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Structural Engineering and Materials

EFFECT OF STIFFENER WELD DETAIL ON CYCLIC PERFORMANCE OF END-PLATE MOMENT CONNECTIONS

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Report No. CE/VPI-ST-20/05

July 2020

EXECUTIVE SUMMARY

There are currently two types of special moment frame (SMF) stiffened end-plate connections allowed in AISC 358-16: four-bolt extended stiffened (4ES), and eight-bolt extended stiffened (8ES) (AISC 2016a). Most of the specimens used for the qualification testing of the two configurations were fabricated using A36 beam and stiffener steel or A572 Gr. 50 beam and A36 stiffener plate steel. Recently, qualification testing was attempted for a new 12 bolt, stiffened end-plate configuration using built-up 24 in. and 44 in. deep beams. The beam webs and end-plates were A572 Gr. 55 steel while the beam flanges were A529 Gr. 55 steel. Four qualification tests were not successful because of brittle fracture of a beam flange prior to completion of the AISC 341-16 (AISC 2016b) loading protocol. Because of this unexpected failure mode, two tests each using the 4ES and 8ES end-plate configurations and A992 hot-rolled beams with A572 Gr. 50 stiffener material were conducted. Of particular interest was the effect of the weld configuration at the toe of the stiffeners. A complete description the testing program and results is the subject of this report.

The testing program was conducted with four specimens, two having W24×76 beams with 4ES configurations and two with W36×150 beams having 8ES configurations. A cyclic displacement protocol was applied to the specimen in accordance with special moment frame (SMF) qualification testing in AISC 341-16. One each of the 4ES and 8ES connection specimens had the stiffener-to-beam flange weld wrapped around the toe of the stiffener while the other two specimens had welds on the sides of the stiffener only.

All four specimens passed SMF qualification per AISC 341-16 by surviving cycles up to 4% story drift while retaining 80% of the nominal plastic moment at the face of the column. The 4ES and 8ES specimens with welds only on the sides of the stiffener survived 10 cycles and 1 cycle at 5% story drift, respectively, before the flange experienced full fracture. Specimens with the stiffener-to-beam flange weld wrapped around the toe experienced flange fracture one cycle sooner.

These tests showed that 4ES and 8ES connections with A992 rolled beams and A572 Grade 50 stiffener material designed and detailed according to AISC 358-16 are capable of reaching SMF

qualification and are appropriate for use in special moment resisting frames. It is recommended to have the weld from the stiffener to the beam flange on the sides only (not wrapped around the toe of the stiffener) because those specimens exhibited more deformation capacity. However, connections with weld wrapped around the toe of the stiffener also satisfied SMF qualification.

ACKNOWLEDGEMENTS

This report is based on work supported by the American Institute of Steel Construction and in-kind donations by Concentric Steel LLC of Detroit Michigan. The authors gratefully acknowledge the work of Devin Huber of AISC in making this project possible.

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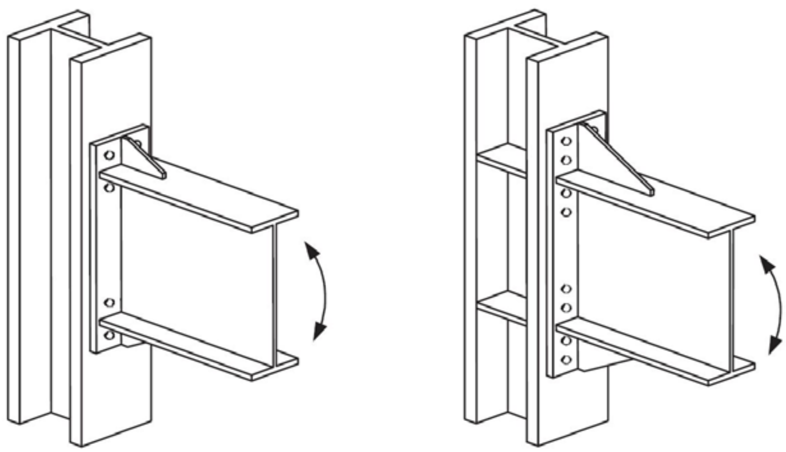
CHAPTER 1 - INTRODUCTION

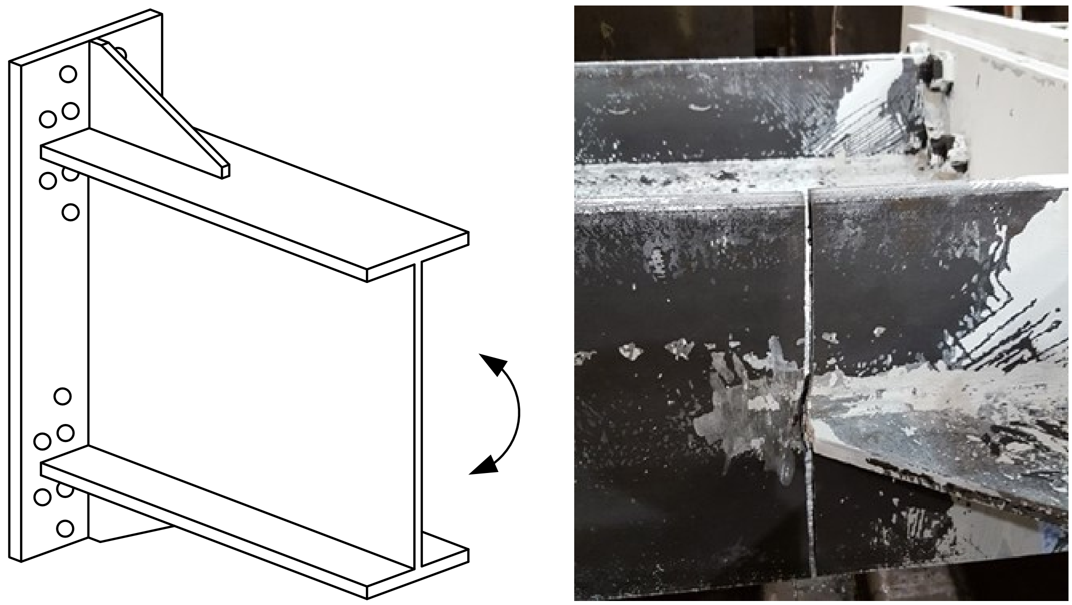
There are currently two types of special moment frame (SMF) stiffened end-plate connections prequalified in AISC 358-16: four-bolt extended stiffened (4ES), and eight-bolt extended stiffened (8ES) configurations as shown in Figure 1.1 (AISC 2016a). Most of the specimens used for the qualification testing were fabricated using A36 beams and stiffener material or A572 Gr. 50 beams with A36 stiffener material.



(a) Four-bolt extended stiffened (4ES) (b) Eight-bolt extended stiffened (8ES)

Figure 1.1. The two types of stiffened end-plate SMRF connections allowed in AISC 358-16

Recently, qualification testing was attempted for a new 12 bolt, stiffened end-plate configuration using built-up 24 in. and 44 in. deep beams. The bolt configuration is shown in Figure 1.2(a). The beam webs and end-plates were A572 Gr. 55 steel while the beam flanges were A529 Gr. 55 steel. The four qualification tests were not successful because of brittle fracture of a beam flange prior to completion of the AISC 341-16 (AISC 2016b) loading protocol. Figure 1.2(b) shows a typical flange fracture. Because of this unexpected failure mode, two tests each using the 4ES and 8ES end-plate configurations and A992 hot-rolled beams with A572 Gr. 50 stiffener material were conducted. Table 1.1 describes four built-up beam specimens from two previous testing programs.



(a) 12 bolt extended stiffened configuration (12ES)   (b) Typical fracture pattern

Figure 1.2. Fracture at the toe of the stiffener for specimen 12ES-1.125-1.25-24
[from Szabo et al. (2017)]

Table 1.1. Matrix of Recent Tests on 12 Bolt Extended Stiffened End-Plate Connections

Specimen	Beam Depth, d (in)	Material Specification	Measured or Mill Certification Yield Stress, F_y	Stiffener Weld Detailing	Result
12ES-1.25-1.50-44a[1] Manufacturer 1	44	Flange: A529 Gr. 55 Web: A572 Gr. 55 Stiffener: A572 Gr. 55	59.3 ksi [3] 59.2 ksi [3] 63.6 ksi [4]	Sides Only	Fracture after first cycle at 2% drift
12ES-1.125-1.25-24[1] Manufacturer 1	24	Flange: A529 Gr. 55 Web: A572 Gr. 55 Stiffener: A572 Gr. 55	59.3 ksi [3] 69.7 ksi [3] 69.7 ksi [3]	Sides Only	Fracture after first cycle at 3% drift
12ES-1.25-1.50-44b[2] –Manufacturer 2	44	Flange: A529 Gr. 55 Web: A572 Gr. 55 Stiffener: A572 Gr. 55	58.3 ksi [4] 60.5 ksi [4] 69.7 ksi [3]	Sides Only	Fracture after first cycle at 1.5% drift
12ES-1.25-1.50-44c[2] - Manufacturer 2	44	Flange: A529 Gr. 55 Web: A572 Gr. 55 Stiffener: A572 Gr. 55	58.3 ksi [4] 60.5 ksi [4] 69.7 ksi [3]	Wrap Around Toe	Fracture after first cycle at 2% drift

[1] From Szabo et al. (2017)
[2] From Zarat-Basir et al. (2020)
[3] Measured yield stress of coupons taken from specimen
[4] Measured yield stress from mill certification reports

The specimens, materials, and fracture surfaces of the 12ES-1.25-1.50-44a and 12ES-1.125-1.25-24 specimens were investigated after testing. It was found that the material used for the stiffeners had a higher yield stress than the beam flange as shown in Table 1.1.

Finite element simulations were conducted in the Szabo et al. (2017) study. The finite element studies determined that having stiffeners with a higher yield stress drives more plastic strain into the beam flange at the toe of the stiffener, which can contribute to earlier fracture. This finding was consistent with the fractures observed during testing, where fractures initiated at the toe of the stiffener as shown in Figure 1.2b.

The finite element study suggested that a simple change in the stiffener weld detailing could reduce the concentration of plastic strain and delay fracture. By wrapping the weld around the toe of the stiffener (Figure 1.3b vs. Figure 1.3a), the plastic strains in the beam flange were predicted to be spread out more and thus delay fracture (Szabo et al. 2017). It is noted that AISC 358-16 does not specify whether the stiffener weld should be on the sides or wrap around the toe.

(a) Welds on sides of stiffener only (b) Weld wraps around toe of stiffener

Figure 1.3. Weld detailing for the stiffener [from Zarat-Basir et al. (2020)]

One of the primary testing programs used in the prequalification of the 8ES configuration was by Sumner and Murray (2002), where it was found that W30×99 and W36×150 beams satisfied the SMF qualification criteria. The material specifications used in that program were A36 stiffener plates and A572 Gr. 50 beams, as was typical practice at that time. However, it is becoming more common for engineers to specify plates that are A572 Gr. 50 and rolled beams that are A992, meaning the stiffener plates and beams are expected to have a similar yield stress.

The main objective of the research reported herein is to examine whether the brittle fracture found in previous testing programs on built-up beams with twelve bolt extended stiffened connections will also occur for the 4ES and 8ES end-plate moment connection configurations with rolled beams and Grade 50 stiffener material. Another objective of this research is to determine whether wrapping the stiffener-to-beam flange weld around the toe of the stiffener is beneficial in delaying fracture.

CHAPTER 2 - EXPERIMENTAL PROGRAM

This chapter describes the test setup, test specimens, and instrumentation plan used in the four connection tests.

2.1 Test Specimens

Four stiffened extended end-plate connections were tested at the Thomas M. Murray Structural Engineering Laboratory at Virginia Tech: two W24×76 beams with a 4ES configuration and two W36×150 beams with an 8ES configuration. The only difference between the similarly sized specimens was the stiffener weld detail. On one specimen of each beam size, the stiffener-to-beam flange fillet weld was wrapped around the toe, while on the other specimen of the same beam size, the weld was not wrapped around the toe. The test matrix is given in Table 2.1 and an example of each connection type is shown in Figure 2.1. An example of each weld detail is also shown in Figure 2.2.

These beam sizes were chosen because they represent the largest rolled shapes that can be used with their respective connections. This means the extreme fiber strains will be greater than other beam sections, and thus these specimens may be considered a worst-case configuration for fracture potential at the toe of the stiffener. The connections were designed in accordance with AISC 358-16 (AISC 2016a).

Table 2.1. Test matrix

Specimen Name	**Beam Size**	**Bolt Configuration**	**Stiffener Weld Detailing**	**Purpose**
4ES-1.375-1.25-24sides	W24×76	Four-Bolt Extended Stiffened (4ES)	Sides Only	Largest beam for 4ES. Verify existing provisions for 4ES.
4ES-1.375-1.25-24wrap	W24×76	Four-Bolt Extended Stiffened (4ES)	Wrap Around Toe	Investigate the effect of wrap around weld on ductility.
8ES-1.375-1.5-36sides	W36×150	Eight-Bolt Extended Stiffened (8ES)	Sides Only	Largest beam allowed for 8ES. Verify existing provisions for 8ES.
8ES-1.375-1.5-36wrap	W36×150	Eight-Bolt Extended Stiffened (8ES)	Wrap Around Toe	Investigate the effect of wrap around weld on ductility.

(a) 4ES

(b) 8ES

Figure 2.1. Example connections onW24×76 and W36×150 beams, with whitewash applied

(a) Welds on stiffener sides only

(b) Weld wrapped around stiffener toe

Figure 2.2. Examples of stiffener weld details

Specimen details are shown in Figure 2.3 and 2.4 and material properties are given in Table 2.2. Shop drawings, mill certification reports, and weld procedure specifications are provided in Appendices A, E and R, respectively. The 1-3/8 in. diameter bolts were pretensioned using a pneumatic impact wrench and the turn-of-the-nut tightening method.

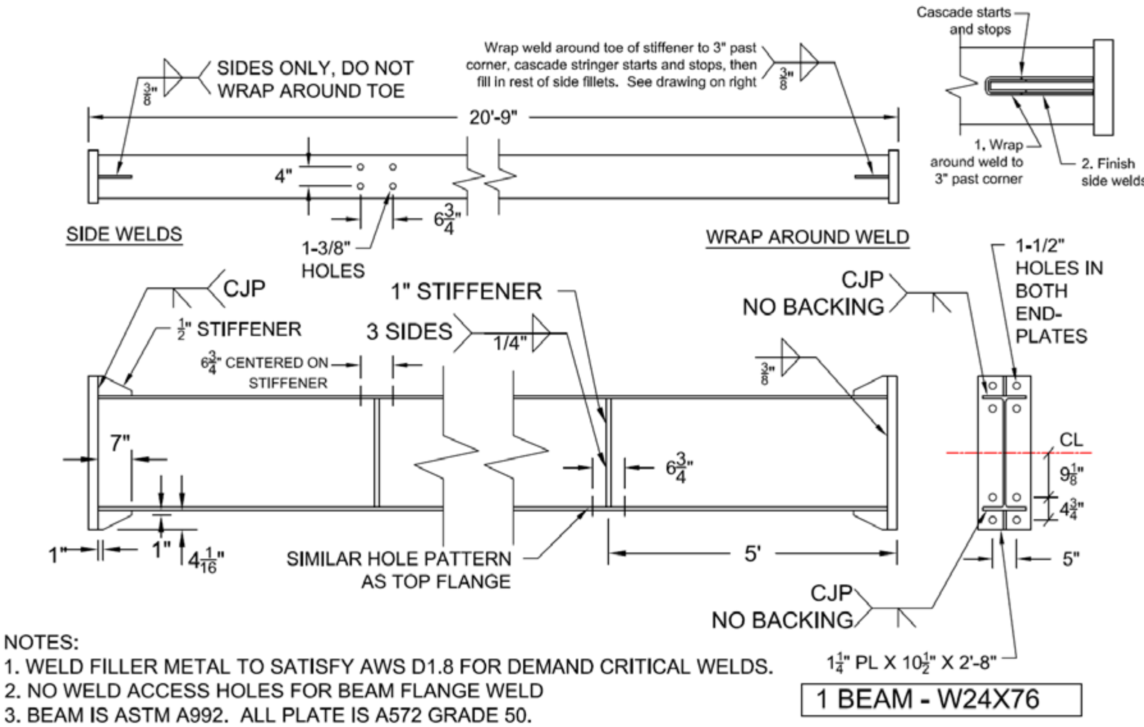


Figure 2.3. Details of the W24×76 specimens (Each end represents one specimen)

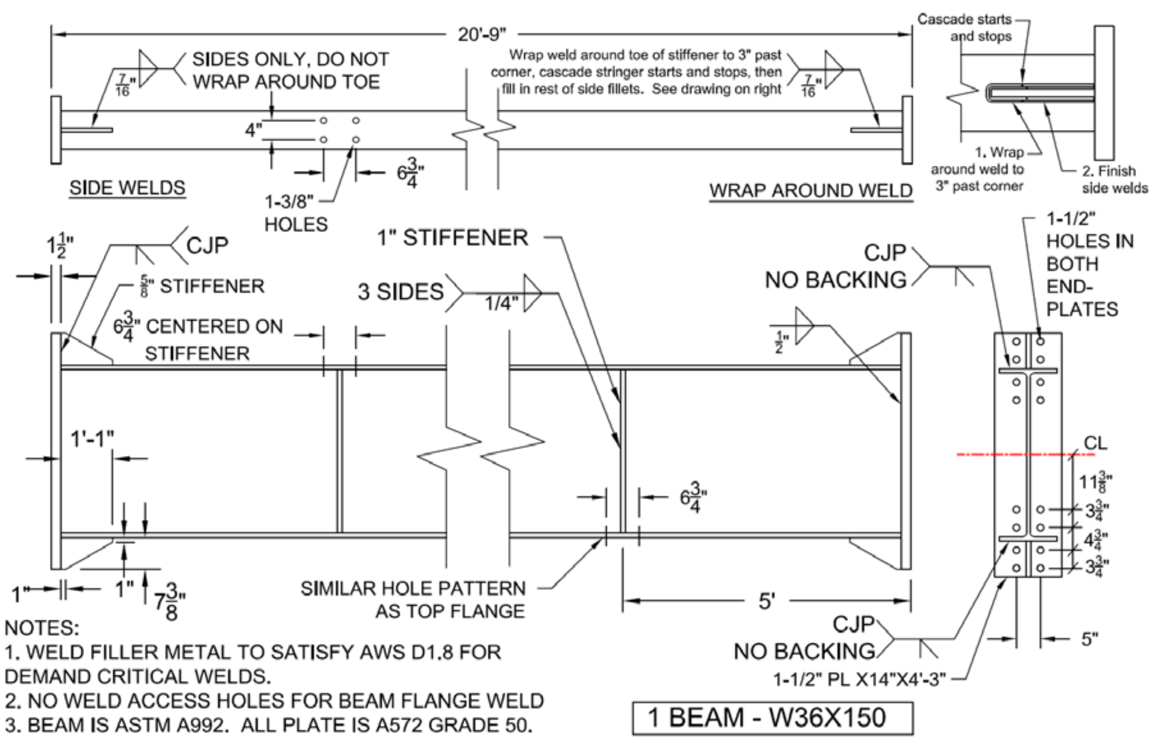


Figure 2.4. Details of the W36×150 specimens (Each end represents one specimen)

Table 2.2. Material Properties from Mill Certification Reports

	Material Specification	Yield Stress (ksi)	Ultimate Stress (ksi)
W24x76	A992	55.8	69.9
W36x150	A992	53.4	69.1
1/2 in. Thick Stiffener for W24 Specimens	A572 Gr. 50	66.0	73.6
5/8in. Thick Stiffener for W36 Specimens	A572 Gr. 50	54.6	77.4
1-1/4 in. Thick End-Plate for W24 Specimens	A572 Gr. 50	56	70
1-1/2 in. Thick End-Plate for W36 Specimens	A572 Gr. 50	50	75

2.2 Test Setup

The specimen and setup configuration, shown in Figures 2.5 and 2.6, simulated an exterior moment connection in a frame with 32 ft. wide bays and 12 ft. story heights. The reaction column was a W14×398 that was reused for each of the four tests. Load was applied using an MTS 201.70 servohydraulic actuator, which had a tension capacity of 220 kips and a compression capacity of 330 kips. Lateral bracing was provided at the end of the plastic hinge and near the point of loading to limit lateral torsional buckling outside of the plastic hinge region. Additional drawings of the test setup are given in Appendix B.

