# Four-Bolt Unstiffened End-Plate Moment Connections with 36-in.-Deep Beams for Intermediate Moment Frames

Maria A. Mercado-Celin, Matthew R. Eatherton, and Thomas M. Murray


## ABSTRACT

Previous testing has shown that four-bolt extended, unstiffened end-plate moment connections can be used with beams up to 24 in. deep and develop sufficiently ductile performance to satisfy seismic qualification. It is desirable to extend this depth limit to allow deeper beams for intermediate moment frames. Three moment connection specimens using built-up 36-in.-deep beams with a four-bolt extended, unstiffened end-plate moment connection were tested to determine if they satisfy the intermediate moment frame qualification criteria given in AISC 341-16 (2016a). The web of the specimens satisfied the moderately ductile section criteria for web slenderness, which is required for intermediate moment frames, but not highly ductile section criteria required for special moment frames.

All three specimens passed qualification criteria for intermediate moment frames (IMFs) as given in AISC 341-16 by retaining at least 80% of the nominal plastic moment strength through 2% story drift. The observed failure modes included lateral torsional buckling of the specimen, flange local buckling, net section fracture of the beam flange, and failure of the beam flange-to-beam web fillet weld. The results of these tests support current moderately ductile limits associated with lateral bracing and cross-section slenderness, given that the associated specimens reached IMF qualification criteria but not special moment frame (SMF) criteria. It was concluded that four-bolt extended, unstiffened end-plate moment connections satisfy intermediate moment frame requirements with a beam depth of 36 in. and that a modification is needed for beam web to flange welds in built-up sections near the moment connection.



**Keywords:** end-plate moment connection, intermediate moment frame, lateral-torsional buckling, beam net section fracture, moderately ductile section.


## INTRODUCTION

Previous cyclic testing on four-bolt extended, unstiffened (4E) end-plate moment connections with beams deeper than 24 in. have failed to satisfy the intermediate or special moment frame (IMF or SMF) qualification criteria given in the 2016 edition of the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2016a), hereafter referred to as AISC 341-16. Ryan and Murray (1999) tested one 4E specimen with a 55-in.-deep beam that resulted in bolt fracture at 0.01 rad rotation, and Blumenbaum and Murray (2004) tested two 4E specimens with 60-in.-deep rafters that resulted in bolt fracture and out-of-plane buckling at 0.01 rad to 0.02 rad. of inelastic rotation. Conversely, tests by Sumner et. al. (2000) and Meng and Murray (1996) reported successful SMF qualification tests of 4E moment connection specimens with W24×68 and W24×76 sections.

To facilitate longer spans, it is desirable to allow the use of 4E connections in IMF or SMF with beams deeper than 24 in. For that reason, a testing program was conducted to determine the required connection detailing such that the connection has sufficient ductility to achieve IMF qualification criteria with 36-in.-deep beams. The testing program utilized specimens with beam webs satisfying moderately ductile, but not highly ductile criteria and beam lateral bracing that satisfied either IMF or SMF criteria. In addition to evaluating IMF qualification, this study also represents a unique opportunity to investigate the distinction between IMF and SMF in terms of inelastic lateral torsional buckling and local buckling behavior.

## EXPERIMENTAL PROGRAM

### Test Specimens

The specimens were designed according to the procedures in the 2022 AISC *Prequalified Connections for Special and Intermediate Moment Frames for Seismic Applications* (AISC, 2022), hereafter referred to as AISC 358-22. The test beams were built-up sections with ⅝ in. × 8 in. flanges and ½-in.-thick webs. The flanges satisfy highly ductile


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| Table 1. Test Matrix | | | | | |
|---|---|---|---|---|---|
| Specimen Identification | Beam Depth, *d* (in.) | Bolts | End-Plate Thickness (in.) | Double Fillet Weld Length (in.)[1] | Lines of Lateral Bracing |
| 4E-1.5-1.75-36a | 36 | 1½ in. A490 | 1¾ | 24 | 2 |
| 4E-1.5-1.75-36b | 36 | 1½ in. A490 | 1¾ | 24 | 3 |
| 4E-1.5-1.75-36c | 36 | 1½ in. A490 | 1¾ | 48 | 3 |

[1] Length of double-sided fillet weld at the beam web-to-beam flange, measured from the outside face of the end-plate (see dimension L3 in Figure 1).

| Table 2. Specimen Thicknesses (all units are inches) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Specimen | Web Thickness | | Lower-Flange Thickness | | Upper-Flange Thickness | | End-Plate Thickness | |
| | Nominal | Measured | Nominal | Measured | Nominal | Measured | Nominal | Measured |
| 4E-1.5-1.75-36a | 0.50 | — | 0.625 | 0.628 | 0.625 | 0.624 | 1.75 | 1.781 |
| 4E-1.5-1.75-36b | 0.50 | 0.535 | 0.625 | 0.64 | 0.625 | 0.625 | 1.75 | 1.765 |
| 4E-1.5-1.75-36c | 0.50 | 0.535 | 0.625 | 0.625 | 0.625 | 0.64 | 1.75 | 1.765 |

| Table 3. Specimen Dimensions | | | |
|---|---|---|---|
| Specimen | Length, L1 | Length, L2 | Length, L3 |
| 4E-1.5-1.75-36a | 20'-2⅞" | 5'-0" | 24" |
| 4E-1.5-1.75-36b | 23'-8⅞" | 8'-6" | 24" |
| 4E-1.5-1.75-36c | 23'-8⅞" | 8'-6" | 48" |

| Table 4. Material Properties | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Specimen | Element | Material Specification | From Mill Report | | | Measured Properties | | |
| | | | $F_y$ (ksi) | $F_u$ (ksi) | Elongation | $F_y$ (ksi) | $F_u$ (ksi) | Elongation |
| 4E-1.5-1.75-36a | Beam flange | A529 Gr. 55 | 55.4 | 76.6 | 25% | 53.7 | 74.1 | 41% |
| 4E-1.5-1.75-36a | Beam web | HSLA55 | 64.8 | 78.5 | 32% | — | — | — |
| 4E-1.5-1.75-36a | End plate | A572 Gr. 50 | 59.0 | 82 | 38% | — | — | — |
| 4E-1.5-1.75-36b,c | Beam flange | A529 Gr. 55 | 56.6 | 77.7 | 23% | 53.1 | 77.3 | 38% |
| 4E-1.5-1.75-36b,c | Beam web | HSLA55 | 64.9 | 76.1 | 34% | 59.7 | 72.9 | 24% |
| 4E-1.5-1.75-36b,c | End plate | A572 Gr. 50 | 55.2 | 81.4 | 25% | — | — | — |

