic Rotation
Moment (ft*kips)
7500
6000
4500
3000
1500
0
-1500
-3000
-4500
-6000
-7500
-0.025
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.010
0.015
0.020
0.025
Panel Zone Inelastic Rotation (rad)
M+
θ+

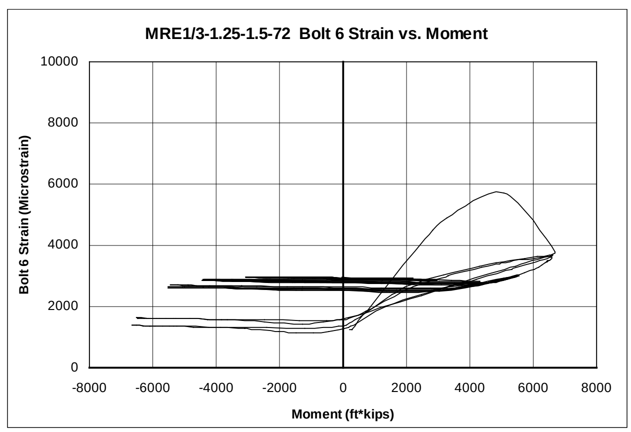

MRE1/3-1.25-1.5-72 Bolt 1 Strain vs. Moment
Bolt 1 Strain (Microstrain)
10000
8000
6000
4000
2000
0
-8000 -6000 -4000 -2000 0 2000 4000 6000 8000
Moment (ft*kips)

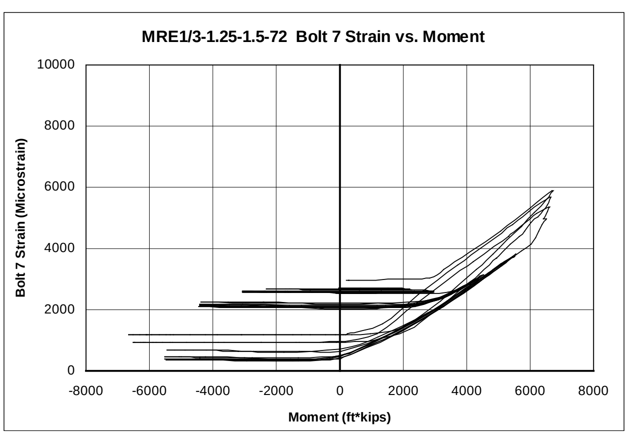

MRE1/3-1.25-1.5-72 Bolt 2 Strain vs. Moment
Bolt 2 Strain (Microstrain)
10000
8000
6000
4000
2000
0
-8000 -6000 -4000 -2000 0 2000 4000 6000 8000
Moment (ft*kips)

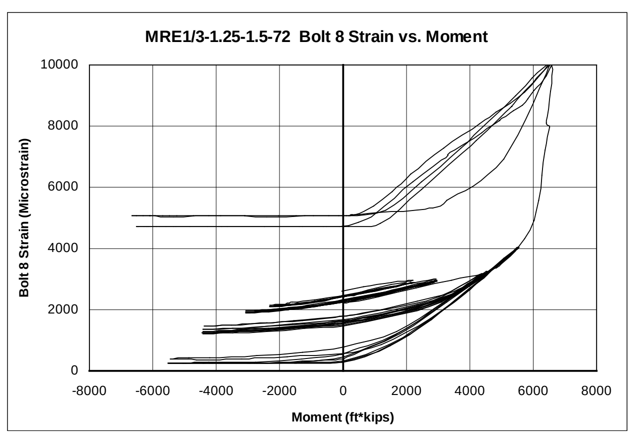

MRE1/3-1.25-1.5-72 Bolt 3 Strain vs. Moment
Bolt 3 Strain (Microstrain)
10000
8000
6000
4000
2000
0
-8000 -6000 -4000 -2000 0 2000 4000 6000 8000
Moment (ft*kips)

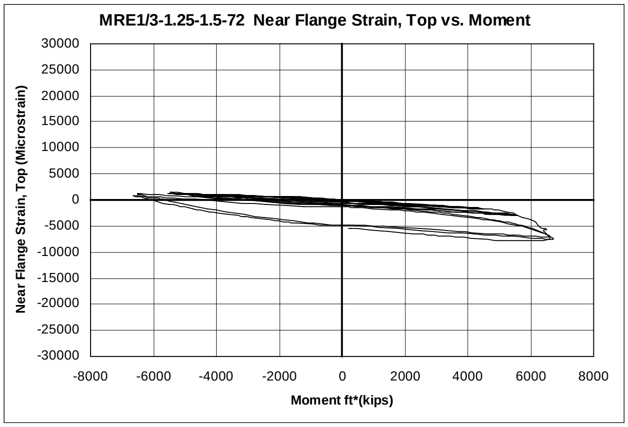

MRE1/3-1.25-1.5-72 Bolt 4 Strain vs. Moment
Bolt 4 Strain (Microstrain)
10000
8000
6000
4000
2000
0
-8000 -6000 -4000 -2000 0 2000 4000 6000 8000
Moment (ft*kips)