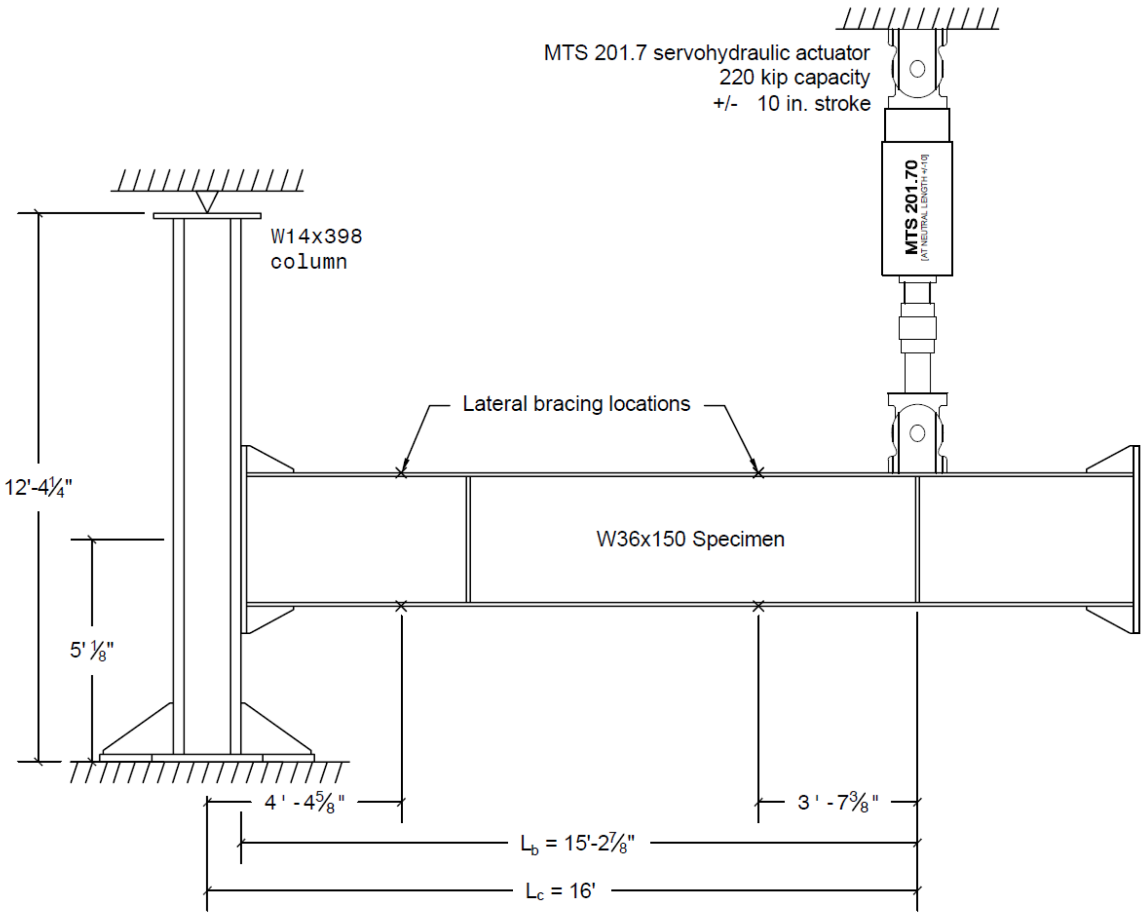


Figure 2.5. Schematic drawing of beam-to-column moment connection test setup

Figure 2.6. Photograph of test setup

2.3 Instrumentation Plan

The instrumentation plan, shown in Figure 2.7, included thirteen displacement sensors that were used to control the loading application and decompose the story drift into components. The internal actuator load cell and linear voltage differential transformer measured the applied force and displacement, respectively. Two instrumented spring calipers were placed at the centerlines of the top and bottom beam flanges to measure end-plate separation from the column flange. All sensors were connected to a National Instruments data acquisition system which was managed using National Instruments Signal Express software.

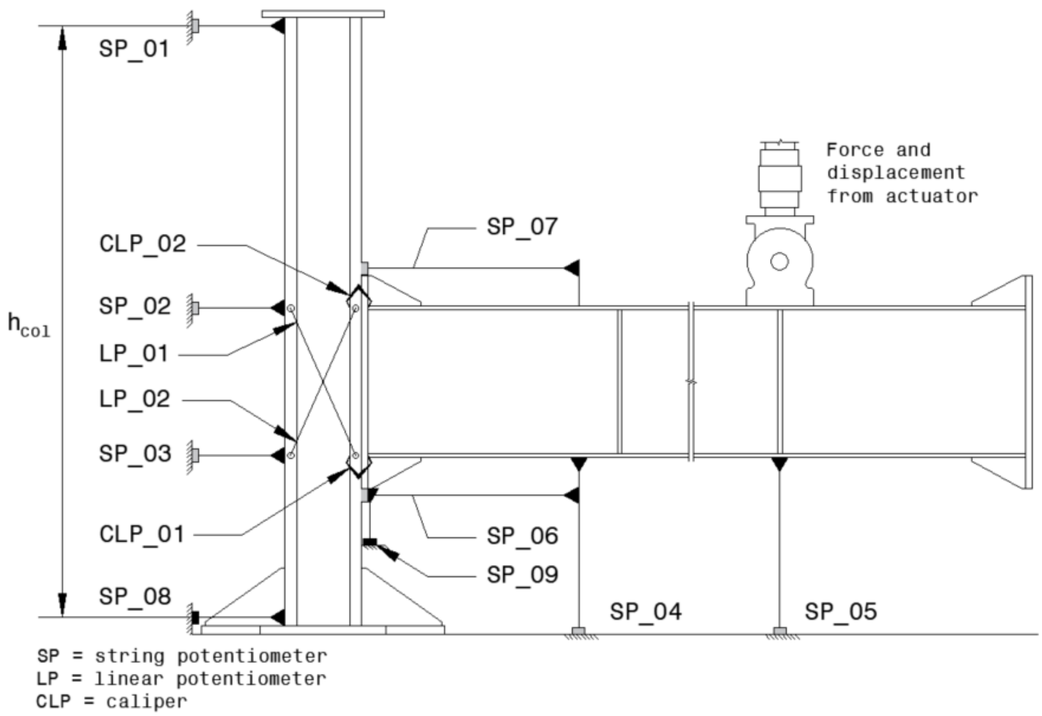
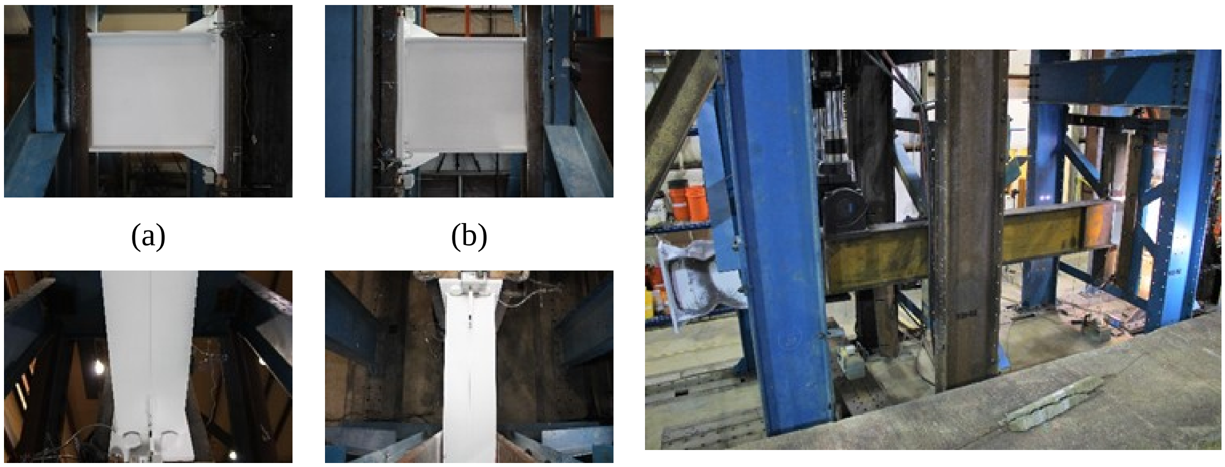


Figure 2.7. Instrumentation plan for all tests

In addition to these sensors, the tests were recorded with Canon digital SLR cameras; four cameras captured different angles of the beam-column connection and one camera captured an overall view of the setup. An example of the camera views is shown in Figure 2.8. Pictures were taken every six seconds using the GB Timelapse software and were compiled into timelapse videos to show the behavior of the connection over the course of the test. Hydrated lime whitewash was also used to indicate where the steel had yielded and where stresses were likely concentrated. The whitewash was a 1:1 mixture of lime and water that adhered to the beam when applied, but fell off when the mill scale flaked off.



(a) (b) (c) (d) (e)

Figure 2.8. Composite of camera views for connection tests

2.4 Loading Protocol and SMF Qualification Criteria

The loading protocol followed the qualification displacement protocol for special moment frames as given in AISC 341-16 (AISC 2016b) and shown is in Table 2.3. After completing the two 4% story drift cycles, the specimens were subjected to 5% story drift cycles until failure. For all tests, the specimens were loaded at a rate of 0.0002 radians/second through the first two cycles of 5% story drift, after which the rate was doubled to 0.0004 radians/second. These rates correspond to vertical displacement of approximately 2.3 in./min. and 4.6 in./min., respectively.

Table 2.3. Loading protocol for qualification testing of end-plate moment connections

Story Drift		Number of Cycles
Radians	Percent	
0.00375	0.375%	6
0.005	0.5%	6
0.075	0.75%	6
0.01	1%	4
0.015	1.5%	2
0.02	2%	2
0.03	3%	2
0.04	4%	2
0.05	5%	Until fracture

To ensure the loading protocol was accurately followed, the applied story drift was calculated real time and used in an active feedback loop to control the movement of the actuator. This was accomplished in the MTS Multipurpose Testware Software with the Calculations Module. The applied story drift, θ_{app}, was calculated using Eq. (2.1),

$$\theta_{app} = \frac{-\delta_{SP\_05}}{L_c} + \frac{\delta_{SP\_01} - \delta_{SP\_08}}{h_{col}} \tag{2.1}$$

where:

h_{col} = distance between SP_01 and SP_08, 11.4 ft.
L_c = distance from the actuator centerline to column centerline, 16 ft.
δ_{SP_01} = displacement measured by SP_01
δ_{SP_05} = displacement measured by SP_05
δ_{SP_08} = displacement measured by SP_08

The qualification criterion for special moment frame connections in AISC 341-16 is that the connection must maintain a moment at the face of the column that is at least 80% of the nominal flexural resistance, through 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 point of loading to the face of the column, 15.2 ft. This was then compared to the nominal flexural resistance of the section: 833 kip-ft. for W24×76 beams or 2,420 kip-ft. for W36×150 beams (AISC 2017).

CHAPTER 3 - RESULTS

This section presents the general results of the four tests, and then describes the buckling and fracture propagation for the individual specimens. Finally, the story drift components are presented for three of the four tests; sensor data for 8ES-1.375-1.5-36wrap was inadvertently not recorded so the story drift decomposition could not be completed.

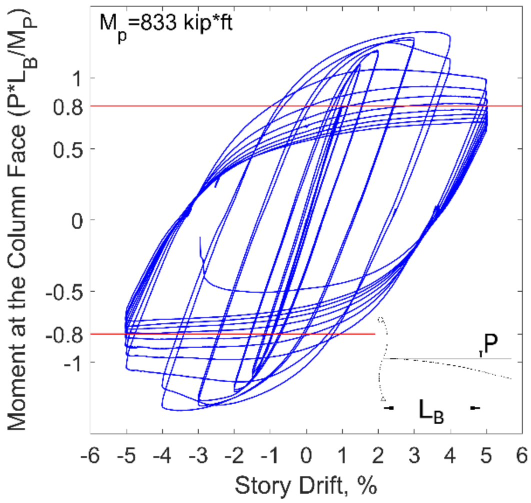
3.1 Moment-Rotation Behavior

All four specimens met the qualification criterion, as the moment at the face of the column was at least 80% of the nominal flexural resistance of the beam through the first cycle of 4% story drift. The beam moments at the face of the column at the peaks of the first 4% story drift cycle are given in Table 3.1, along with the value of 80% of the nominal plastic flexural resistance. Because the moment was calculated using the measured actuator force, the additional moment due to the weight of the actuator swivel and the weight of the beam are not included. The weight of the actuator swivel, W36x150 beam specimen, and the W24x76 beam specimen were 2000 lbs, 3000 lbs, and 1500 lbs, respectively which leads to a moment at the face of the column equal to 730 k-in. and 550 k-in. for the W36x150 and W24x76 specimens respectively. The numbers in parentheses in Table 3.1 give the peak moment adjusted for these weights. The moment-rotation response of the specimens is shown in Figure 3.1 without correction for beam or actuator swivel weight.

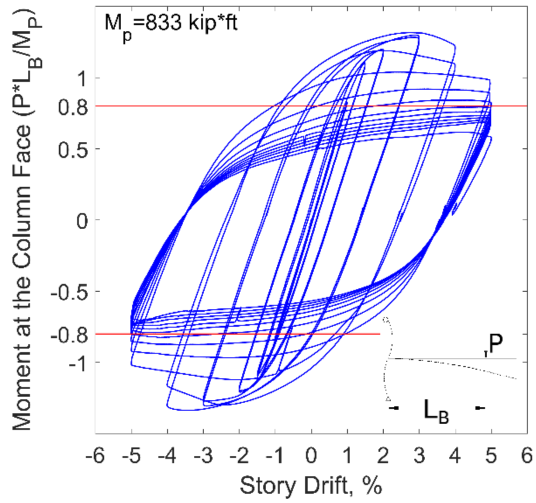
Table 1.1. Moment at the face of the column during the first cycle at 4% story drift

Specimen Name	Moment at Negative Peak* (k-in)	Moment at Positive Peak* (k-in)	80% of nominal M_p (k-in)
4ES-1.375-1.25-24sides	13,030 (12,480)	12,740 (13,290)	8,000
4ES-1.375-1.25-24wrap	13,000 (12,450)	12,120 (12,670)	8,000
8ES-1.375-1.5-36sides	31,980 (31,250)	30,300 (31,030)	23,240
8ES-1.375-1.5-36wrap	34690 (33,960)	32310 (33,040)	23,240

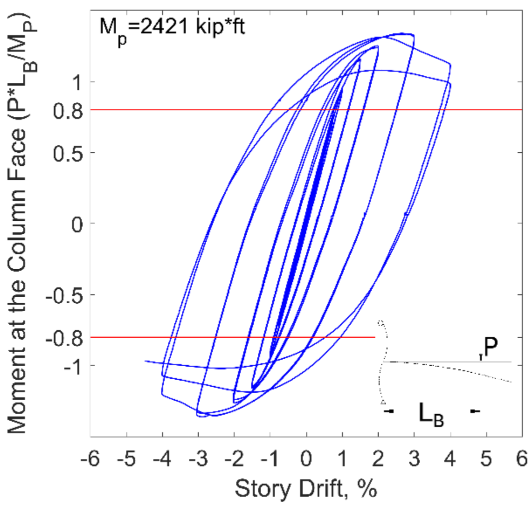
*Number is parentheses is adjusted for the weight of the actuator swivel and the beam.