section criteria, per AISC 341-16 (AISC, 2016a), and the webs satisfy moderately ductile section criteria. The detailing of the moment connection satisfied requirements of AISC 358-22. Table 1 is the text matrix, Figure 1 shows beam and end-plate details, and Tables 2 and 3 give specific dimensions. The built-up beams were fabricated using A572/A572M (ASTM, 2021) Grade 55 steel for the flanges, HSLA55 steel for the webs, and A572 Grade 50 for the end plates with material properties given in Table 4.

The specimen identification is given with the bolt configuration ("4E" for four-bolt extended, unstiffened), bolt diameter ["1.5" for the 1.5 in. ASTM F3125/3125M (ASTM, 2022) Grade A490 bolts], end-plate thickness (1.75 in.), beam depth (36 in.), and then a letter to distinguish among the three specimens. The specimens are therefore labeled, 4E-1.5-1.75-36a, 4E-1.5-1.75-36b, and 4E-1.5-1.75-36c.

In accordance with AISC 358-22, Section 6.3.1(1), the beam web-to-beam flange weld was specified to be ⅜ in. double-sided fillet with a minimum required length of 24 in., which was used for Specimens 4E-1.5-1.75-36a and 4E-1.5-1.75-36b. The length of double-sided fillet weld was increased to 48 in. for Specimen 4E-1.5-1.75-36c.

To make efficient use of the steel beams, both ends of each beam had end plates attached so that two unique specimens could be created with one beam. A set of four holes in the beam flange were included in all specimens, centered at a distance, L2, from the outside face of the end plate (see Figure 1 and Table 3). The holes in the beam flange facilitate connection to the actuator when testing the far side end-plate connection. The distance from outside face of end plate to centerline of the bolt holes, L2, was extended for Specimens 4E-1.5-1.75-36b and 4E-1.5-1.75-36c.

### Test Setup

Three tests were conducted in accordance with AISC 341-16, Chapter K, at the Thomas M. Murray Structural Engineering Laboratory at Virginia Tech. A schematic of the test setup is shown in Figure 2, and a photograph is in Figure 3. The setup used in this study simulated an exterior moment connection in a structure with 32-ft-wide bays and a 12 ft story height. The reaction column was a W14×398. Force was applied by an MTS 201.70 servo hydraulic actuator with a ±10 in. stroke and a tension capacity of 220 kips.

Lateral bracing for specimen 4E-1.5-1.75-36a was provided at the end of the plastic hinge and near the point of loading as shown in Figure 2. This lateral bracing satisfies IMF requirements in AISC 341-16, Section D1.2a.1, but not SMF requirements in Section D1.2b. An additional line of lateral bracing was added for specimen 4E-1.5-1.75-36b and specimen 4E-1.5-1.75-36c (see Figure 3), to meet the unbraced length required for SMF. More details are provided in the section on lateral torsional buckling of Specimen 4E-1.5-1.75-36a.

Lateral bracing consisted of steel frames with adjustable steel angles that were moved until the face of the angle was approximately 1⁄16 in. away from the flange tips. Lithium grease was applied to the face of the angles to reduce friction during the test. An example of the lateral bracing is shown in Figure 4.

### Instrumentation Plan

The instrumentation plan is shown in Figure 5. String potentiometers SP_01, SP_02, SP_03, and SP_08 were used to measure column movement. String potentiometers SP_04 and SP_05 measured beam vertical movement. Sensors SP_06 and SP_07 measured axial deformation of the plastic hinge zone, while SP_09 measured vertical end-plate movement with respect to the column. Two instrumented spring calipers, CLP_01 and CLP_02, were used to measure the end-plate separation from the column flange. Linear potentiometers LP_01 and LP_02 measured panel-zone shear deformations. The applied force and displacement were