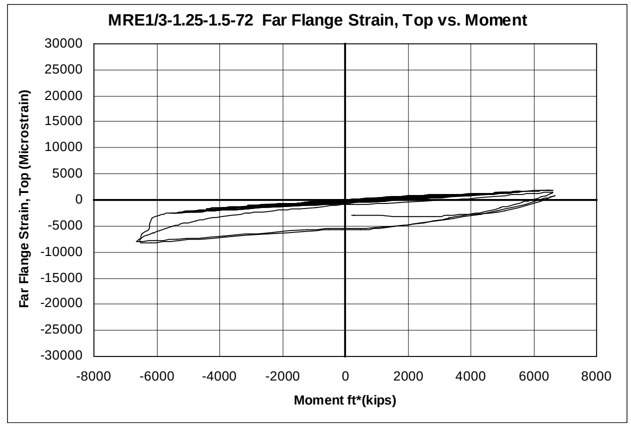

MRE1/3-1.25-1.5-72 Bolt 5 Strain vs. Moment
Bolt 5 Strain (Microstrain)
10000
8000
6000
4000
2000
0
-8000 -6000 -4000 -2000 0 2000 4000 6000 8000
Moment (ft*kips)

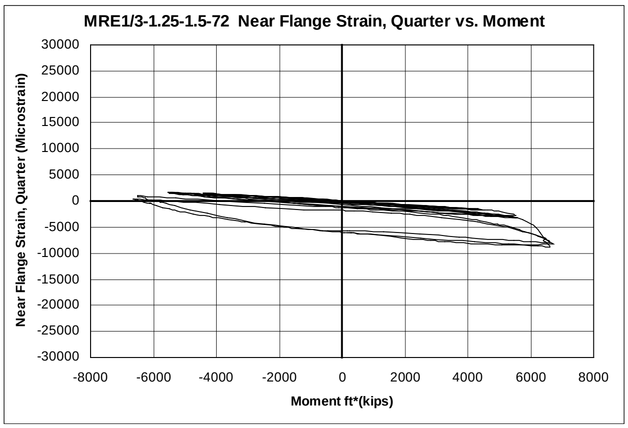

MRE1/3-1.25-1.5-72 Bolt 6 Strain vs. Moment
Bolt 6 Strain (Microstrain)
10000
8000
6000
4000
2000
0
-8000 -6000 -4000 -2000 0 2000 4000 6000 8000
Moment (ft*kips)

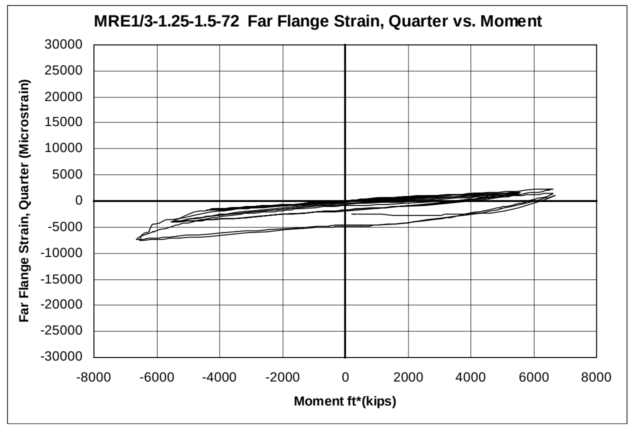

MRE1/3-1.25-1.5-72 Bolt 7 Strain vs. Moment
Bolt 7 Strain (Microstrain)
10000
8000
6000
4000
2000
0
-8000 -6000 -4000 -2000 0 2000 4000 6000 8000
Moment (ft*kips)

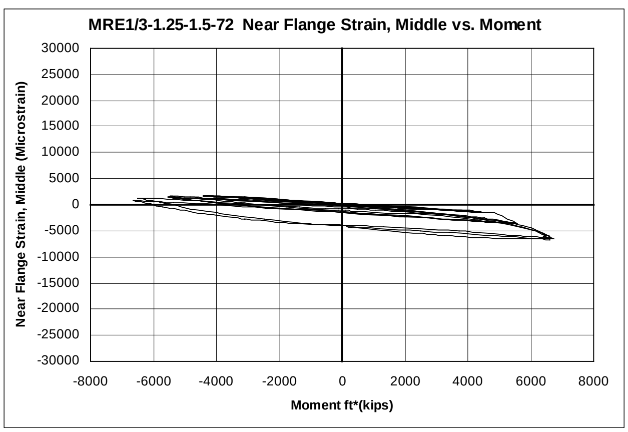

MRE1/3-1.25-1.5-72 Bolt 8 Strain vs. Moment
Bolt 8 Strain (Microstrain)
10000
8000
6000
4000
2000
0
-8000 -6000 -4000 -2000 0 2000 4000 6000 8000
Moment (ft*kips)