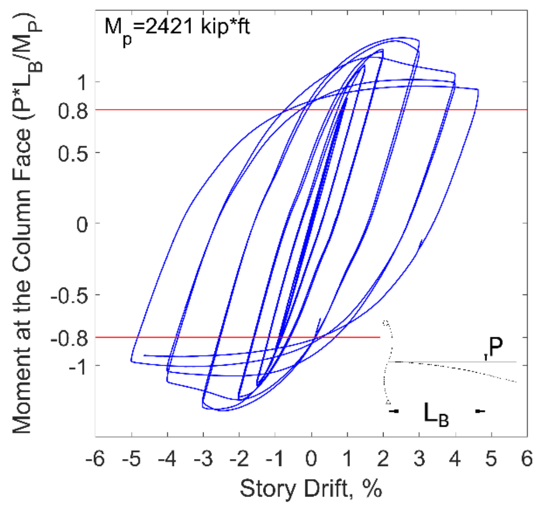
(a) 4ES-1.375-1.25-24wrap



(b) 4ES-1.375-1.25-24sides



(c) 8ES-1.375-1.5-36wrap



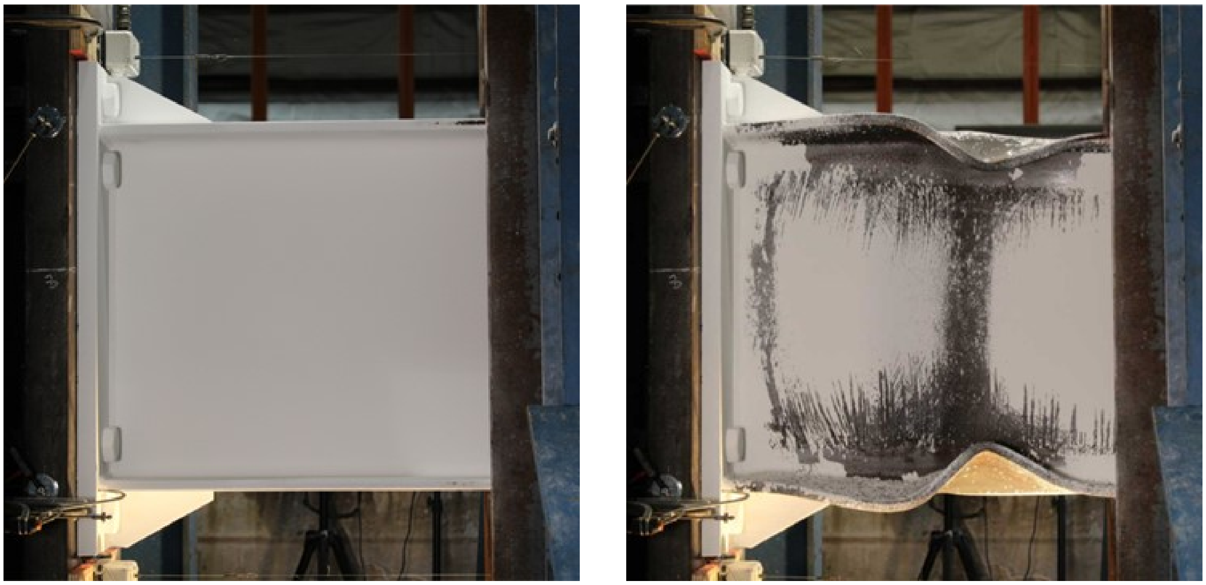
(d) 8ES-1.375-1.5-36sides

Figure 3.1. Moment-rotation plots for end-plate connections

3.2 Buckling and Fracture Behavior

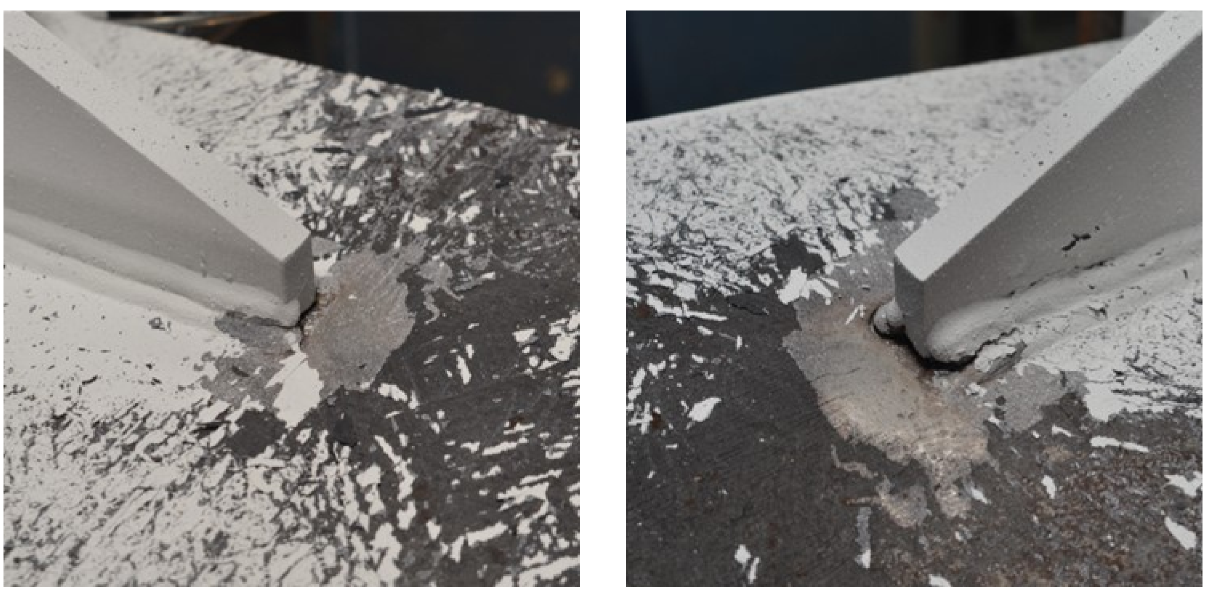
3.2.1 Specimen 4ES-1.375-1.25-24sides

The condition of the specimen before testing and after fracture of the top flange is shown in Figure 3.2. There was substantial flange local buckling that began in the first 4% story drift cycle and increased through the remaining cycles. A fracture was found at the toe of the top flange stiffener welds, shown in Figure 3.3, at the end of the 3% story drift cycles. A similar fracture was found on the bottom flange stiffener welds, shown in Figure 3.4, but neither fracture propagated away from the stiffener welds. A field of fractures also formed on the inside of the local buckles on both flanges. The fractures on the top flange local buckles grew and coalesced to form a 5.5 in. long ductile tear in the ninth cycle of 5% story drift, shown in Figure 3.5. This fracture was located about 3 in. from the toe of the stiffener.



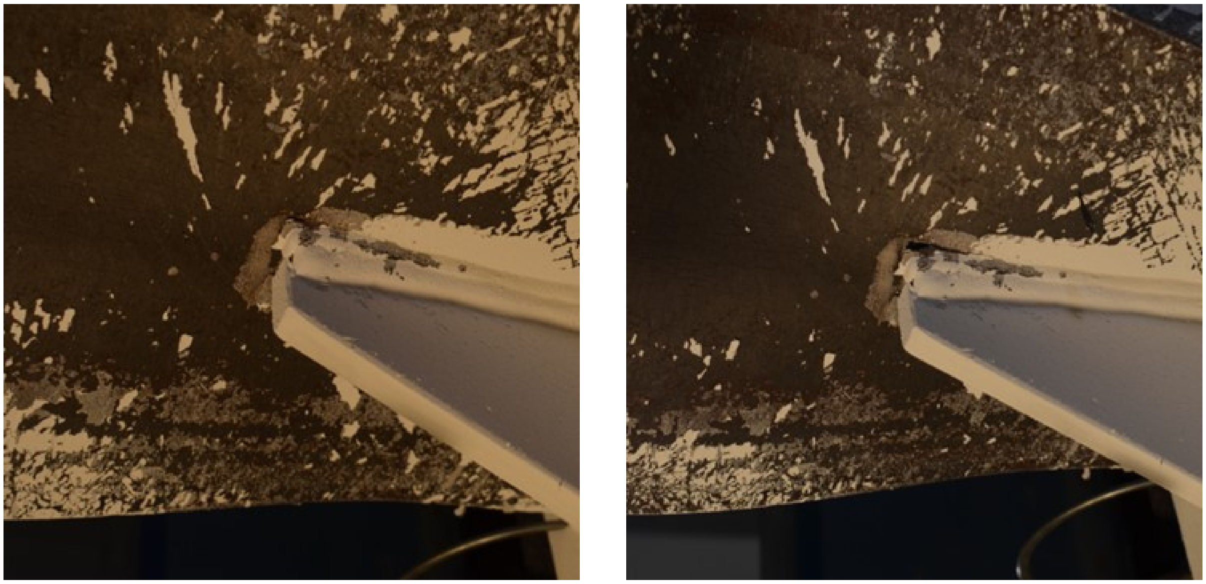
(a) Undeformed (b) Deformed

Figure 3.2. Undeformed and deformed conditions of 4ES-1.375-1.25-24sides



(a) End of 3% story drift cycles (b) End of third 5% story drift cycles

Figure 3.3. Top flange fracture initiation and propagation of 4ES-1.375-1.25-24sides



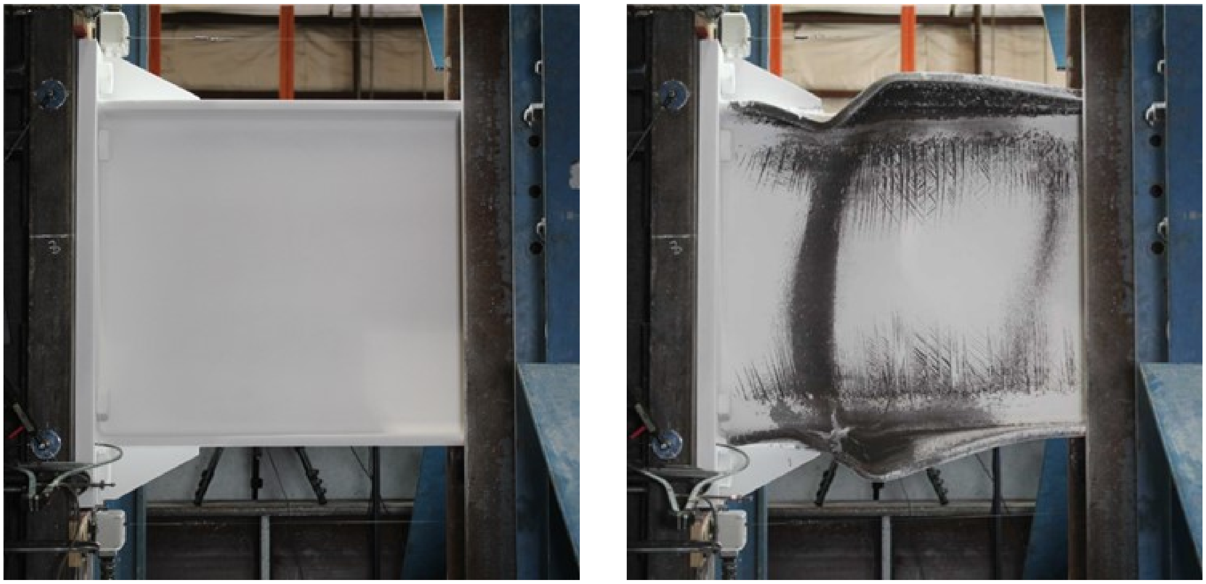
(a) End of 4% story drift cycles (b) End of fifth 5% story drift cycle

Figure 3.4. Bottom flange fracture initiation and propagation of 4ES-1.375-1.25-24sides

Figure 3.5. Top flange fracture of 4ES-1.375-1.25-24sides

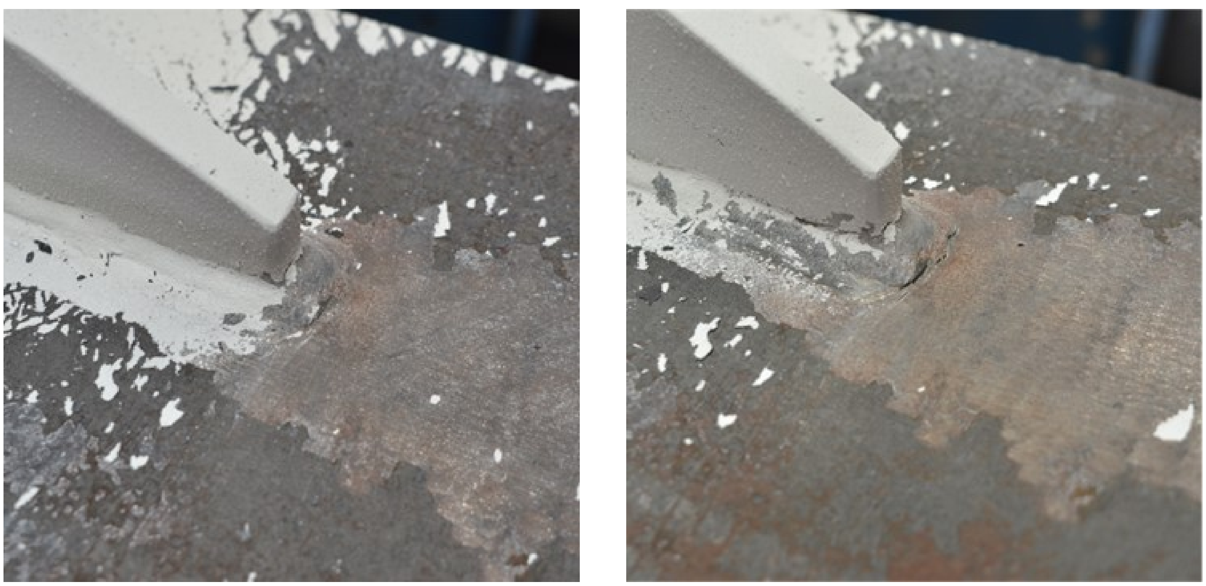
3.2.2 Specimen 4ES-1.375-1.25-24wrap

The condition of the specimen before testing and after fracture of the bottom flange is shown in Figure 3.6. On the top flange, fractures were found at the toe of the stiffener welds at the end of the 4% story drift cycles, shown in Figure 3.7a. This fracture grew slightly in the 5% story drift cycles, but not beyond the size shown in Figure 3.7b. On the bottom flange, fractures at the toe of the stiffener welds were observed in the first cycle of 5% story drift. These fractures grew steadily, shown in Figures 3.8a to 3.8c, until a ductile tear formed and spread across the bottom flange, shown in Figure 3.8d. This fracture also spread 2 in. into the web, shown in Figure 3.9.



(a) Undeformed (b) Deformed

Figure 3.6. Undeformed and deformed conditions of 4ES-1.375-1.25-24Wrap



(a) End of 4% story drift cycles (b) End of third 5% story drift cycle

Figure 3.7. Top flange fracture initiation and propagation4ES-1.375-1.25-24Wrap

(a) End of third 5% story drift cycle

(b) End of fifth 5% story drift cycle

(c) End of eighth 5% story drift cycle

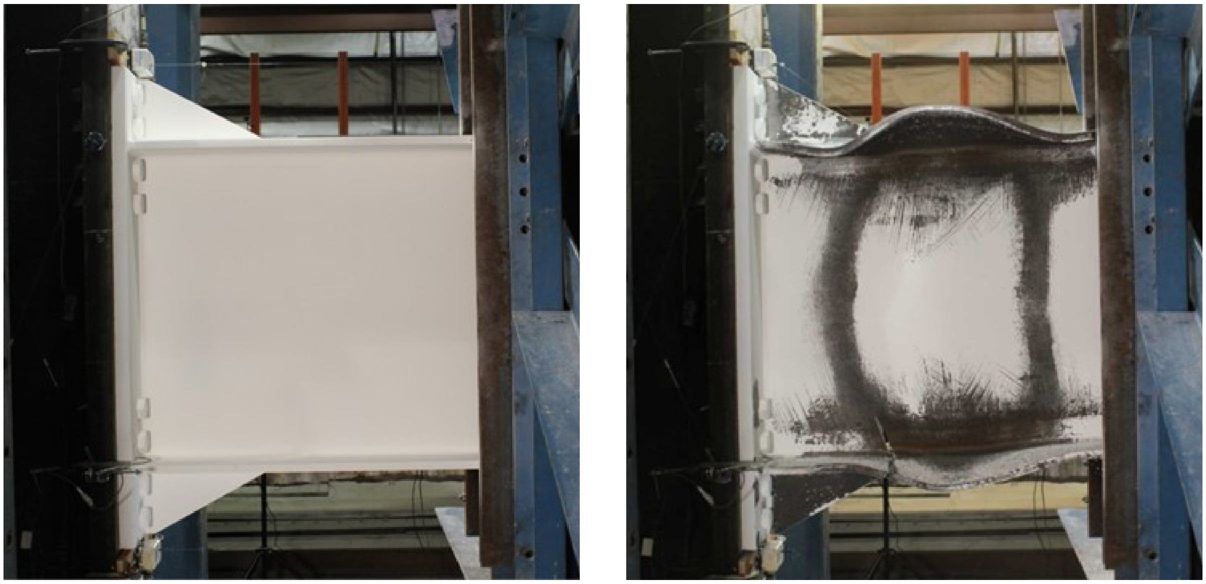
(d) End of ninth 5% story drift cycle

Figure 3.8. Bottom flange fracture initiation and propagation4ES-1.375-1.25-24Wrap

Figure 3.9. Partial fracture of the beam web 4ES-1.375-1.25-24Wrap

3.2.3 Specimen 8ES-1.375-1.5-36sides

The condition of the specimen before testing and after a brittle fracture of the bottom flange is shown in Figure 3.10. There was moderate buckling of the top and bottom flanges that began in the first 2% story drift cycle. A fracture at the toe of the top flange stiffener weld was found in the same cycle, and slowly propagated during the 3% and 4% story drift cycles, shown in Figure 3.11. After the first cycle of 5% story drift, a small fracture was found at the toe of the bottom flange stiffener, shown in Figure 3.12b. In the following cycle, a sudden, brittle fracture initiated at the toe of the bottom flange stiffener and propagated across the entire flange. This fracture spread 4.5 into the web and is shown in Figure 3.13.