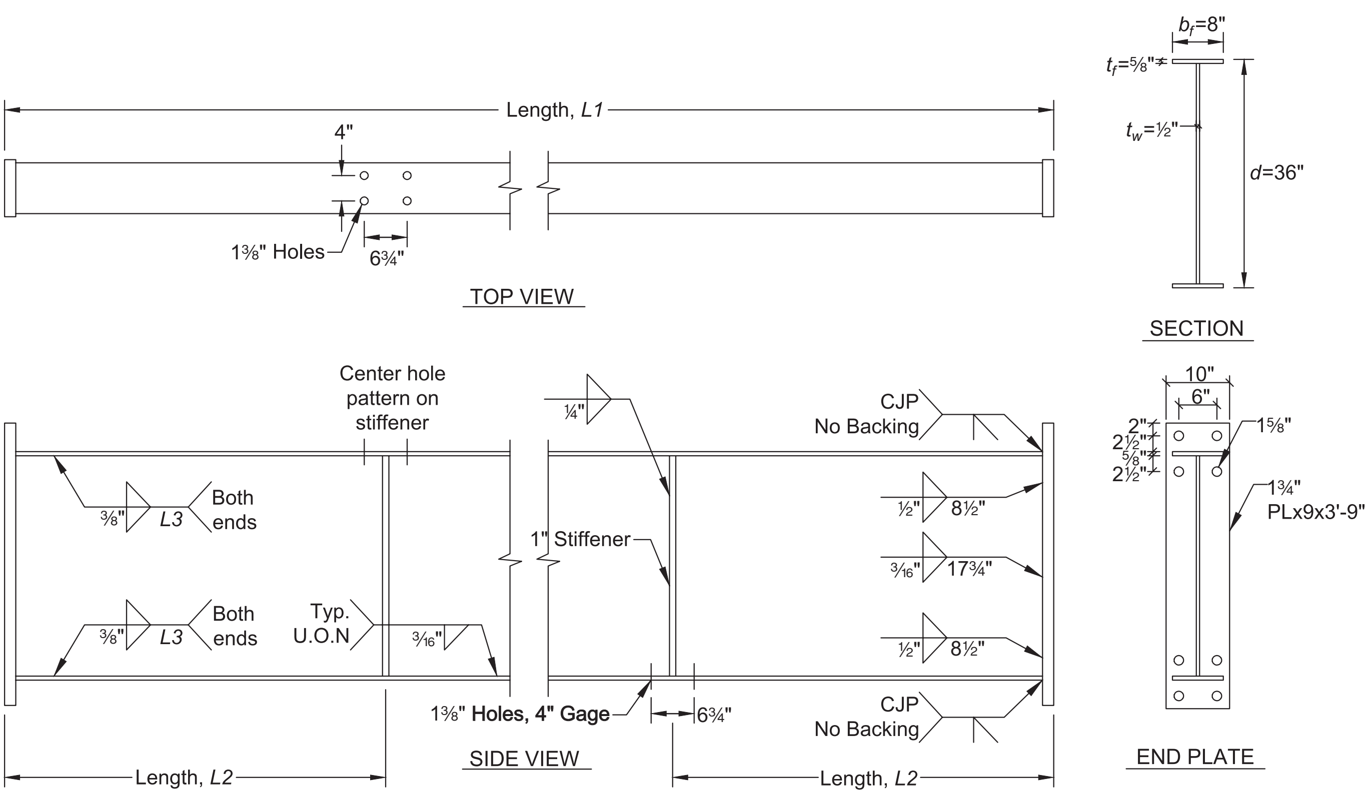



*Fig. 1. Details of the beam.*

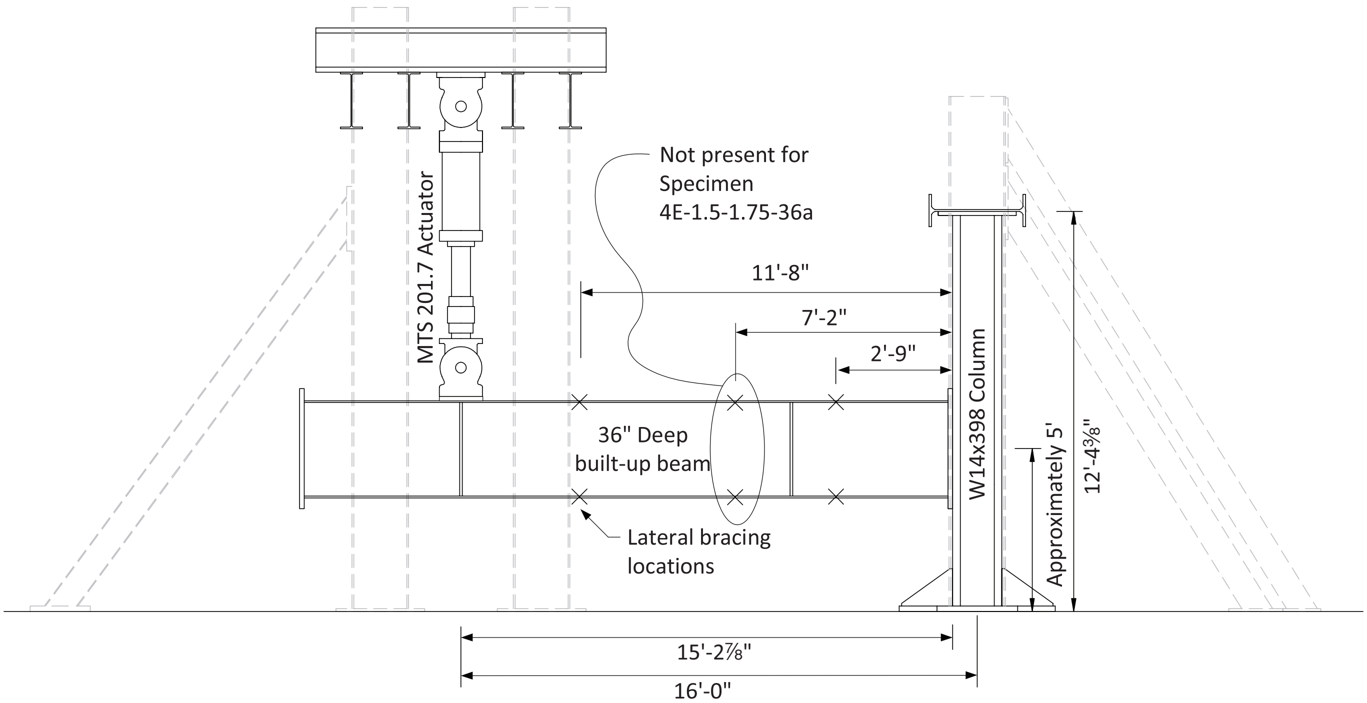



*Fig. 2. Schematic of test setup.*

*Fig. 3. Photograph of the test setup.*


*Fig. 4. Beam lateral bracing.*

| Table 5. AISC 341-16 Displacement Protocol for Moment Connection Tests | | |
|---|---|---|
| Story Drift Target | | Number of Cycles |
| Radians | Percent | |
| 0.00375 | 0.375 | 6 |
| 0.005 | 0.5 | 6 |
| 0.0075 | 0.75 | 6 |
| 0.01 | 1 | 4 |
| 0.015 | 1.5 | 2 |
| 0.02 | 2 | 2 |
| 0.03 | 3 | 2 |
| 0.04 | 4 | 2 |
| 0.045 | 4.5 | Until fracture |

measured by the actuator's internal load cell and displacement transducer. All sensors were connected to a National Instruments data acquisition system, which was managed using National Instruments Signal Express software (NI, 2015). Measurements from all sensors were recorded at 3 Hz.