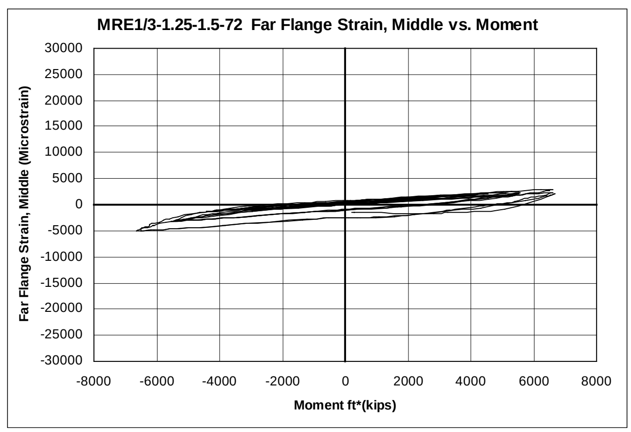

MRE1/3-1.25-1.5-72 Near Flange Strain, Top vs. Moment
Near Flange Strain, Top (Microstrain)
Moment ft*(kips)


MRE1/3-1.25-1.5-72 Far Flange Strain, Top vs. Moment
Far Flange Strain, Top (Microstrain)
Moment ft*(kips)

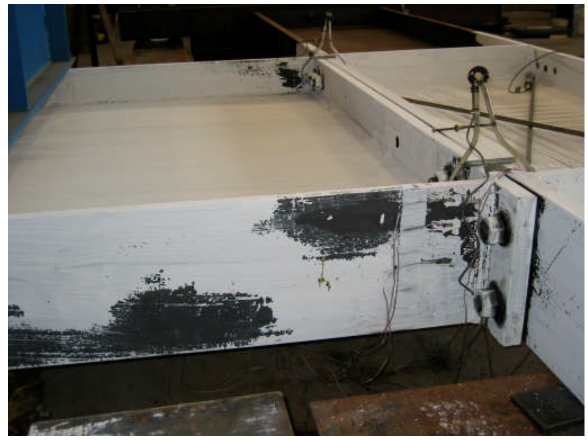

MRE1/3-1.25-1.5-72 Near Flange Strain, Quarter vs. Moment
Near Flange Strain, Quarter (Microstrain)
Moment ft*(kips)


MRE1/3-1.25-1.5-72 Far Flange Strain, Quarter vs. Moment
Far Flange Strain, Quarter (Microstrain)
Moment ft*(kips)

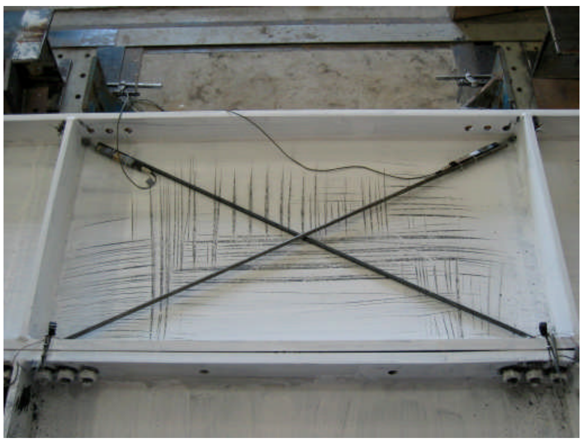

MRE1/3-1.25-1.5-72 Near Flange Strain, Middle vs. Moment
Near Flange Strain, Middle (Microstrain)
Moment ft*(kips)


MRE1/3-1.25-1.5-72 Far Flange Strain, Middle vs. Moment
Far Flange Strain, Middle (Microstrain)
Moment ft*(kips)

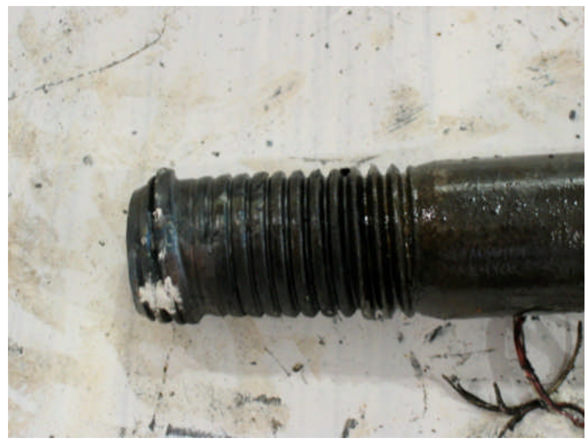
Near Flange



Far Flange



Panel Zone



Close-Up of Stripped Threads of Bolt from Far Flange

MRE1/3-1.25-1.5-72 AFTER TESTING

VITA

Stephen E. Blumenbaum was born on February 3, 1977 in Narragansett, Rhode Island. In December 1999, he received a Bachelor of Science Degree in Civil Engineering from Virginia Polytechnic Institute and State University. He then worked as a design engineer for Alpha Corporation in Dulles, Virginia, focusing on the design of temporary structures and excavation support systems. In January 2003, he returned to Virginia Polytechnic Institute and State University to pursue a Master of Science in Civil Engineering. He will begin working for Walter P. Moore in Tampa, Florida in Fall 2004.