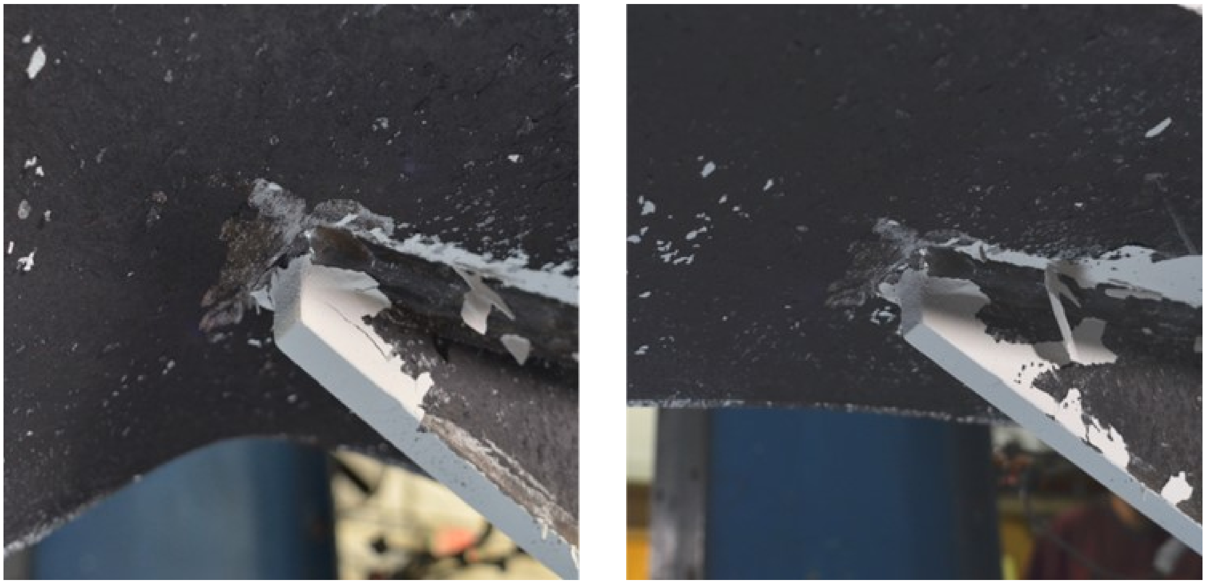


(a) Undeformed (b) Deformed

Figure 3.10. Undeformed and deformed conditions of 8ES-1.375-1.5-36sides

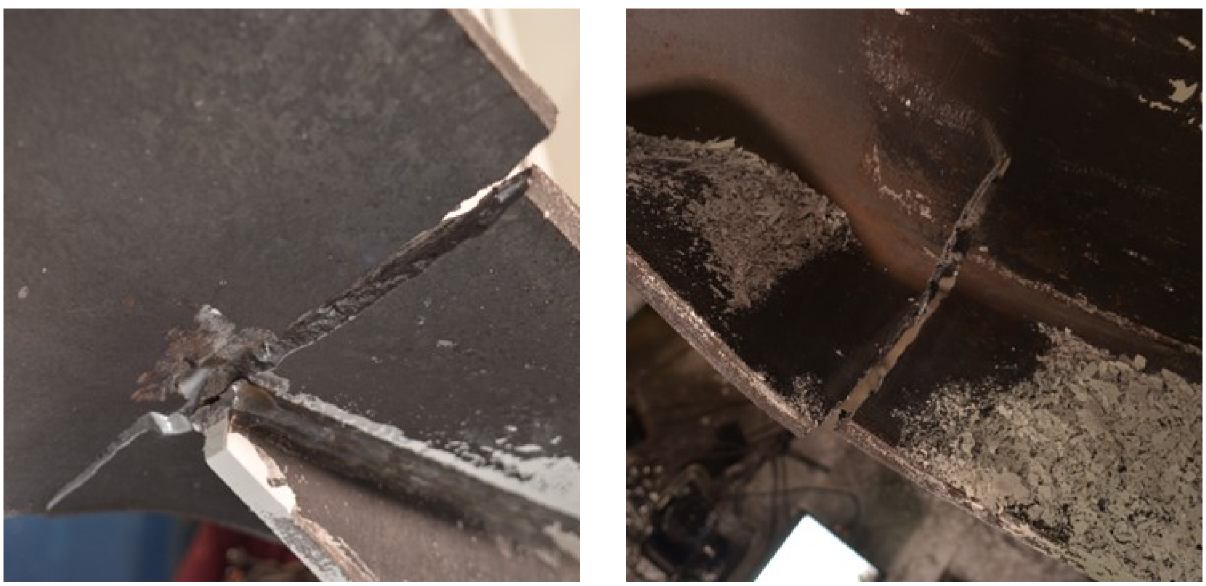
(a) End of 2% story drift cycles (b) End of 4% story drift cycles

Figure 3.11. Top flange fracture initiation and propagation 8ES-1.375-1.5-36sides



(a) End of 3% story drift cycles (b) End of first 5% story drift cycle

Figure 3.12. Bottom flange fracture initiation and propagation 8ES-1.375-1.5-36sides

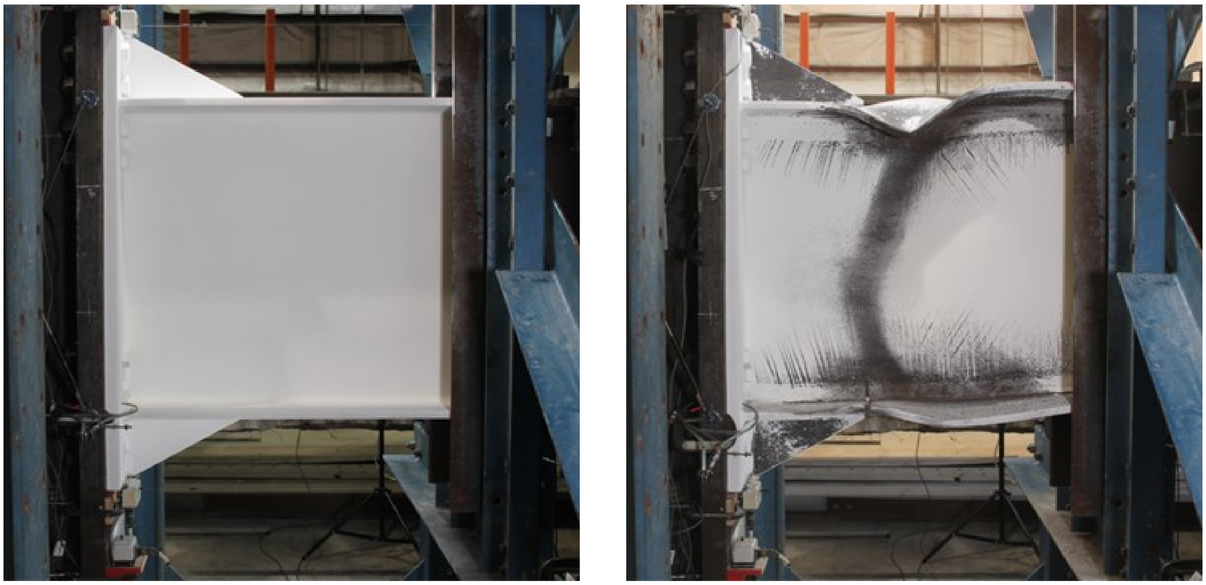


(a) Bottom flange fracture (b) Partial web fracture

Figure 3.13. Brittle fracture of 8ES-1.375-1.5-36sides 8ES-1.375-1.5-36sides

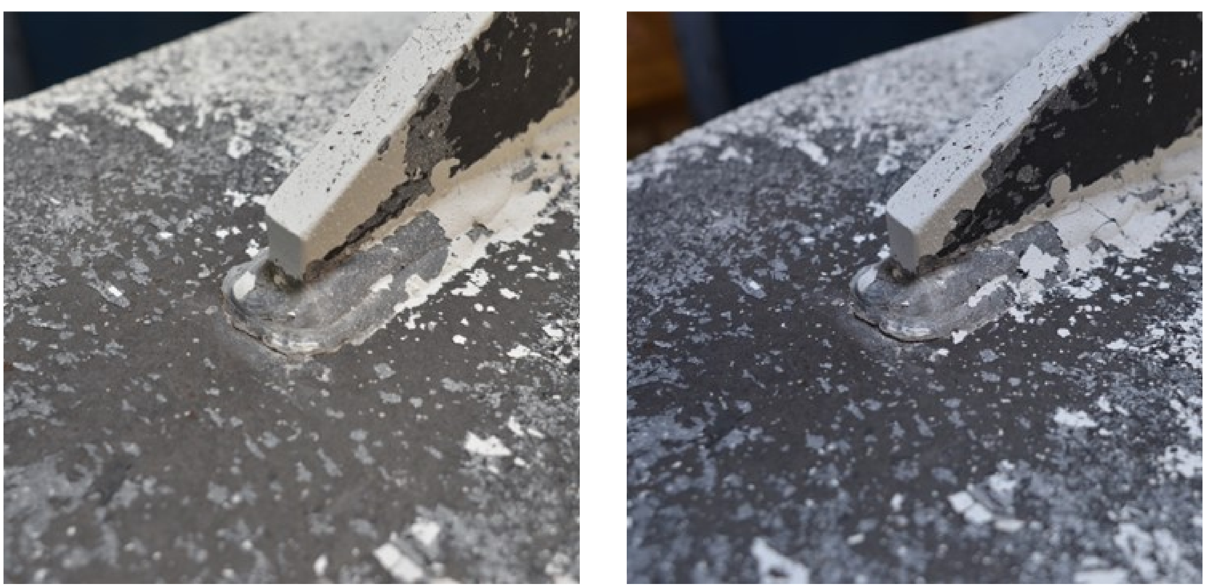
3.2.4 Specimen 8ES-1.375-1.5-36wrap

The condition of the specimen before testing and after a brittle fracture of the bottom flange is shown in Figure 3.14. There was moderate local buckling of the top flange but little buckling of the bottom flange. A fracture was found at the toe of the top flange stiffener weld at the end of the 3% story drift cycles, shown in Figure 3.15, but only grew slightly in the 4% story drift cycles. In the first 5% story drift cycle, there was a sudden, brittle fracture of the bottom flange that initiated at the toe of the stiffener weld. This fracture propagated across the flange width and 4 in. into the web, shown in Figure 3.16. There were no visible fractures observed on the bottom flange preceding the brittle fracture.



(a) Undeformed (b) Deformed

Figure 3.14. Undeformed and deformed conditions of 8ES-1.375-1.5-36wrap



(a) End of third 5% story drift cycle (b) End of second 4% story drift cycle

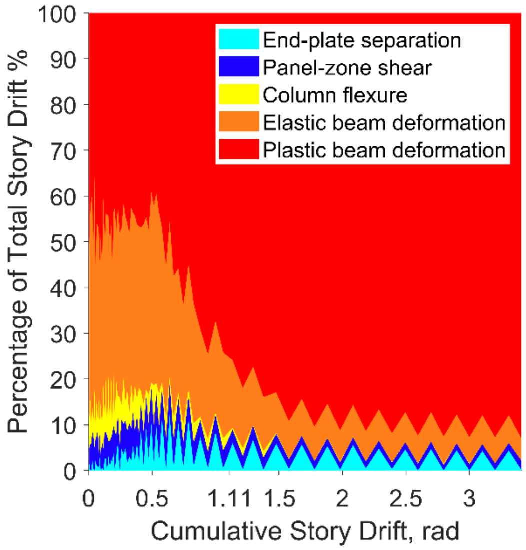
Figure 3.15. Top flange fracture initiation and propagation 8ES-1.375-1.5-36wrap

(a) Bottom flange fracture (b) Partial web fracture

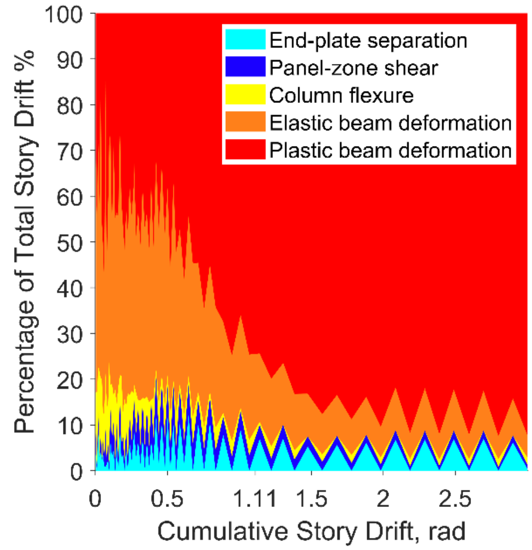
Figure 3.16. Brittle fracture of 8ES-1.375-1.5-36wrap

3.3 Story Drift Decomposition

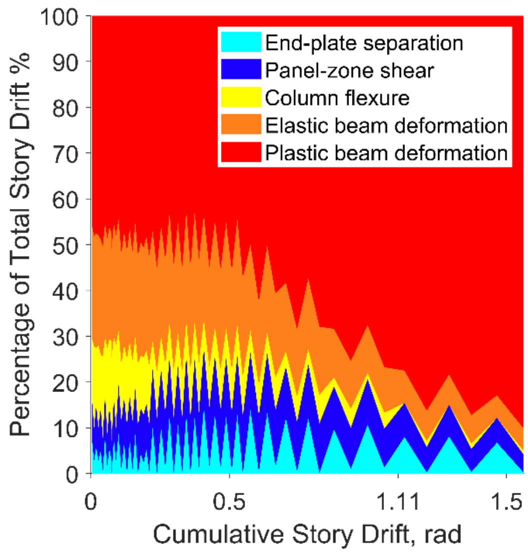
AISC 341-16 states that for qualification testing, the percentage of the total inelastic rotation in the test specimen that is developed in each member or connection element shall be within 25% of the anticipated percentage of the total inelastic rotation in the prototype that is developed in the corresponding member or connection element. To determine if this requirement was met, the contributions of each component of the specimen to the story drift were isolated using the displacement sensors described in Chapter 2. As shown graphically in Figure 3.17, during the 4% story drift cycles, the inelastic rotation in the plastic hinge region was between 74% and 87% of the applied story drift. Results are not shown for Specimen 8ES-1.375-1.5-36wrap because of a problem with the data acquisition.



(a) 4ES-1.375-1.25-24sides



(b) 4ES-1.375-1.25-24wrap



(c) 8ES-1.375-1.5-36sides

Figure 3.17. Components of story drift due to different sources of deformation

CHAPTER 4 – DISCUSSION AND CONCLUSIONS

A research program was conducted to (1) evaluate whether four- and eight-bolt bolt extended stiffened moment connections were susceptible to premature beam flange fracture when using 50 ksi yield strength hot-rolled beam and stiffener plate material, and (2) to evaluate whether wrapping the stiffener-to-beam flange weld around the toe of the stiffener increased the deformation capacity of the connection or not. Four full-scale moment connection specimens were tested: two 4ES connections and two 8ES connections using the largest hot-rolled beam sections allowed for special moment frame (SMF) connections per AISC 358-16. One each of the 4ES and 8ES specimens had the stiffener-to-beam flange on the sides of the stiffener only, and the other two specimens had the weld wrapped around the toe of the stiffener.

All four specimens passed the SMF qualification criteria set forth in AISC 341-16. This implies that extended stiffened end-plate moment connections designed per AISC 358-16 and using A572 Gr. 50 stiffener plates with A992 rolled beam shapes have sufficient ductility to be used in special moment frames.

Specimens with the stiffener-to-beam flange weld wrapped around the toe of the stiffener had slightly larger flexural resistance (between 1% and 8%), but failed at one less cycle of rotation than their counterparts with the weld on the sides of the stiffener only. While all specimens passed the SMF qualification criteria and are thus considered adequate for use in SMF, detailing the weld on the sides of the stiffener only is recommended.

Even though previous finite element studies suggested that wrapping the weld around the toe of the stiffener spreads out the inelastic strains (Szabo et al. 2017), it is possible that the increased triaxiality in the stress state at the toe of the stiffener led to slightly larger fracture potential overall. Three of the specimens experienced fracture at the toe of the stiffener, while the only specimen that exhibited fracture at the local buckles of the plastic hinge was the 4ES specimen with W24×76 beam and welds on the side of the stiffener.

REFERENCES

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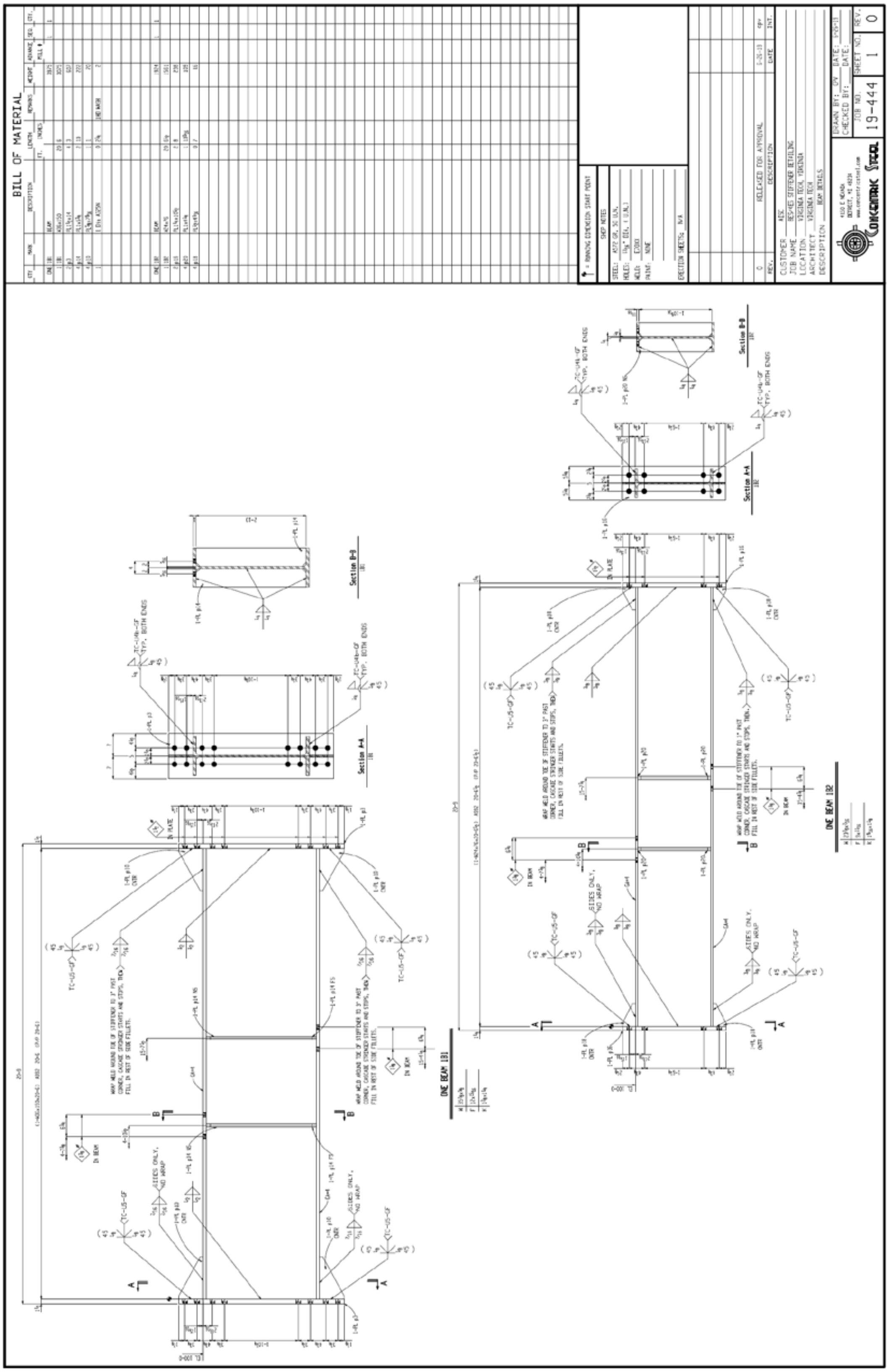
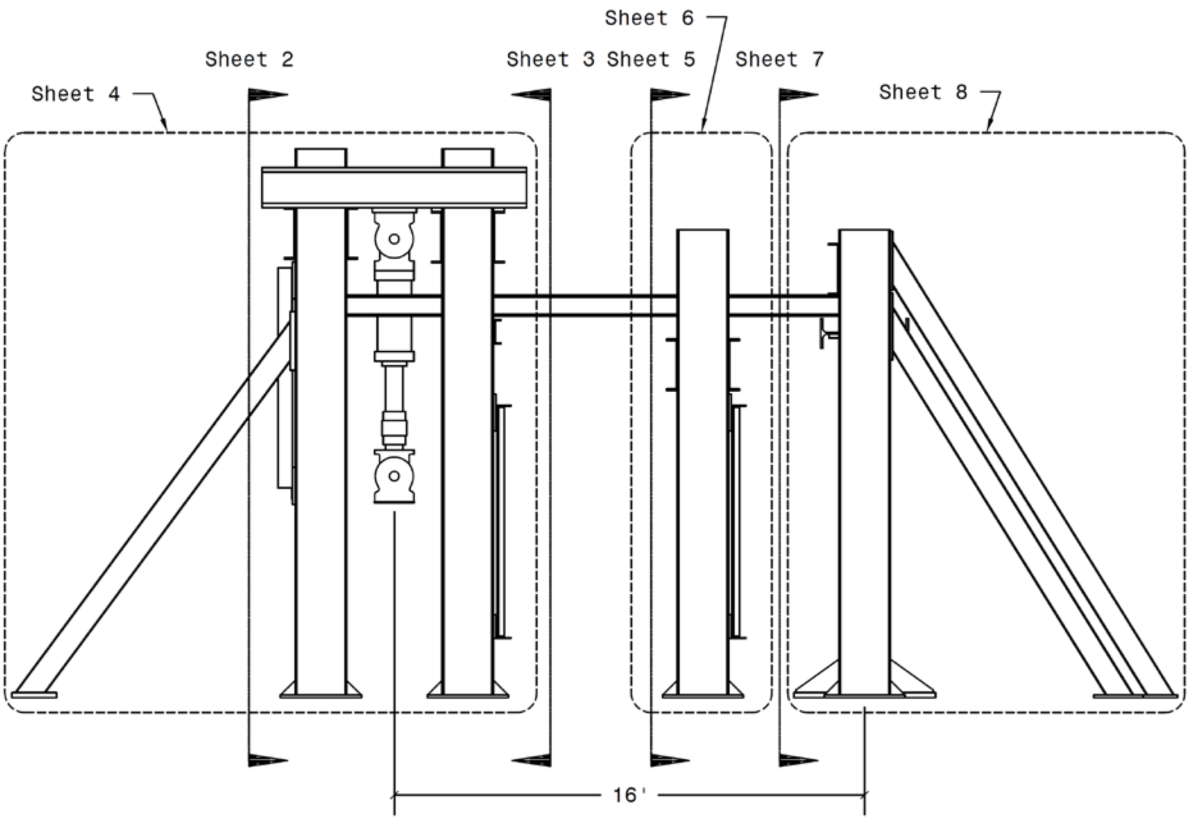
Zarat-Basir, M., Eatherton, M.R., and Murray, T.M. (2020). "Testing Of Stiffened and Unstiffened Twelve Bolt Extended End-Plate Moment Connections." *Report No. CE/VPI-ST-20/01*, Virginia Tech, Blacksburg, VA.