### Displacement Protocol and Qualification Criteria for IMF and SMF

The cyclic displacement protocol from AISC 341-16, Chapter K, for qualification testing of IMF and SMF moment connections was used and is given in Table 5. The maximum displacement that could be applied corresponded to

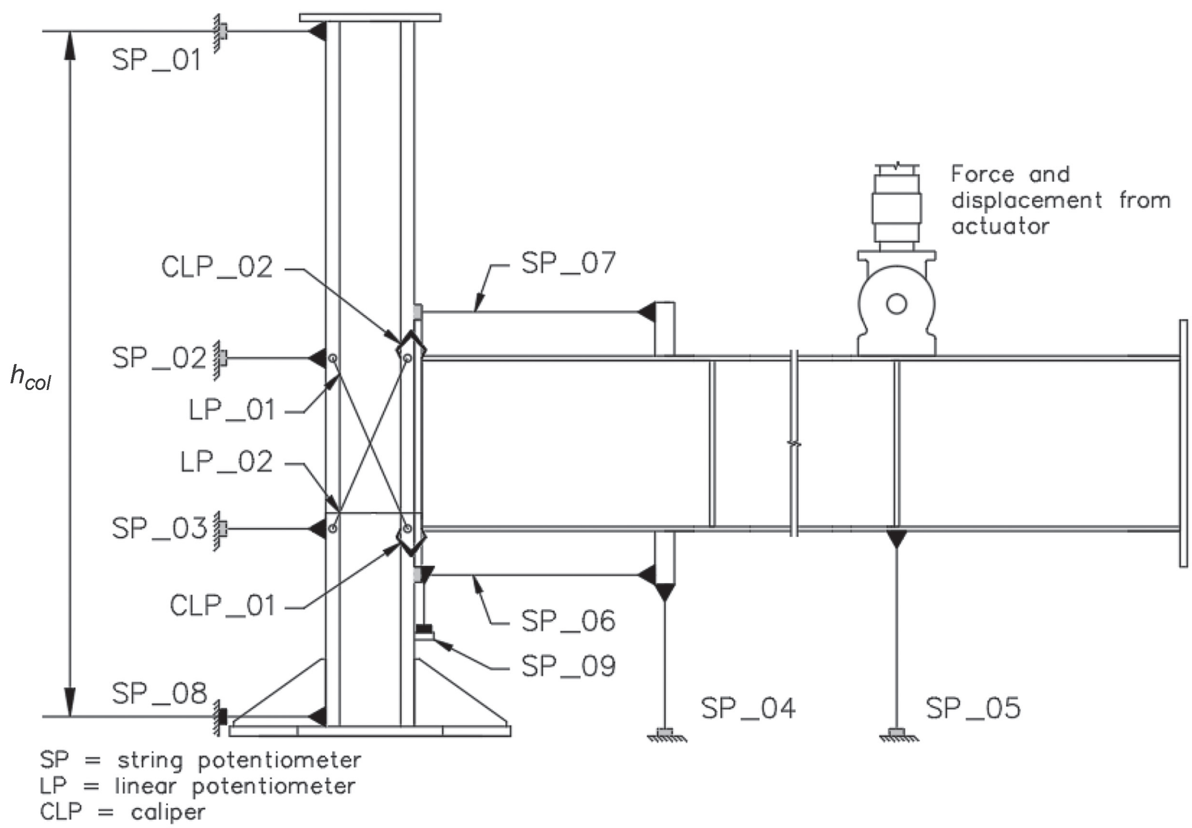



*Fig. 5. Instrumentation plan for tests.*

4.5% story drift based on the location and maximum stroke of the actuator. The displacement rate was 0.00025 rad/s, which corresponds to a vertical displacement at the actuator of approximately 2.88 in./min.

The story drift was calculated real-time within the MTS control software (MTS, 2012) using external feedback, and then used to control the actuator displacement to produce the story drifts given in Table 5. The applied story drift, $\theta_{APP}$, was calculated in the MTS MultiPurpose Testware (MTS, 2011) calculations module using Equation 1.

$$\theta_{APP} = \frac{-\delta_{SP\text{-}05}}{L_{cl}} - \frac{\delta_{SP\text{-}01} - \delta_{SP\text{-}08}}{h_{col}} \tag{1}$$

where

$L_{cl}$ = distance from the actuator centerline to column centerline, 192 in.

$h_{col}$ = distance between SP_01 and SP_08, 137 in.

$\delta_{SP\text{-}01}$ = displacement measured by SP_01, in.

$\delta_{SP\text{-}05}$ = displacement measured by SP_05, in.

$\delta_{SP\text{-}08}$ = displacement measured by SP_08, in.

The qualification criteria for intermediate moment frame connections in AISC 341-16 states that the connection must maintain a moment strength at the face of the column of at least 80% of the nominal beam plastic moment strength through the first cycle of 2% story drift. For special moment frame connections, the requirement is similar but is evaluated at the first cycle of 4% story drift. The moment at the face of the column was calculated as the applied load multiplied by the distance from the actuator to the face of the column, 15.23 ft. Based on the beam cross section shown in Figure 1 with plastic section modulus, $Z_x$ = 328 in.$^3$, and a nominal yield stress of $F_y$ = 55 ksi, the nominal plastic moment strength of the beam is $M_p$ = 1,500 kip-ft.

## RESULTS

### IMF Qualification

All three specimens passed qualification criteria for intermediate moment frames (IMFs) as given in AISC 341-16 by retaining at least 80% of the nominal plastic moment strength through 2% story drift. Table 6 gives the moment strength for each specimen at the positive and negative peak during the first cycle at 2% story drift, values significantly larger than $0.8M_p$ = 1,200 kip-ft. The three specimens had almost identical behavior through the 2% story drift cycles, as shown in Figure 6, with no observable flange local buckling or strength degradation while the flanges yielded. The measured initial stiffness of the specimens was 158,000 kip-ft/rad that, after removing the elastic beam stiffness of 193,000 kip-ft/rad, results in a connection stiffness of 863,000 kip-ft/rad. The minimum connection stiffness to be considered a fully restrained connection per the AISC *Specification for Structural Steel Buildings* (AISC, 2016b), hereafter referred to as AISC 360-16, Section B3.4 Commentary, is $20EI/L$ = 644,000 kip-ft/rad using a beam span equal to 32 ft less the column depth. Since the measured connection stiffness is greater than this minimum, the connection is considered fully restrained.

Specimen 4E-1.5-1.75-36c survived to the 4% story drift cycles, but as shown in Table 6 and due to local buckling of the beam section, the moment strength dropped to approximately 95% of the required moment strength to reach special moment frame (SMF) qualification (1,150 kip-ft compared to the required 1,200 kip-ft). The end-plate separation remained small for all tests with a maximum value of approximately 0.12 in. for one specimen and was less than 0.05 in. for the other two specimens. There was no observed yielding or damage to the end-plates or bolts during any of the tests. This suggests that the design procedures in AISC 358 for the end plate and bolts were sufficient for this connection.

### Specimen Response

The first two specimens experienced several undesirable limit states that were progressively mitigated until the third specimen for which all these limit states were prevented. As shown in Table 7, Specimens 4E-1.5-1.75-36a and 4E-1.5-1.75-36b survived to the 3% story drift cycles, while Specimen 4E-1.5-1.75-36c exhibited excellent fracture resistance and ductility surviving multiple cycles at 4.5% story drift before the test was stopped. The individual limit states for Specimens 4E-1.5-1.75-36a and 4E-1.5-1.75-36b are discussed in the subsequent paragraphs. Figure 7 shows the final condition of each specimen. Greater deformations were achieved as the specimens and test setup were retrofitted based on the failure modes observed.

#### *Beam Net Section Fracture of Specimen 4E-1.5-1.75-36a (Not Related to Qualification)*

Specimen 4E-1.5-1.75-36a experienced a net section fracture of the beam at the location of a set of holes approximately 4.75 ft from the face of the column, as shown in Figure 8. AISC 360-16, Section F13 (AISC, 2016b), provides a way to check for such a limit state based on a ratio of the flange net area to the flange gross area. The required moment strength at the location of the bolt holes associated with the beam developing the maximum probable moment strength, $M_{pr}$, at the column face is calculated as, $M_u$ = 1,230 kip-ft. The nominal moment strength calculated in accordance with AISC 360-16, Section F13, is found to be $M_n$ = 1,040 kip-ft, which is less than the required moment strength, $M_u$, and therefore suggests that the beam flange is susceptible to beam net section fracture. It is noted that

| Table 6. Summary of Qualification Status for Each Specimen | | | | |
|---|---|---|---|---|
| Specimen | First Cycle at Story Drift of | Moment at Positive Peak[1] | Moment at Negative Peak[1] | Qualification[2] |
| 4E-1.5-1.75-36a | 2% | 1660 kip-ft | 1620 kip-ft | IMF |
| 4E-1.5-1.75-36b | 2% | 1650 kip-ft | 1760 kip-ft | IMF |
| 4E-1.5-1.75-36c | 2% | 1710 kip-ft | 1710 kip-ft | IMF |
| | 4% | 1210 kip-ft | 1150 kip-ft | |

[1] Moment calculated at the column face
[2] Qualification is reached if moment strength is greater than $0.8M_p$ = 1200 kip-ft

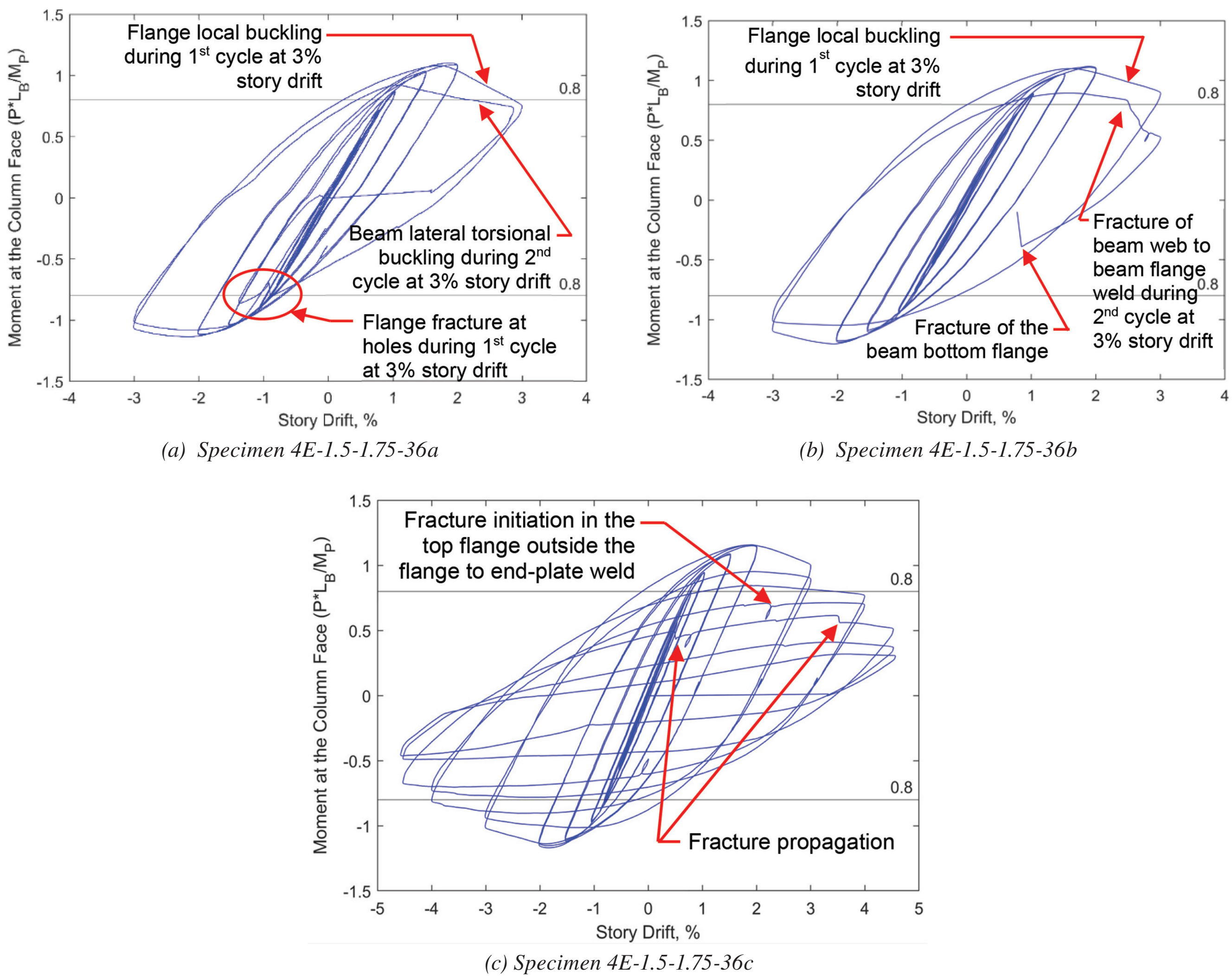
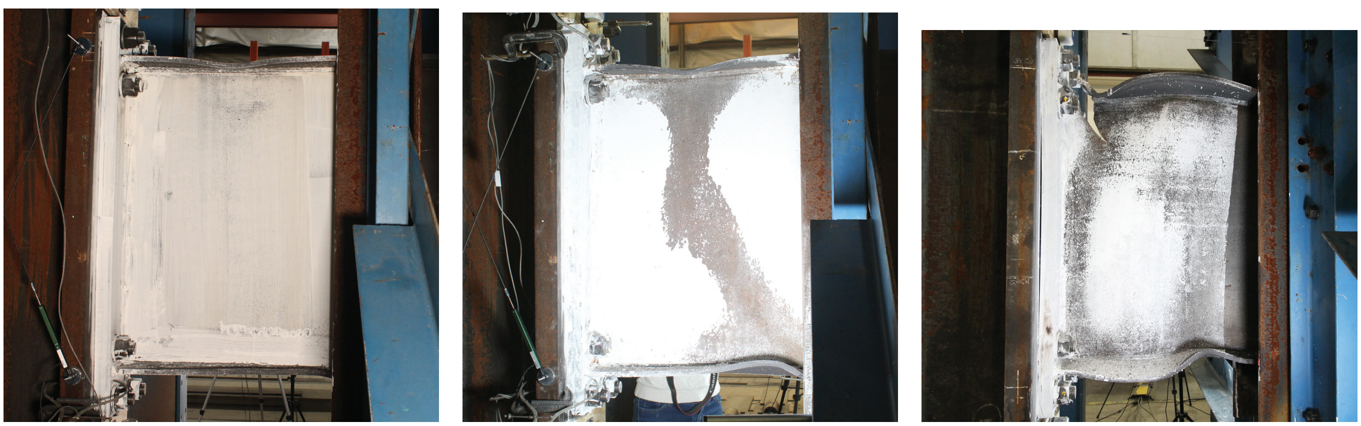