APPENDIX A - TEST SPECIMEN SHOP DRAWING



APPENDIX B - TEST SETUP DRAWINGS



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200 PATTON HALL
BLACKSBURG, VA 24061
P: 540-231-6635

PROJECT:
Wrap-Around Welds on End-Plate Stiffeners

SHEET NOTES:
1. Scale is 3/8"=2'

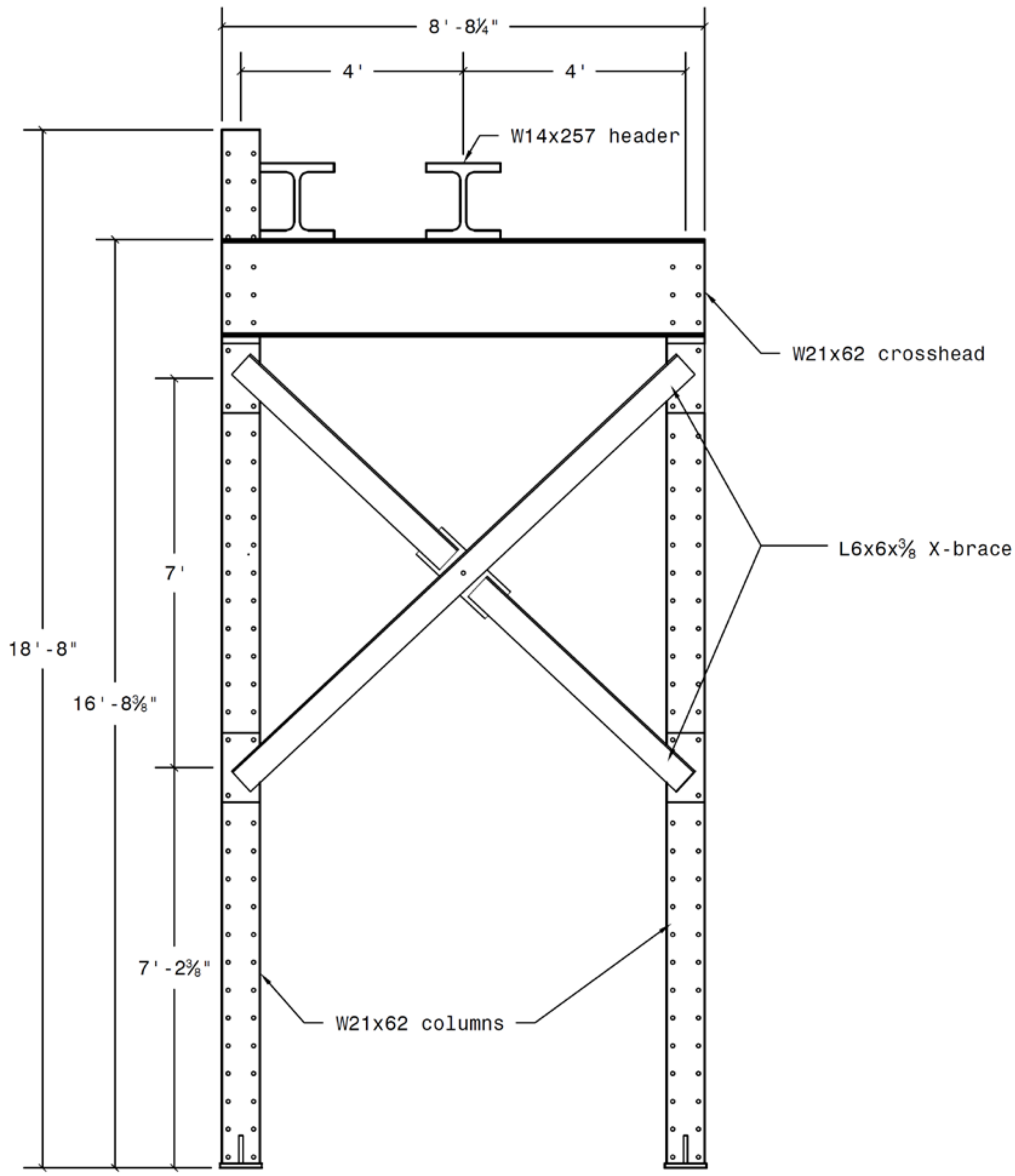
REVISIONS:

NO.	DATE	BY

DESIGNED BY:
DRAWN BY: R. Stevens
CHECKED BY:
DATE: 7/1/2020
PROJECT NO.:

SHEET TITLE:
Moment connection test setup

SHEET NO.: 1 of 9



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PROJECT:

Wrap-Around Welds on End-Plate Stiffeners

SHEET NOTES:

1. Scale is 3/8"=1'
2. MTS 201.7 actuator not shown

REVISIONS:

NO.	DATE	BY

DESIGNED BY:

DRAWN BY: R. Stevens

CHECKED BY:

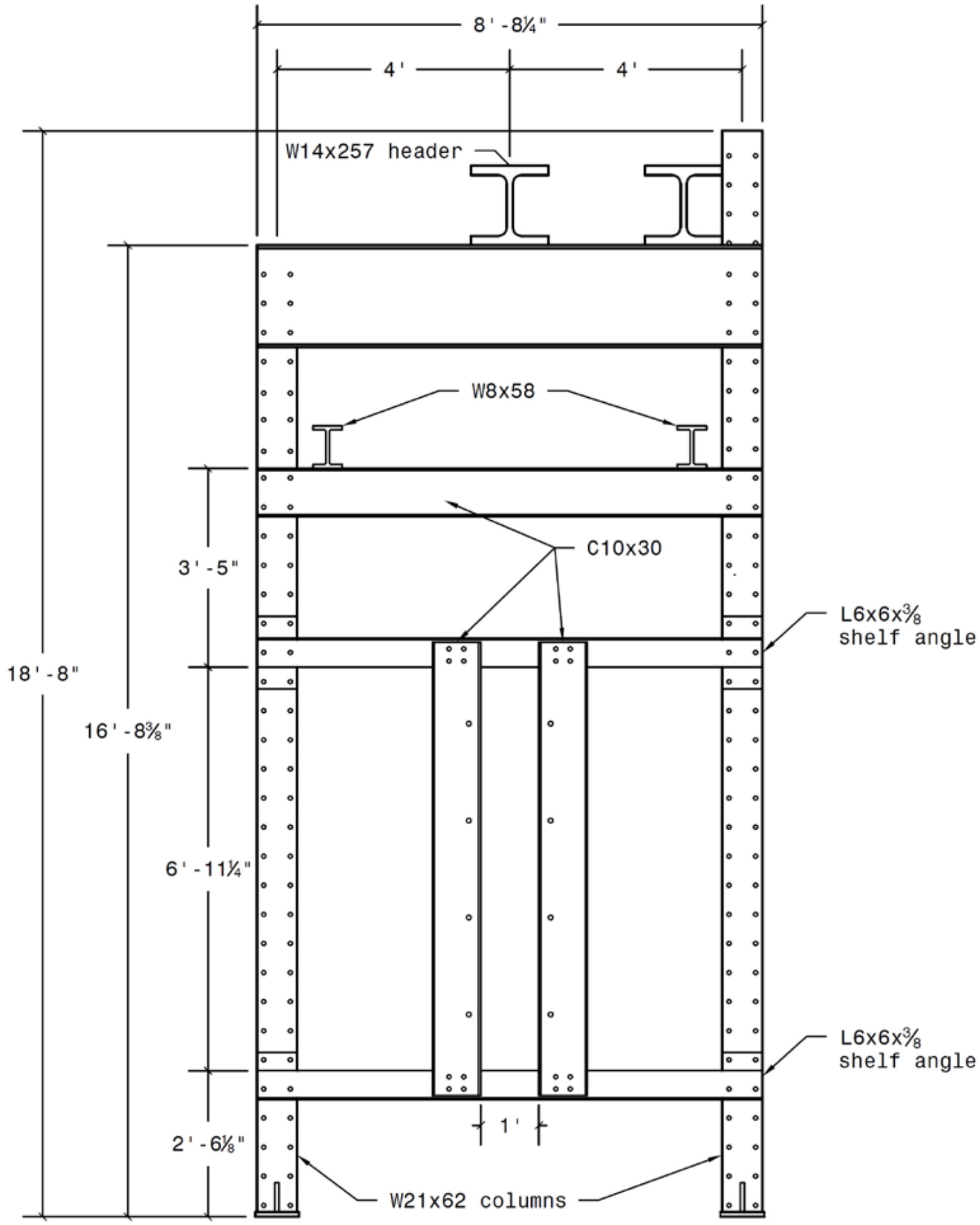
DATE: 7/1/2020

PROJECT NO.:

SHEET TITLE:

South elevation of actuator frame

SHEET NO.: 2 of 9



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PROJECT:
Wrap-Around Welds on End-Plate Stiffeners

SHEET NOTES:
1. Scale is 3/8"=1'
2. MTS 201.7 actuator not shown

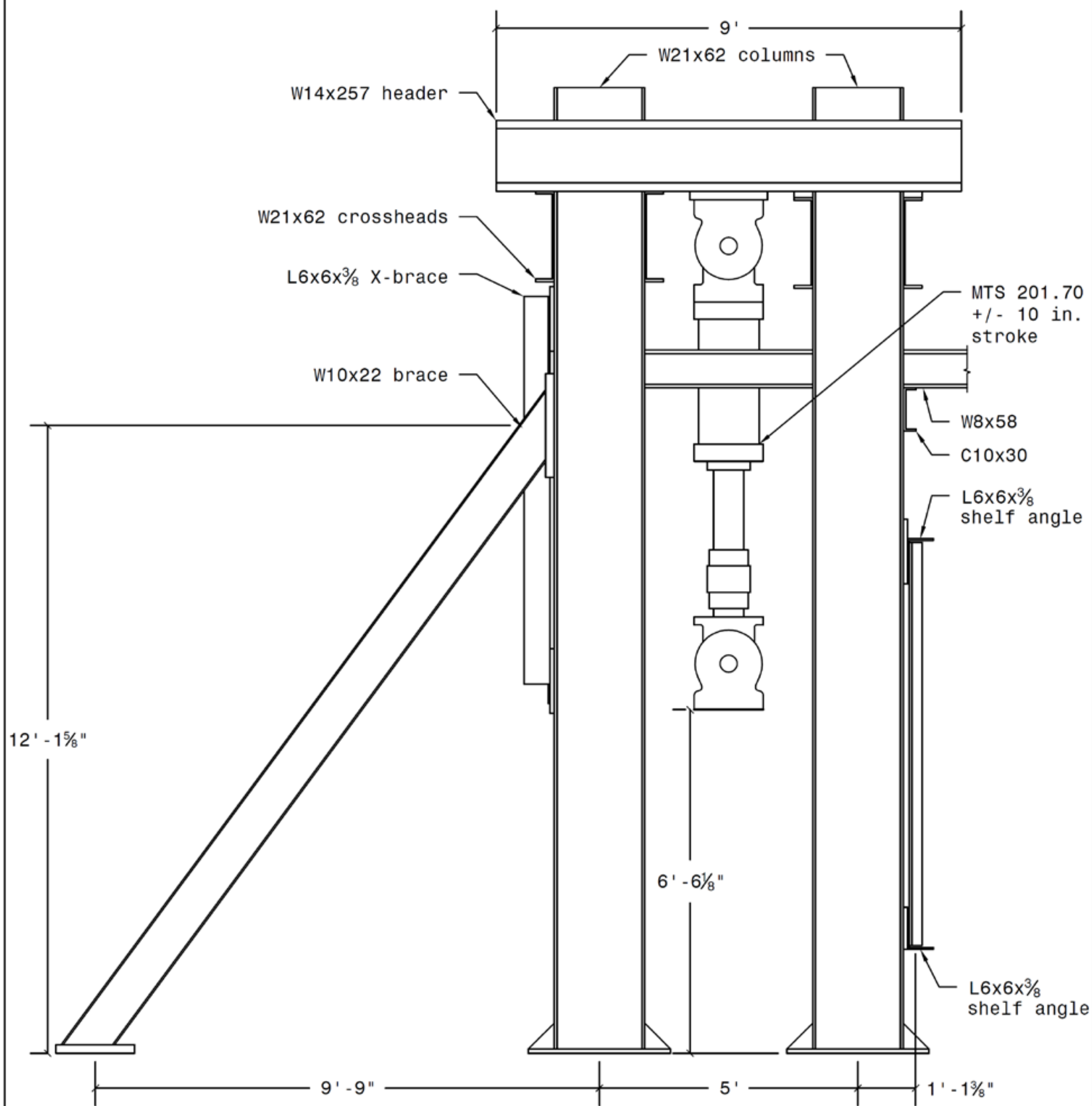
REVISIONS:

NO.	DATE	BY

DESIGNED BY:
DRAWN BY: R. Stevens
CHECKED BY:
DATE: 7/1/2020
PROJECT NO.:

SHEET TITLE:
North elevation of actuator frame

SHEET NO.: 3 of 9



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ENGINEERING
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BLACKSBURG, VA 24061
P: 540-231-6635

PROJECT:
Wrap-Around Welds on End-Plate Stiffeners

SHEET NOTES:
1. Scale is 3/8"=1'

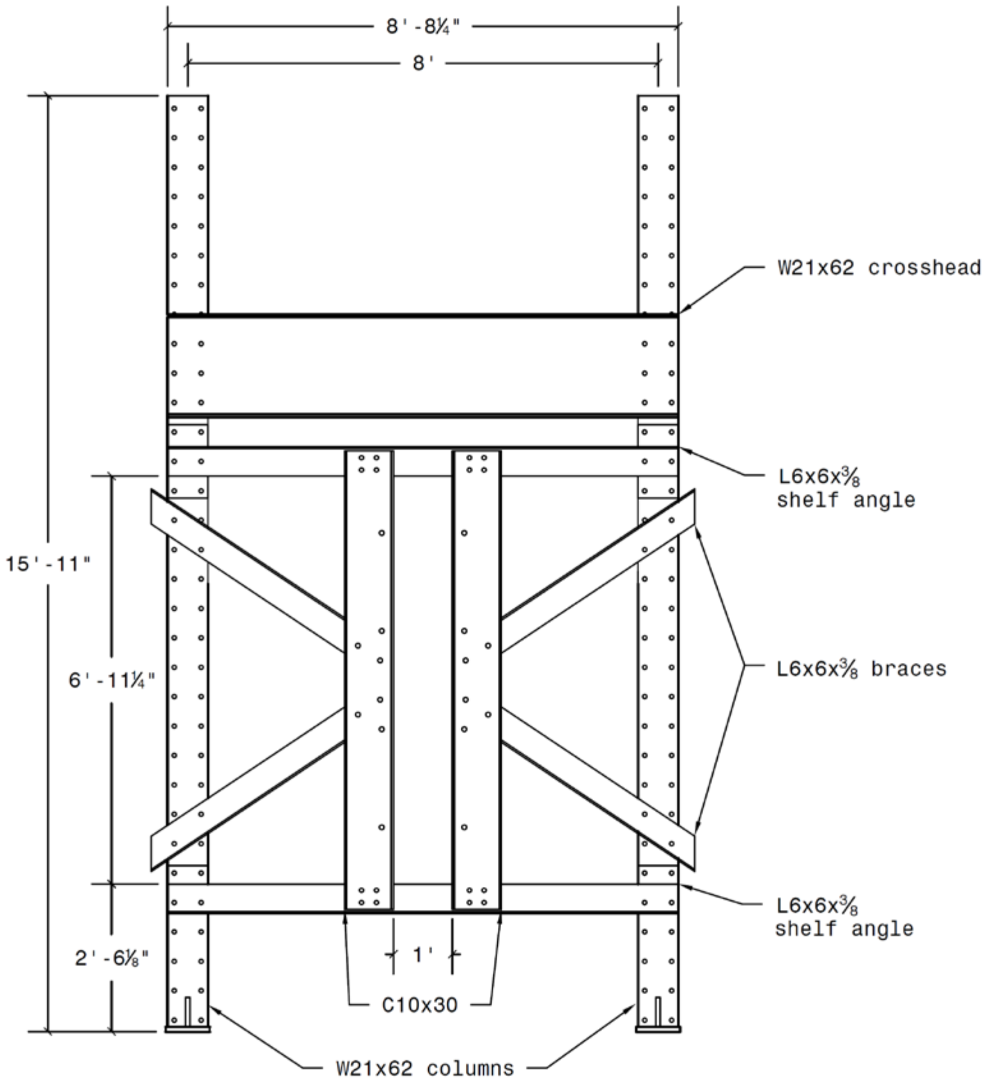
REVISIONS:

NO.	DATE	BY

DESIGNED BY:
DRAWN BY: R. Stevens
CHECKED BY:
DATE: 7/1/2020
PROJECT NO.:

SHEET TITLE:
East elevation of actuator frame

SHEET NO.: 4 of 9



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PROJECT:

Wrap-Around Welds on End-Plate Stiffeners

SHEET NOTES:

1. Scale is 3/8"=1'
2. W8x58 not shown

REVISIONS:

NO.	DATE	BY

DESIGNED BY:

DRAWN BY: R. Stevens

CHECKED BY:

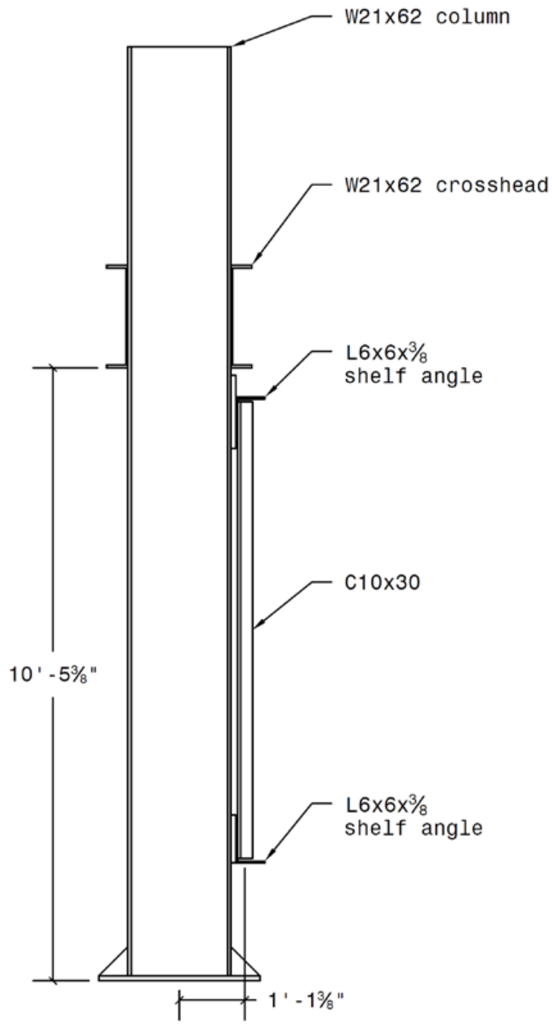
DATE: 7/1/2020

PROJECT NO.:

SHEET TITLE:

North elevation of lateral-torsional buckling bracing frame

SHEET NO.: 5 of 9



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PROJECT:
Wrap-Around Welds on End-Plate Stiffeners

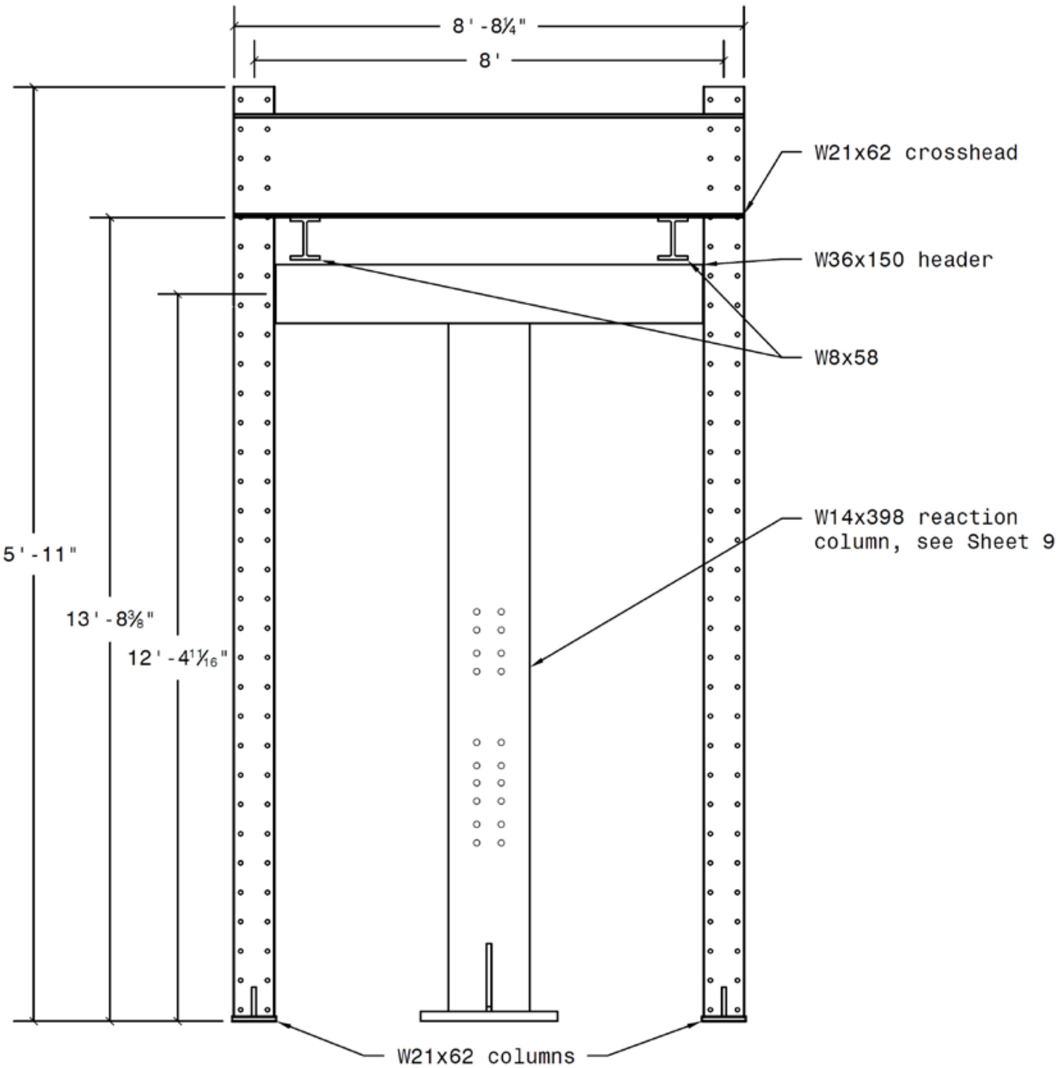
SHEET NOTES:
1. Scale is 3/8"=1'
2. W8x58 not shown

REVISIONS:		
NO.	DATE	BY

DESIGNED BY:	
DRAWN BY:	R. Stevens
CHECKED BY:	
DATE:	7/1/2020
PROJECT NO.:	

SHEET TITLE:
East elevation of lateral-torsional buckling bracing frame

SHEET NO.: 6 of 9



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P: 540-231-6635

PROJECT:
Wrap-Around Welds on End-Plate Stiffeners

SHEET NOTES:
1. Scale is 3/8"=1'

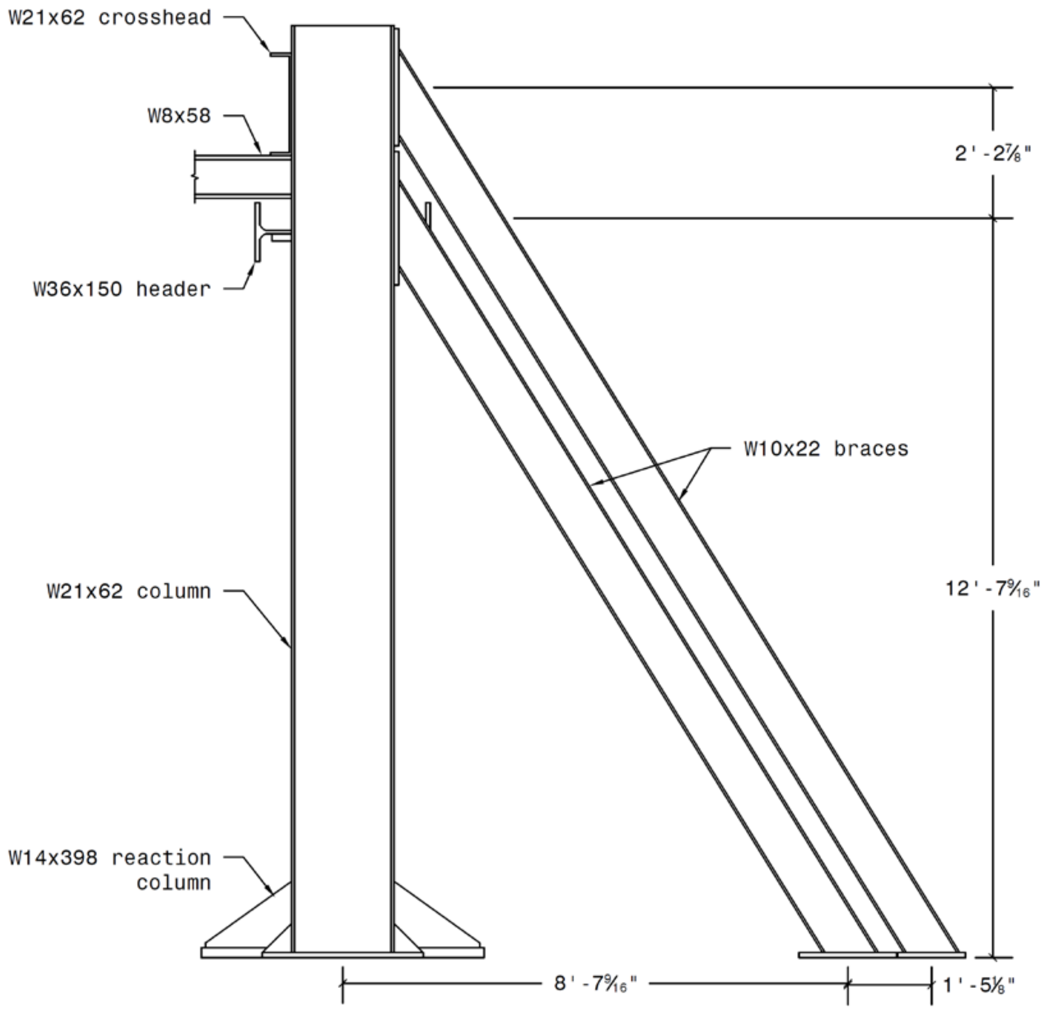
REVISIONS:

NO.	DATE	BY

DESIGNED BY:
DRAWN BY: R. Stevens
CHECKED BY:
DATE: 7/1/2020
PROJECT NO.:

SHEET TITLE:
South elevation of column frame

SHEET NO.: 7 of 9



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PROJECT:
Wrap-Around Welds on End-Plate Stiffeners

SHEET NOTES:
1. Scale is 3/8"=1'

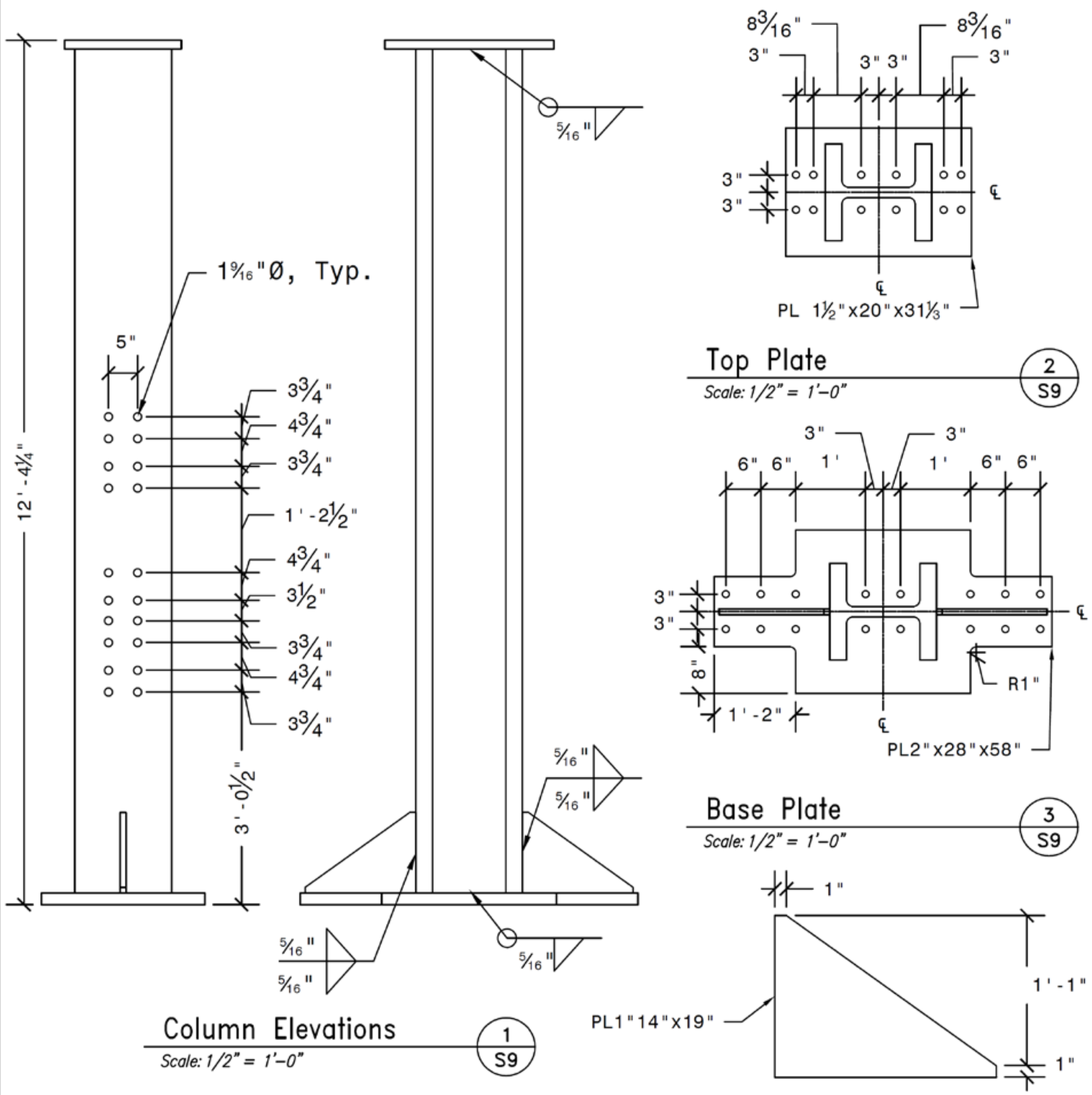
REVISIONS:

NO.	DATE	BY

DESIGNED BY:
DRAWN BY: R. Stevens
CHECKED BY:
DATE: 7/1/2020
PROJECT NO.:

SHEET TITLE:
East elevation of column frame

SHEET NO.: 8 of 9



ADDITIONAL SHEET NOTES:
1. Holes in the column flange are centered horizontally on the flange.
2. Base plate and top plate holes are 1 1/16" Ø.

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PROJECT:
Wrap-Around Welds on End-Plate Stiffeners

SHEET NOTES:
1. The column shall be centered on the base plate and top plate in both directions.
2. Holes in the column are only drilled on one flange, as shown.

REVISIONS:

NO.	DATE	BY

DESIGNED BY:
DRAWN BY: R. Stevens
CHECKED BY:
DATE: 7/1/2020
PROJECT NO.:

SHEET TITLE:
W14x398 reaction column

SHEET NO.: 9 of 9

APPENDIX C - STORY DRIFT DECOMPOSITION PROCEDURE

This appendix presents equations for decomposing the applied story drift into components due to panel zone shear, column flexure, end-plate separation, elastic deformation, and plastic deformation. This is necessary because the second qualification criteria requires that the actual inelastic rotations be within 25% of the anticipated inelastic rotations in the prototype connections (AISC 2016b). Unless otherwise stated, these equations are adapted from Toellner (2013) and Szabo (2017).

C.1 Panel Zone Shear

The calculations for story drift due to panel zone shear and rigid body rotation of the panel zone are adapted from Uang and Bondad (1996). The average panel zone shear strain, γ, was first calculated using Eq. (C.1),

$$\gamma = \frac{\sqrt{(d_c - t_{cf})^2 - (d_b - t_{bf})^2}}{2(d_c - t_{cf})(d_b - t_{bf})} \tag{C.1}$$

where: d_b = beam depth

d_c = column depth

t_{bf} = beam flange thickness

t_{cf} = column flange thickness

δ_{LP_01} = displacement of LP_01

δ_{LP_02} = displacement of LP_02.

Equation (C.1) assumes that the end of LP_01 and LP_02 are placed at the intersections of the centerlines of the beam and column flanges. The deflection at the loading point, δ_{PZ}, due to panel zone shear was calculated using Eq. (C.2), and the story drift due to panel zone shear, θ_{PZ}, was calculate using Eq. (C.3).

$$\delta_{PZ} = \gamma\left(L_{cl} - \frac{d_c}{2}\right) - \frac{\gamma d_b}{h_{col}} L_{cl} \tag{C.2}$$

$$\theta_{PZ} = \frac{\delta_{PZ}}{L_{cl}} \tag{C.3}$$

where: d_b = beam depth

d_c = column depth

h_{col} = distance between SP_01 and SP_08

L_{cl} = distance from the actuator centerline to the column centerline.

C.2 Column Flexure

Column deformations include rigid body rotations of the frame, panel zone shear, and column flexure. While rigid body rotation was accounted for during testing using the active feedback control, it needs to be removed again to isolate the rotation due to column flexure. The story drift due to rigid body rotation of the column, θ_{RB}, was calculated using Eq. (C.4). The story drift due to the column flexure, θ_{CF}, was then calculated using Eq. (C.5).

$$\theta_{RB} = \frac{\delta_{SP\_01} - \delta_{SP\_08}}{h_{col}} \tag{C.4}$$

$$\theta_{CF} = \frac{\delta_{SP\_02} - \delta_{SP\_03}}{L_{sp1}} - \theta_{RB} - \theta_{PZ} \tag{C.5}$$

where: h_{col} = distance between SP_01 and SP_08

L_{sp1} = distance between SP_02 and SP_03

δ_{SP_01} = displacement of SP_01

δ_{SP_02} = displacement of SP_02

δ_{SP_03} = displacement of SP_03

δ_{SP_08} = displacement of SP_08.