*Fig. 6. Hysteretic behavior of the three specimens.*

| Table 7. Summary of Failure Mode for Each Specimen | | |
|---|---|---|
| **Specimen** | **Peak Story Drift Attained** | **Failure Mode** |
| 4E-1.5-1.75-36a | 2 cycles at 3% | Lateral torsional buckling |
| 4E-1.5-1.75-36b | 2 cycles at 3% | Fracture of beam web-to-beam flange weld, then flange fracture |
| 4E-1.5-1.75-36c | 3 cycles at 4.5% | Flange local buckling, fracture of the beam flange outside the CJP weld at the end-plate |

at the time of fracture, the moment at the face of the column was approximately 65% of $M_{pr}$, which was used in the calculation, but during the previous cycle, the moment at the column face was close to $M_{pr}$. It is possible that net section fracture initiated during the previous cycle but was not noticed. The specimen was repaired by welding a flange plate to the outside of the bottom flange, as shown in Figure 9, in an effort to continue the test.

For Specimens 4E-1.5-1.75-36b and 4E-1.5-1.75-36c, the distance from the column face to the first set of bolt holes was increased to approximately 8.25 ft, which was selected so that the required moment strength, $M_u$ = 750 kip-ft was



*(a) Specimen 4E-1.5-1.75-36a* *(b) Specimen 4E-1.5-1.75-36b* *(c) Specimen 4E-1.5-1.75-36c*

*Fig. 7. Final state at the connection of each specimen.*

*Fig. 8. Bottom flange fracture at bolt holes of Specimen 4E-1.5-1.75-36a.*

*Fig. 9. Bottom flange repair with welded plate of Specimen 4E-1.5-1.75-36a.*

less than the design moment, $\phi M_n$ = 778 kip-ft. Neither of these specimens experienced net section fracture of the beam, suggesting that the requirements of AISC 360, Section F13, were effective in addressing this limit state.

### *Lateral Torsional Buckling of Specimen 4E-1.5-1.75-36a*

After the bottom flange repair, Specimen 4E-1.5-1.75-36a was further tested and underwent lateral torsional buckling in the unbraced length equal to 107 in, as shown in Figure 10. According to AISC 341-16, Section D1.2, the maximum unbraced length for moderately ductile, $L_{b,Mod}$, and highly ductile, $L_{b,High}$, are given by Equations 2 and 3.

$$L_{b,Mod} = \frac{0.19 r_y E}{R_y F_y} \tag{2}$$
$$= \frac{0.19(1.40\text{ in.})(29{,}000\text{ ksi})}{(1.1)(55\text{ ksi})}$$
$$= 128\text{ in.}$$

$$L_{b,High} = \frac{0.095 r_y E}{R_y F_y} \tag{3}$$
$$= \frac{0.095(1.40\text{ in.})(29{,}000\text{ ksi})}{(1.1)(55\text{ ksi})}$$
$$= 64\text{ in.}$$

where

$E$ = modulus of elasticity, ksi

$F_y$ = nominal yield stress, ksi

$R_y$ = ratio of the expected yield stress to the specified minimum yield stress

$r_y$ = radius of gyration about the $y$-axis, in.

The provided unbraced length for Specimen 4E-1.5-1.75-36a was 107 in., meeting the moderately ductile section requirements for IMF but not the highly ductile section requirements for SMF. This is consistent with the results obtained where Specimen 4E-1.5-1.75-36a experienced lateral torsional buckling after passing 2% story drift cycles (IMF) but before reaching 4% story drift (SMF).

The unbraced length for Specimen 4E-1.5-1.75-36c, was 54 in., which satisfied the highly ductile section and thus SMF design requirements. This is also consistent with the resulting behavior in that the specimen reached and exceeded 4% story drift without experiencing lateral torsional buckling.

### *Buckling and Fracture Behavior of Specimen 4E-1.5-1.75-36b*

Specimen 4E-1.5-1.75-36b underwent beam flange and web local buckling, shown in Figures 11 and 12, with associated reduction in moment strength. In addition, Figure 13 shows the weld fracture at the beam web to the beam flange joint where the weld transitioned from a ⅜ in. double-sided fillet weld to a ³⁄₁₆ in. single-sided fillet weld. The requirement for the double-sided fillet weld comes from AISC 358-22, Section 6.3.1(1), which requires specific welds at moment-connected ends of welded built-up sections, within at least the depth of beam or three times the width of flange, whichever is less. In this region, the beam web and flanges shall be connected using either a complete-joint-penetration (CJP) groove weld or a pair of fillet welds, each having a size 75% of the beam web thickness but not less than ¼ in. For the specimens in this study, the requirement results in a ⅜ in. double-sided fillet weld (or CJP weld) that extends for 24 in. Beyond this region, the beam web to flange weld is

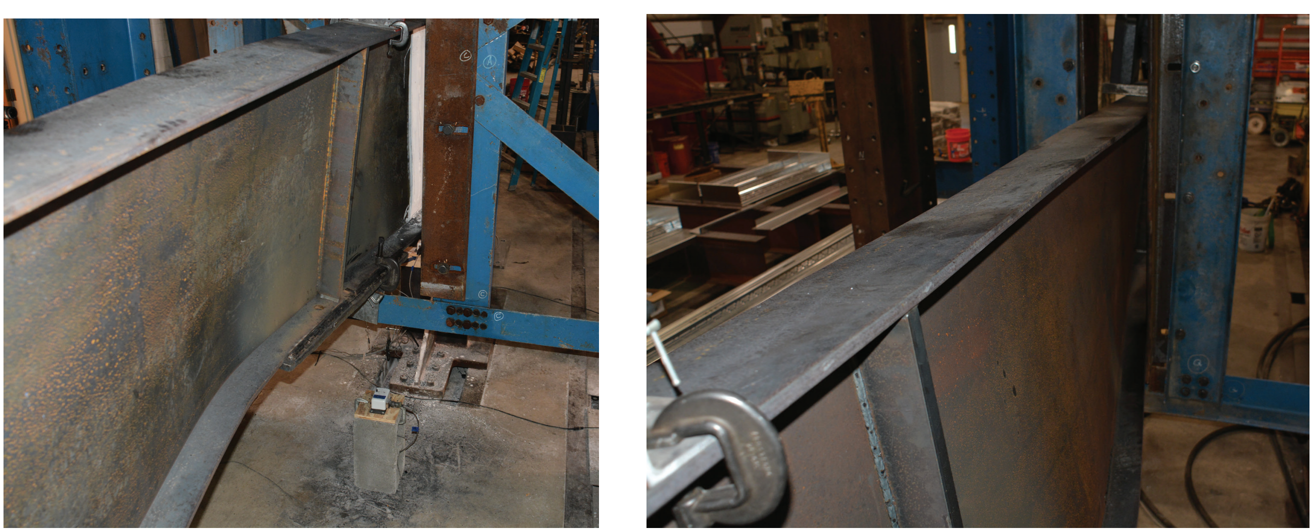


*Fig. 10. Lateral torsional buckling of the beam of Specimen 4E-1.5-1.75-36a.*

designed for shear transfer that, for these specimens, allows the use of a 3⁄16 in. single-sided fillet weld. This type of single-sided weld is common for the web to flange joint in built-up beams typically used in metal buildings.

Previously tested 4E specimens that satisfied SMF requirements used sections satisfying highly ductile section criteria (Meng and Murray, 1996) and generally resulted in local buckling confined to a length of the beam that was the lesser of the depth of the beam or three times the flange width. It is also noted that other previous end-plate moment connection tests (configurations other than 4E) with built-up beam sections satisfying highly ductile section criteria did not experience this type of beam web to flange weld fracture (Szabo et al., 2017; Zarat-Basir et al., 2020). Specimen 4E-1.5-1.75-36b in the current test series had a beam web that satisfied moderately ductile section criteria, but not highly ductile section criteria, and experienced local buckling that extended over a length of approximately 4 ft (Figure 12). Previous tests with built-up beam sections satisfying moderately ductile but not highly ductile section

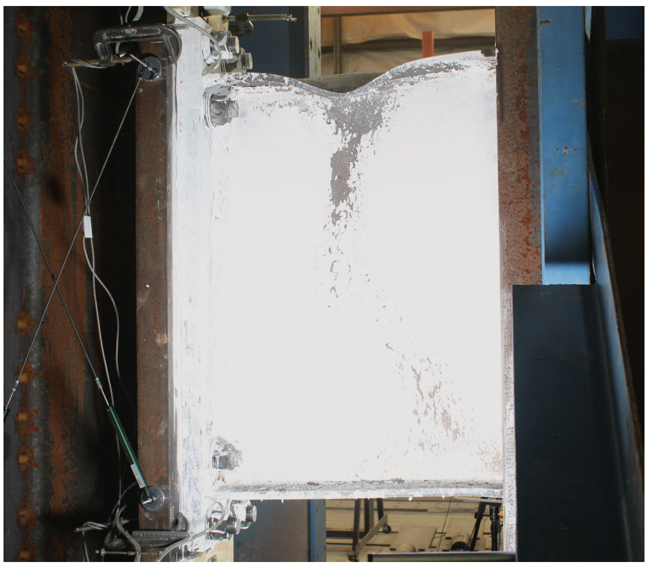


*Fig. 11. Flange local buckling during the first cycle at 3% story drift.*

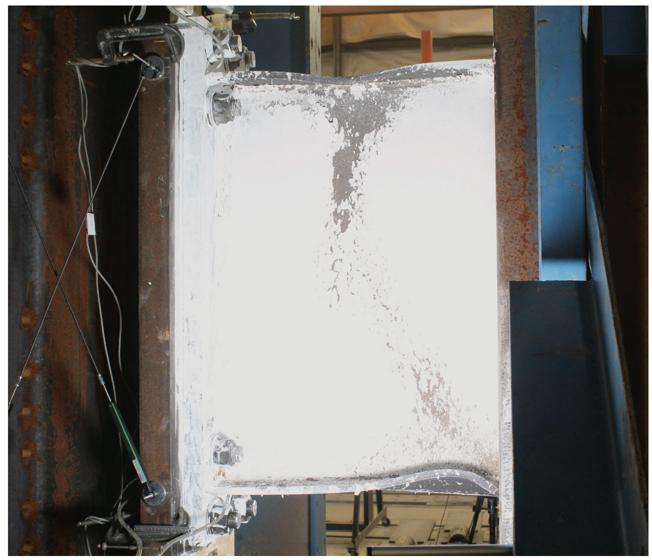


*Fig. 12. Beam flange local buckling extending approximately 3 ft to 4 ft from the face of the column.*