C.3 End-plate separation

The bolts were pretensioned when the test specimens were bolted to the reaction column, but during testing the moment created in the plastic hinge can create bolt forces that exceed the pretension. If this occurs, the end-plate with separate from the column flange, which contributes to the applied drift. This separation was measured by the spring calipers CLP_01 and CLP_02, placed at the centerlines of the top and bottom beam flanges. The story drift due to the end-plate separation, θ_{EP}, was calculated using Eq. (C.6),

$$\theta_{EP} = \frac{-\delta_{CLP}}{d_b - t_{bf}} \tag{C.6}$$

where: d_b = beam depth

t_{bf} = beam flange thickness

δ_{CLP} = displacement measured by the caliper.

Equation (C.6) assumes that small-angle theory applies, and that the beam is rotating about the centerline of the opposite flange.

C.4 Elastic Beam Deformation

Outside of the plastic hinge region, the beam is deforming elastically, which has flexure and shear components. The shear component was calculated by Timoshenko (1955). The elastic deflection at the loading point, δ_{el}, was calculated using Eq. (C.8), which required the shape factor α from Eq. (C.7). The story drift due to the elastic beam deformation outside the plastic hinge region, θ_{EL}, was then calculated using Eq. (A.9).

$$\alpha = \frac{A}{8I_x t_w}\left(b_f {d_b}^2 - b_f {h_w}^2 + t_w {h_w}^2\right) \tag{C.7}$$

$$\delta_{el} = \frac{P{L_{el}}^3}{3EI_x} + \frac{PL_{el}\alpha}{AG} \tag{C.8}$$

$$\theta_{EL} = \frac{\delta_{el}}{L_{cl}} \tag{C.9}$$

where: A = cross-sectional area

b_f = flange width

d_b = beam depth

E = modulus of elasticity

G = shear modulus

h_w = clear distance between flanges

I_x = moment of inertia of the beam

L_{cl} = distance from the actuator centerline to the column centerline

L_{el} = distance from SP_04 to the actuator centerline

P = applied load

t_w = web thickness.

C.5 Plastic Beam Deformation

In these tests, deformation in the plastic hinge region was expected to be the greatest story drift component. To measure this rotation, string potentiometers SP_06 and SP_07 were attached to the column flange above and below the beam. The end of the string potentiometers were attached to the beam at the same location as SP_04. The rotation due to deformation in the plastic hinge region, θ_{PH}, was then calculated using Eq. (C.10),

$$\theta_{PH} = \frac{\delta_{SP\_06} - \delta_{SP\_07}}{L_{sp2}} - \theta_{EP} \tag{C.10}$$

Where: L_{sp2} = distance between SP_06 and SP_07

δ_{SP_06} = displacement of SP_06

δ_{SP_07} = displacement of SP_07.

It is important to note that Eq. (C.10) also includes elastic beam rotation. Thus, in the early cycles when no plastic deformation is expected, θ_{PH} will be nonzero.

C.6 Total Story Drift

The total story drift is calculated using Eq. (C.11).

$$\theta_{total} = \theta_{CF} + \theta_{PZ} + \theta_{EP} + \theta_{EL} + \theta_{PH} \quad \text{(C.11)}$$

If the calculated total story shear is checked against the applied story drift, discrepancies will be found. This is due to simplifying assumptions made in the decomposition and drift components that were not captured by this instrumentation plan.

APPENDIX D - DATA ADJUSTMENTS

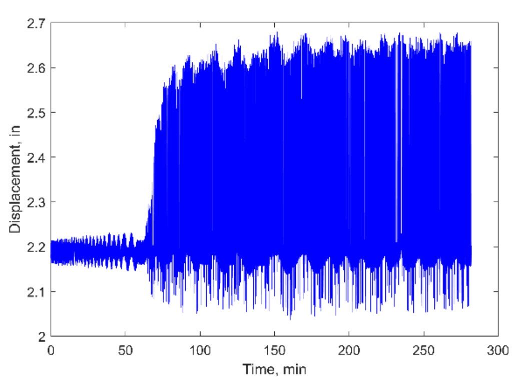
This appendix describes adjustments made to the test data before it was used for the story drift decomposition. Data adjustments fell into two categories: general modifications applied to all data from all tests and specific adjustments for times when the instrumentation was disturbed.

General adjustments included removing any data recorded after fracture occurred and shifting all sensor measurements to zero, except for the force measurement, which was handled separately for each test. At the start of each test, the actuator was moved until the load cell measured a force equal to the weight of the actuator clevis plus half of the specimen weight. This removed any load on the connection that would not be present in a real structure, where the self-weight of the beam is distributed equally between the connections at either end. The initial force was then used in the data analysis as a load adjustment factor when zeroing the force measurements. The load adjustment factor was 2.9 kips for W24×76 beams and 3.9 kips for W36×150 beams.

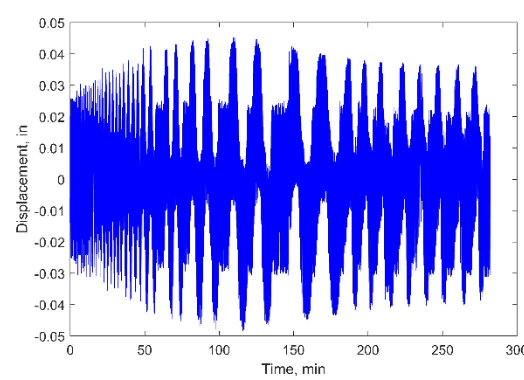
D.1 Specimen 4ES-1.375-1.25-24sides

Specific data adjustments for this specimen involved:

1. Adjusted LP_01 data to account for noise in the sensor data that was not attributed to the panel zone deformation. The unadjusted and adjusted plots are shown in Figure D.1.
2. Adjusted LP_02 data to account for noise in the sensor data that was not attributed to the panel zone deformation. The unadjusted and adjusted plots are shown in Figure D.2.

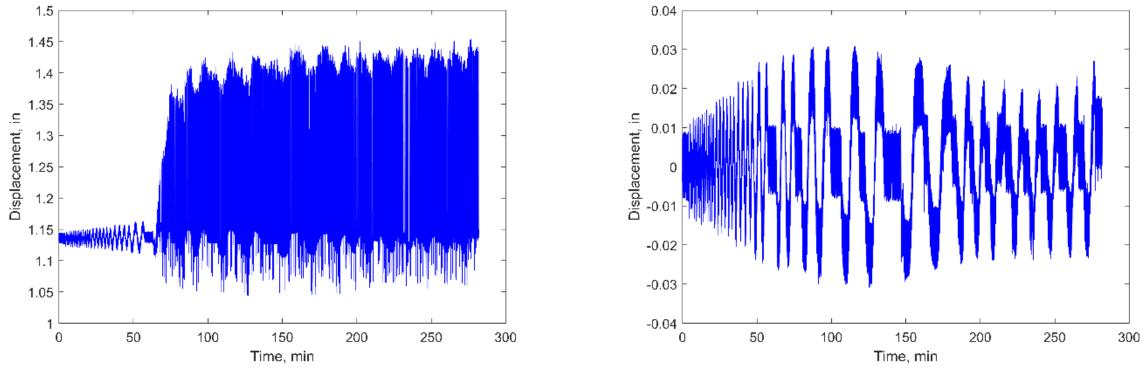


(a) Unadjusted



(b) Adjusted

Figure D.1. LP_01 adjustment for 4ES-1.375-1.25-24sides



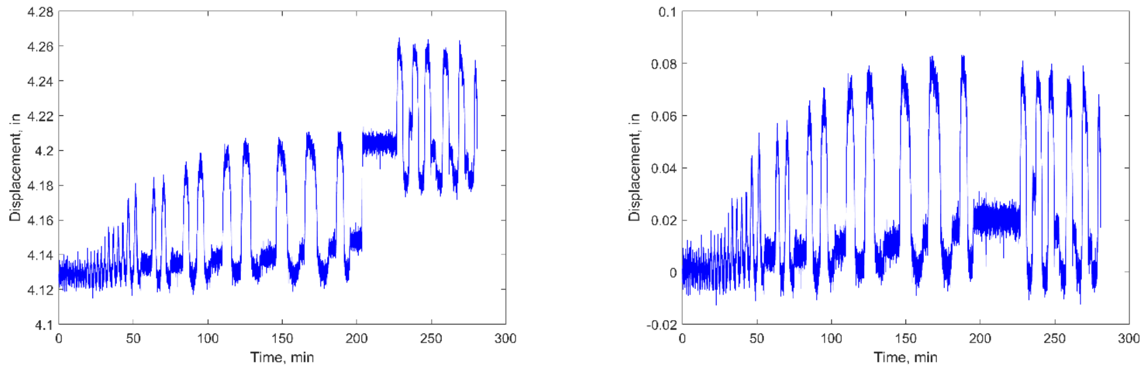
(a) Unadjusted (b) Adjusted

Figure D.2. LP_02 adjustment for 4ES-1.375-1.25-24sides

D.2 Specimen 4ES-1.375-1.25-24wrap

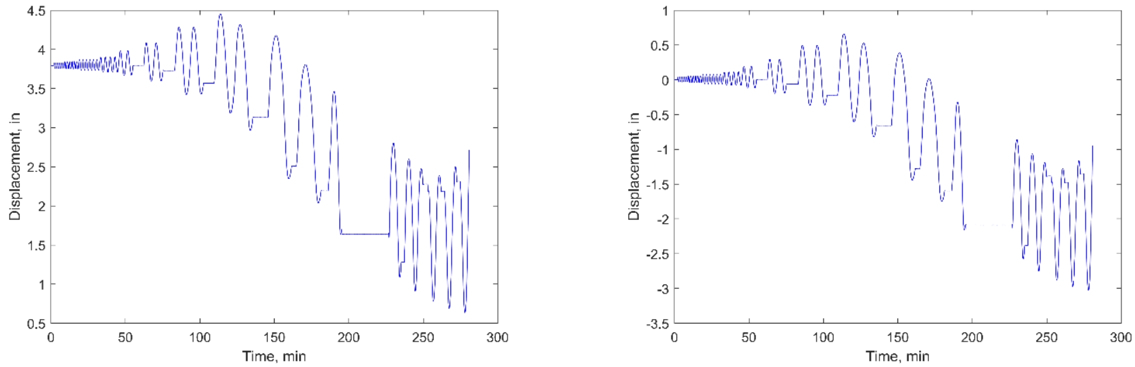
Specific data adjustments for this specimen involved:

1. Adjusted CLP_01 data to account for when the caliper was bumped during the test. The unadjusted and adjusted plots are shown in Figure D.3.
2. Adjusted SP_06 data to account for noise that was not attributed to deformation in the plastic hinge. The unadjusted and adjusted plots are shown in Figure D.4.



(a) Unadjusted (b) Adjusted

Figure D.3. CLP_01 adjustment for Specimen 4ES-1.375-1.25-24wrap



(a) Unadjusted (b) Adjusted

Figure D.4. SP_06 adjustment for Specimen 4ES-1.375-1.25-24wrap

D.3 Specimen 8ES-1.375-1.5-36sides

There were no specific data adjustments for this specimen.

D.4 Specimen 8ES-1.375-1.5-36wrap

There were no specific data adjustments for this specimen.

APPENDIX E – MILL CERTIFICATION REPORTS

Customer Name	Customer PO#	Invoice No	Shipper No	Heat Number
AMERICAN INSTITUTE OF STEEL COI	ENG052919A	1514481	1479480	60122456

GERDAU

US-ML-PETERSBURG
25801 HOFHEIMER WAY
PETERSBURG, VA 23803-8905
USA

CERTIFIED MATERIAL TEST REPORT

Page 1 1

CUSTOMER SHIP TO	CUSTOMER BILL TO	GRADE	SHAPE / SIZE	DOCUMENT ID
INFRA METALS CO 1601 BROADWAY ST MARSEILLES,IL 61341-9326 USA	INFRA METALS CO 55 PENT HWY WALLINGFORD,CT 06492 USA	A992/A572-50	Wide Flange Beam 36 X 150# 920 X 223	0000000000

LENGTH	PCS	WEIGHT	HEAT BATCH
30'00"	2	9,000 LB	60122456/07

SALES ORDER	CUSTOMER MATERIAL N°	SPECIFICATION / DATE or REVISION
5908237/000030		ASTM A6-14 ASTM A709-15 ASTM A992-11 (2015) A572-15 CSA G40 21-13 345WM

CUSTOMER PURCHASE ORDER NUMBER	BILL OF LADING	DATE
CE-545880	1330-0000095543	01/07/2018

CHEMICAL COMPOSITION

C %	Mn %	P %	S %	Si %	Cu %	Ni %	Cr %	Mo %	Sn %	V %	Nb %	Al %
0 08	1 21	0 016	0 037	0 19	0 38	0 13	0 10	0 030	0 015	0 002	0 017	0 003

CHEMICAL COMPOSITION

CEqvA6 %
0 34

MECHANICAL PROPERTIES

YS 0.2% PSI	UTS PSI	YS MPa	UTS MPa	Y/T ratio %	G/L Inch
53100	69100	366	476	0 770	8 000
53700	69200	370	477	0 780	8 000

MECHANICAL PROPERTIES

G/L mm	Elong %
200 0	28 60
200 0	29 30

COMMENTS NOTES

The above figures are certified chemical and physical test records as contained in the permanent records of company We certify that these data are correct and in compliance with specified requirements This material, including the billets, was melted and manufactured in the USA CMTR complies with EN 10204 3 1

Bhaskar

BHASKAR YALAMANCHILI
QUALITY DIRECTOR

Phone (409) 769 1014 Email Bhaskar Yalamanchili@gerdau.com

Alice L Pitchford

ALICE PITCHFORD
QUALITY ASSURANCE MGR

Phone (804) 524-2851 Email Alice pitchford@gerdau.com

Heat Number	Shipper No	Invoice No	Customer PO#	Customer Name
60125226	1479480	1514481	ENG052919A	AMERICAN INSTITUTE OF STEEL COI

GERDAU

US-ML-PETERSBURG
25801 HOFHEIMER WAY
PETERSBURG, VA 23803-8905
USA

CERTIFIED MATERIAL TEST REPORT

Page 1/1

CUSTOMER SHIP TO	CUSTOMER BILL TO	GRADE	SHAPE / SIZE	DOCUMENT ID.
INFRA METALS CO 1601 BROADWAY ST MARSEILLES,IL 61341-9326 USA	INFRA METALS CO 55 PENT HWY WALLINGFORD,CT 06492 USA	A992/A572-50	Wide Flange Beam / 24 X 76# / 610 X 113	0000000000

LENGTH	PCS	WEIGHT	HEAT / BATCH
50'00"	0	34,200 LB	60125226/02

SALES ORDER	CUSTOMER MATERIAL N°
6872156/000020	

CUSTOMER PURCHASE ORDER NUMBER	BILL OF LADING	DATE
CE-559764	1330-0000107253	08/20/2018

SPECIFICATION / DATE or REVISION
ASTM A6-17
ASTM A709-17
ASTM A992-11 (2015), A572-15
CSA G40.21-13 345WM

CHEMICAL COMPOSITION

C %	Mn %	P %	S %	Si %	Cu %	Ni %	Cr %	Mo %	Sn %	V %	Nb %	Al %
0.08	1.07	0.012	0.034	0.21	0.27	0.12	0.11	0.020	0.008	0.002	0.015	0.004

CHEMICAL COMPOSITION

CEqyA6 %
0.31

MECHANICAL PROPERTIES

YS 0.2% PSI	UTS PSI	YS MPa	UTS MPa	Y/T rati %	G/L Inch
55400	69400	382	479	0.800	8.000
56100	70300	387	485	0.800	8.000

MECHANICAL PROPERTIES

G/L mm	Elong. %
200.0	30.10
200.0	30.10

COMMENTS / NOTES

The above figures are certified chemical and physical test records as contained in the permanent records of company. We certify that these data are correct and in compliance with specified requirements. This material, including the billets, was melted and manufactured in the USA. CMTR complies with EN 10204 3.1.