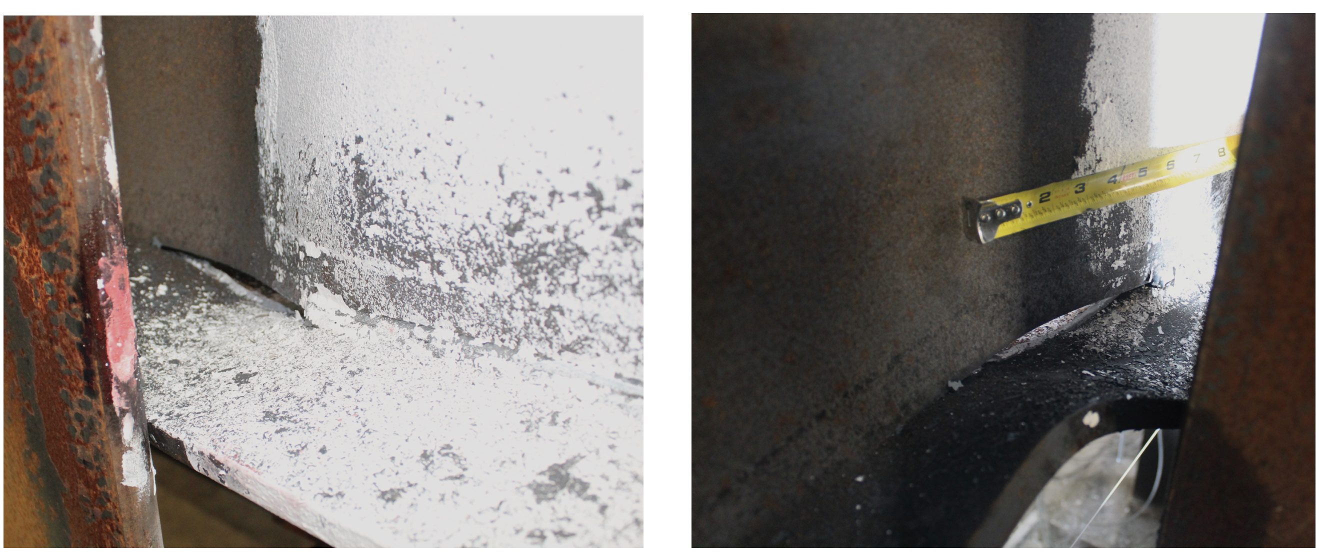


*Fig. 13. Fracture of beam web to beam flange weld of Specimen 4E-1.5-1.75-36b.*

criteria similarly showed buckling over larger lengths (Blumenbaum and Murray, 2004). The larger zone of local buckling either caused or was exacerbated by the fracture of the weld joining the beam web and flange for Specimen 4E-1.5-1.75-36b.

It appears that the larger web slenderness, compared to previous SMF qualification tests, contributed to the local buckling extending over a longer length and the observed weld fracture. Therefore, it is concluded that moment connections with moderately ductile section classification designed for IMF require longer length of the larger beam web to flange weld. Based on the successful test results of Specimen 4E-1.5-1.75-36c, where the beam web to flange weld did not fracture, a weld length double that currently required in AISC 358-22, Section 6.3.1(1), is recommended with beams satisfying moderately ductile section classification.

## CONCLUSIONS AND RECOMMENDATIONS

Three specimens were tested to evaluate whether four-bolt extended, unstiffened moment connections with 36-in.-deep built-up beams can satisfy IMF qualification criteria in AISC 341-16 (2016a). All three specimens satisfied IMF qualification criteria by retaining a moment strength at the face of the column flange greater than 80% of the nominal plastic moment strength of the beam through a cyclic displacement protocol up to 2% story drift. The specimens exhibited excellent behavior through the 2% story drift cycles with only yielding of the section, no observed local buckling, and no undesirable limit states.

The test results support the beam net section fracture requirements in AISC 360-16, Section F13 (2016b), which successfully predicted net section fracture of the beam in the first specimen and the lack of net section fracture in the following two specimens. The test results also verified the difference in requirements for IMF and SMF in two ways: (1) The first specimen, which satisfied moderately ductile lateral bracing requirements, but not highly ductile requirements, experienced lateral torsional buckling after reaching 2% story drift (IMF), but before reaching 4% story drift (SMF). (2) The beam webs satisfied moderately ductile section classification but not highly ductile section classification. Local buckling of the beam in the third specimen occurred after satisfying the qualification requirements for IMF, but led to a severe reduction in moment strength at 4% story drift which was less than required for SMF qualification.

Based on the results of this study, the following recommendations are made for modifying the requirements in AISC 358-22 (2022):

1. Table 6.1 should be modified to allow a beam depth up to 36 in. for the four-bolt unstiffened (4E) connection configuration for use in intermediate moment frames.
2. Section 6.3.1(1) should be modified to extend the length of the larger weld (CJP or double-filled weld) to a length that is double the current requirement—that is, two times the depth of the beam or six times the flange width, whichever is less, when the beam section satisfies moderately ductile section classification but not highly ductile section classification.

In addition to these recommended changes, net section fracture of the beams and columns in seismic moment frames should be prevented at the location of any holes. For the specimens tested herein, net section fracture was avoided when the design moment strength, calculated in accordance with AISC 360-16, Section F13, was greater than the required moment strength at the hole location, calculated based on the beam reaching its probable maximum moment at the plastic hinge.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