BHASKAR YALAMANCHILI
QUALITY DIRECTOR
Phone: (409) 267-1071 Email: Bhaskar.Yalamanchili@gerdau.com

ALICE PITCHFORD
QUALITY ASSURANCE MGR.
Phone: (804) 524-2851 Email: Alice.pitchford@gerdau.com

SSAB

Test Certificate

1770 Bill Sharp Boulevard, Muscatine, IA 52761-9412, US

Form TC1: Revision 3: Date 7 Feb 2018

Customer:
INFRA-METALS STEEL CORP
1601 BROADWAY STREET

MARSEILLES
IL 61341

Customer P.O.No.: CE-560705
Mill Order No. 41-552172-05
Shipping Manifest: MT359558
Product Description: ASTM A572-50/M345(18)/A709-50/M345(17)
Ship Date: 26 Oct 18
Cert Date: 26 Oct18
Cert No: 061737552
(Page 1 of 1)

Size: 1.250 X 120.0 X 480.0 (IN)

Tested Pieces:			Tensiles:								Charpy Impact Tests												
Heat Id	Piece Id	Piece Dimensions	Tst Loc	YS (KSI)	UTS (KSI)	%RA	Elong % 2in	Elong % 8in	Tst Dir	Hardness	Abs. Energy(FTLB) 1	2	3	Avg	% Shear 1	2	3	Avg	Tst Tmp	Tst Dir	Tst Siz (mm)	BDWTT Tmp	%Shr
B8I630	D05	1.250 (DISCRT)	L	56	70			29	T														

Chemical Analysis

Heat Id	C	Mn	P	S	Si	Tot Al	Cu	Ni	Cr	Mo	Cb	V	Ti
B8I630	.06	1.31	.014	.001	.24	.032	.29	.12	.16	.02	.027	.039	.002

ORGN
USA

KILLED STEEL
MERCURY IS NOT A METALLURGICAL COMPONENT OF THE STEEL AND NO MERCURY WAS INTENTIONALLY ADDED DURING THE MANUFACTURE OF THIS PRODUCT.
MTR EN 10204:2004 INSPECTION CERTIFICATE 3.1 COMPLIANT
100% MELTED AND MANUFACTURED IN THE USA.
PRODUCTS SHIPPED:
B8I630 D05 PCES: 2, LBS: 40838

Cust Part #:

WE HEREBY CERTIFY THAT THIS MATERIAL WAS TESTED IN ACCORDANCE WITH, AND MEETS THE REQUIREMENTS OF THE APPROPRIATE SPECIFICATION

Brian Wales
SENIOR METALLURGIST - PRODUCT

Customer Name	Customer PO#	Invoice No	Shipper No	Heat Number
AMERICAN INSTITUTE OF STEEL COI	ENG052919A	1514482	1479485	B8I630

Customer Name	Customer PO#	Invoice No	Shipper No	Heat Number
AMERICAN INSTITUTE OF STEEL CO	ENG052919A	1514482	1479485	A8K298

SSAB

1770 Bill Sharp Boulevard, Muscatine, IA 52761-9412, US

Test Certificate

Form TC1 Revision 3 Date 7 Feb 2018

Customer:
INFRA-METALS STEEL CORP
1601 BROADWAY STREET

MARSEILLES
IL 61341

Customer P.O.No.: CE-563782

Mill Order No. 41-556715-07

Shipping Manifest: MR366565

Product Description: ASTM A36(14)/A709(18)36/ASME SA36(17)
AASHTO M270(15)36

Ship Date: 18 Jan 19
Cert Date: 18 Jan19

Cert No: 061752638
(Page 1 of 1)

Size: 1 500 X 96 00 X 240 0 (IN)

Heat Id	Piece Id	Piece Dimensions	Tst Loc	YS (KSI)	UTS (KSI)	%RA	Elong % 2in	Elong % 8in	Tst Dir	Hardness	Abs. Energy(FTLB) 1	2	3	Avg	% Shear 1	2	3	Avg	Tst Tmp	Tst Dir	Tst Siz (mm)	BDWTT Tmp %Shr
A8K298	A22	1 507 (DISCRT)	L	50	75			30	T													

Chemical Analysis

Heat Id	C	Mn	P	S	Si	Tot Al	Cu	Ni	Cr	Mo	Cb	V	Ti	B	N	ORGN
A8K298	17	1 24	010	001	19	028	28	14	10	03	001	037	008	0003	0061	USA

KILLED STEEL
MERCURY IS NOT A METALLURGICAL COMPONENT OF THE STEEL AND NO MERCURY WAS INTENTIONALLY ADDED DURING THE MANUFACTURE OF THIS PRODUCT.
' WARNING: THIS PRODUCT CAN EXPOSE YOU TO CHEMICALS INCLUDING NICKEL AND NICKEL COMPOUNDS, WHICH ARE KNOWN TO THE STATE OF CALIFORNIA TO CAUSE CANCER.
FOR MORE INFORMATION GO TO WWW.P65WARNINGS.CA.GOV.
MTR EN 10204:2004 INSPECTION CERTIFICATE 3.1 COMPLIANT
100% MELTED AND MANUFACTURED IN THE USA.
PRODUCTS SHIPPED:
A8K298 A22 PCES: 2, LBS: 19602

(P) Cust Part #

WE HEREBY CERTIFY THAT THIS MATERIAL WAS TESTED IN ACCORDANCE WITH AND MEETS THE REQUIREMENTS OF THE APPROPRIATE SPECIFICATION

Brian Wales
SENIOR METALLURGIST - PRODUCT

Customer Name	Customer PO#	Invoice No	Shipper No	Heat Number
AMERICAN INSTITUTE OF STEEL COI	ENG052919A	1514482	1479485	W2650

Report - Of - Information

FERALLOY PROCESSING COMPANY

OWNER:
INFRA-METALS CO.
1601 BROADWAY ST.
MARSEILLES , IL 61341

SHIP TO:
INFRA-METALS
1601 BROADWAY ST.
MARSEILLES , IL 61341

DATE: 2/19/19

BILL OF LADING: 680 - 229307 - 10
HEAT/MILL COIL: W2650 5232306
SKID NO: 545350
TAG NUMBER: 101781 - 01 NLMK
PROCESSED AS: HR Structural Steel, Grade 50
REFERENCING: ASTM A-572 HSLA Cb-V Structural Steel

PART NO: -
SIZE: .5000 X 60.000 X 120.000
PRODUCT: HR S50 A572
CUSTOMER PO NUMBER: 566696

Element	C	Mn	P	S	Si	Cu	Ni	Cr	Mo	Al	Fe	Sb
Weight %	.0500	.9300	.0150	.0070	.0200	.1700	.0500	.0900	.0200	.0350		
Element	Nb	V	Ti	Zr	B	CA	N	H	O	Sn	Zn	Mg
Weight %	.0010	.0900	.0010		.0002	.0023	.0080			.0200		

Carbon Equiv.(Calc'd): .2596 (IIW) .1223 (AWS D1.1) .2629 (AWS) Corrosion Ind: 3.9700 G101(6.3.1)

Tensile Properties TTM: Toughness(Charpy)

Loc/Dir	YS-(PS	TS-(PS	EL-%	NVal	Y/T	ROA%		Energy	Grain Size
HD	65800	73900	33.0		89.0		Coupon:		
MD	66100	73200	31.0		90.3		Dir'n :		Bend Test
TL							Temp. :		
ML							Units :		Hardness/Scale
								Avg	

The values published in this 'Report-of-Information' are transcribed from information provided by the owner and the owner's suppliers including mills, testing laboratories, etc. Feralloy makes no representation or warrantee based on this information unless it is generated, reviewed and authorized by Feralloy agents.

This material processed at & shipped from:

FERALLOY PROCESSING CO
FERALLOY PROCESSING CO
600 GEORGE NELSON DRIVE
PORTAGE , IN 46368

Heat Number 9501627
Shipper No 1479485
Invoice No 1514482
Customer PO# ENG052919A
Customer Name AMERICAN INSTITUTE OF STEEL CO

NUCOR PLATE MILL

P.O.Box 279
Winton, NC 27986
(252) 356-3700

Mill Test Report

Page 4

1505 River Rd
Cofield, NC 27922
(252) 356-3700

NUCOR It's Our Nature

Issuing Date : 03/25/2019 B/L No. : 529504 Load No : 540551 Our Order No. : 164030/4 Cust. Order No. : CE-565785

Vehicle No: ALY 91698

Specification: 0.6250" x 96.000" x 240.000"
ASTM A572 Grade 50/345-18/A709 Grade 50-18/ AASHTO M270-2017 50
97 Type 2

Sold To: INFRA METALS CO CENTRAL DIVISION
1601 BROADWAY
MARSEILLES,IL 61341

Ship To: INFRA METALS CO CENTRAL DIVISION
1601 BRDWAY
815-795-5041 FAX TO KYLE
MARSEILLES,IL 61341

Marking :

Heat No	C	Mn	P	S	Si	Cu	Ni	Cr	Mo	Al(tot)	V	Nb	Ti	N	Ca	B	Sn	Ceq	Pcm
9501627	0 19	0 93	0 011	0 001	0 21	0 18	0 06	0 08	0 01	0 018	0 026	0 002	0 002		0 0023	0 0001	0 006	0 38	0 26

Tensile Test

Plate Serial No	Pieces	Tons	Dir	(psi) Yield	(psi) Tensile	Elong % in 2"	Elong % in 8"
9501627-09	2	4 08	T	54,100	77,100		20 4
			T	55,100	77 700		20 6

Manufactured to fully killed fine grain practice by Electric Arc Furnace Welding or weld repair was not performed on this material Mercury has not been used in the direct manufacturing of this material Produced as continuous cast discrete plate as-rolled unless otherwise noted in Specification For Mexico shipments nhc-SalesMX@Nucor com
Yield by 0 5EUL method unless otherwise specified Ceq = C+(Mn/6)+((Cr+Mo+V)/5)+((Cu+Ni)/15)
Pcm = C+(Si/30)+(Mn/20)+(Cu/20)+(Ni/60)+(Cr/20)+(Mo/15)+(V/10)+5B
Melted and Manufactured in the USA ISO 9001 2008 certified (#010940) by SRI Quality System Registrar (#0985-09) PED 97/23/EC 7/2 Annex 1 Para 4 3 Compliant DIN 50049 3 1 B/EN 10204 3 1B(2004) DIN EN 10204 3 1(2005) compliant For ABS grades only Quality Assurance certificate 14-MMPQA-723

We hereby certify that the contents of this report are accurate and correct All test results and operations performed by the material manufacturer are in compliance with the applicable specifications including customer specifications

T. A. Depretis

T A Depretis Metallurgist

3/25/2019 12 52 25 PM

Heat Number	Shipper No	Invoice No	Customer PO#	Customer Name
9501292	1479485	1514482	ENG052919A	AMERICAN INSTITUTE OF STEEL COI

NUCOR
PLATE MILL

P.O.Box 279
Winton, NC 27986
(252) 356-3700

Mill Test Report

Page 10

1505 River Rd
Cofield, NC 27922
(252) 356-3700

NUCOR
It's Our Nature

Issuing Date : 03/25/2019 B/L No. : 529504 Load No. : 540551 Our Order No. : 164030/10 Cust. Order No. : CE-565785

Vehicle No. ALY 91698

Specification: 1.0000" x 120.000" x 480.000"
ASTM A572 Grade 50/345-18/A709 Grade 50-18/ AASHTO M270-2017 50
97 Type 2

Sold To: INFRA METALS CO CENTRAL DIVISION
1601 BROADWAY
MARSEILLES,IL 61341

Ship To: INFRA METALS CO CENTRAL DIVISION
1601 BRDWAY
815-795-5041 FAX TO KYLE
MARSEILLES,IL 61341

Marking :

Heat No	C	Mn	P	S	Si	Cu	Ni	Cr	Mo	Al(tot)	V	Nb	Ti	N	Ca	B	Sn	Ceq	Pcm
9501292	0 17	1 19	0 011	0 003	0 02	0 22	0 09	0 05	0 01	0 023	0 043	0 003	0 001		0 0028	0 0001	0 008	0 41	0 25

Tensile Test

Plate Serial No	Pieces	Tons	Dir	(psi) Yield	(psi) Tensile	Elong % in 2"	Elong % in 8"
9501292-06	2	16 33	T	57,600	79,400		18 5
			T	61,200	81,300		21 6

Manufactured to fully killed fine grain practice by Electric Arc Furnace Welding or weld repair was not performed on this material
Mercury has not been used in the direct manufacturing of this material Produced as continuous cast discrete plate as-rolled unless otherwise noted in Specification For Mexico shipments nhc-SalesMX@Nucor com
Yield by 0 5EUL method unless otherwise specified Ceq = C+(Mn/6)+((Cr+Mo+V)/5)+((Cu+Ni)/15)
Pcm = C+(Si/30)+(Mn/20)+(Cu/20)+(Ni/60)+(Cr/20)+(Mo/15)+(V/10)+5B
Melted and Manufactured in the USA ISO 9001 2008 certified (#010940) by SRI Quality System Registrar (#0985-09) PED 97/23/EC 7/2 Annex 1 Para 4 3 Compliant
DIN 50049 3 1 B/EN 10204 3 1B(2004) DIN EN 10204 3 1(2005) compliant For ABS grades only Quality Assurance certificate 14-MMPQA-723

We hereby certify that the contents of this report are accurate and correct All test results and operations performed by the material manufacturer are in compliance with the applicable specifications including customer specifications

T. A. Depretis

T A Depretis Metallurgist

3/25/2019 12 52 25 PM

APPENDIX F – WELD PROCEDURE SPECIFICATIONS

WELDING PROCEDURE SPECIFICATION (WPS) Yes ☐
PREQUALIFIED YES **QUALIFIED BY TESTING** N/A
or PROCEDURE QUALIFICATION RECORDS (PQR) Yes ☐

Identification # GMAW CJP A572PL TO A992WF
Revision 0 Date 1-14-19 By DAW
Authorized by DAW Date 1-14-19
Type—Manual ☐ Semiautomatic ■
Mechanized ☐ Automatic ☐

Company Name CONCENTRIC STEEL LLC
Welding Process(es) GMAW
Supporting PQR No.(s) TC-U4a-GF

JOINT DESIGN USED
Type:
Single ☐ Double Weld ☐
Backing: Yes ■ No ☐
Backing Material: A36 FLAT BAR
Root Opening 1/4 Root Face Dimension N/A
Groove Angle: 45 Radius (J–U) N/A
Back Gouging: Yes ☐ No ■ Method ____

POSITION
Position of Groove: 1G Fillet: ____
Vertical Progression: Up ☐ Down ☐

ELECTRICAL CHARACTERISTICS
Transfer Mode (GMAW) Short-Circuiting ☐
Globular ■ Spray ☐
Current: AC ☐ DCEP ■ DCEN ☐ Pulsed ☐
Power Source: CC ☐ CV ■
Other ____
Tungsten Electrode (GTAW)
Size: ____
Type: ____

BASE METALS
Material Spec. A572 PLATE, A992 WF
Type or Grade ASTM A572 GR 50ksi, ASTM A992 GR 50ksi
Thickness: Groove ____ Fillet 1/8 - 1
Diameter (Pipe) ____

FILLER METALS
AWS Specification A5.18
AWS Classification E70C-6MD H4

TECHNIQUE
Stringer or Weave Bead: STRINGER
Multi-pass or Single Pass (per side) 1/4 MAX PER PASS
Number of Electrodes 1
Electrode Spacing Longitudinal ____
Lateral ____
Angle ____
Contact Tube to Work Distance 5/8"
Peening N/A
Interpass Cleaning: GRINDING AND WIRE BRUSH

SHIELDING
Flux ____ Gas 90% Ar, 10% CO2
Composition ____
Electrode-Flux (Class) ____ Flow Rate 30-40CFH
Gas Cup Size 5/8 TO 3/4

PREHEAT
Preheat Temp., Min. see below
Interpass Temp., Min. see below Max. 350F
1/8" thru 3/4", if below 32F, preheat to 70F, 3/4" thru 1 1/2" = 150F, 1-1/2" thru 2-1/2" = 225F, 2-1/2"+ = 300F

POSTWELD HEAT TREATMENT
Temp. N/A
Time ____

WELDING PROCEDURE

Pass or Weld Layer(s)	Process	Filler Metals		Current		Volts	Travel Speed	Joint Details
		Class	Diam.	Type & Polarity	Amps or Wire Feed Speed			
ALL	GMAW	A5.18 ER70S-6	0.045	DCEP	200-350 IPM 350-500 IPM 500-650 IPM	24-26V 26-29V 29-32V	Varies	

Form N-1 (Front)

WELDING PROCEDURE SPECIFICATION (WPS) Yes ☐
PREQUALIFIED YES QUALIFIED BY TESTING N/A
or PROCEDURE QUALIFICATION RECORDS (PQR) Yes ☐

Identification # GMAW F A572PL TO A992WF
Revision 0 Date 1-14-19 By DAW

Company Name CONCENTRIC STEEL LLC
Authorized by DAW Date 1-14-19
Welding Process(es) GMAW
Type—Manual ☐ Semiautomatic ■
Supporting PQR No.(s) N/A PREQUALIFIED
Mechanized ☐ Automatic ☐

JOINT DESIGN USED
Type:
Single ☐ Double Weld ☐
Backing: Yes ☐ No ☐
Backing Material:
Root Opening 0 Root Face Dimension N/A
Groove Angle: N/A Radius (J–U) N/A
Back Gouging: Yes ☐ No ■ Method

POSITION
Position of Groove: Fillet: 1F
Vertical Progression: Up ☐ Down ☐

ELECTRICAL CHARACTERISTICS
Transfer Mode (GMAW) Short-Circuiting ☐
Globular ■ Spray ☐
Current: AC ☐ DCEP ■ DCEN ☐ Pulsed ☐
Power Source: CC ☐ CV ■
Other
Tungsten Electrode (GTAW)
Size:
Type:

BASE METALS
Material Spec. A572 PLATE, A992 WF
Type or Grade ASTM A572 GR 50ksi, ASTM A992 GR 50ksi
Thickness: Groove Fillet 1/8 - 1
Diameter (Pipe)

FILLER METALS
AWS Specification A5.18
AWS Classification E70C-6MD H4

TECHNIQUE
Stringer or Weave Bead: STRINGER
Multi-pass or Single Pass (per side) 1/4 MAX PER PASS
Number of Electrodes 1
Electrode Spacing Longitudinal
Lateral
Angle
Contact Tube to Work Distance 5/8"
Peening N/A
Interpass Cleaning: WIRE BRUSH

SHIELDING
Flux Gas 90% Ar, 10% CO2
Composition
Electrode-Flux (Class) Flow Rate 30-40CFH
Gas Cup Size 5/8 TO 3/4

PREHEAT
Preheat Temp., Min. see below
Interpass Temp., Min. see below Max. 350F

POSTWELD HEAT TREATMENT
Temp. N/A
Time

1/8" thru 3/4", if below 32F, preheat to 70F, 3/4" thru 1 1/2" = 150F, 1-1/2" thru 2-1/2" = 225F, 2-1/2"+ = 300F

WELDING PROCEDURE

Pass or Weld Layer(s)	Process	Filler Metals		Current		Volts	Travel Speed	Joint Details
		Class	Diam.	Type & Polarity	Amps or Wire Feed Speed			
ALL	GMAW	A5.18 ER70S-6	0.045	DCEP	200-350 IPM 350-500 IPM 500-650 IPM	24-26V 26-29V 29-32V	Varies	

Form N-1 (Front)

APPENDIX G – CONNECTION DESIGN CALCULATIONS

Four Bolt Extended Stiffened End Plate Design

Test Specimens 1 and 2

Input Information

Beam Compression, P_{ub} (kips)	0.0	
Beam Shear due to gravity, $V_{gravity}$ (kips)	0.0	
Column Compression from Gravity Loads, P_{uc} (kips)	0.0	
Column Shear, V_c (kips)	0.0	
Width of End Plate, b_p (in)	10.5	
Extension of End Plate Above Flange, p_{ext} (in)	4.0625	height of stiffener, h_s, is equal to p_{ext}
Thickness of End Plate, t_p (in)	1.25	
Thickness of End-Plate Stiffener, t_s (in)	0.5	
End plate and stiffener yield stress, F_{yp} and F_{ys} (ksi)	50	
End plate ultimate stress, F_{up} (ksi)	65	
Bolt Diameter, d_b (in)	1.375	
Bolt Grade	A490	
Threads included (N) or excluded (X) from shear plane?	N	
Gage, g (in)	5	
Exterior dist to bolt row, p_{fo} (in)	2.0625	
Interior dist to bolt row, p_{fi} (in)	2.0625	
Beam Section	W24x76	
Column Section	W14x398	*Column must be W36 or Smaller*
Yield stress of beam and column, F_y (ksi)	50	
Ultimate stress of beam and column, F_u (ksi)	65	
Yield Stress Ratio, R_y	1.1	
Story Height, H (in)	150	
Bay Width, L (in)	360	
Distance from top of beam to top of column or "no" if not close, d_{top} (in)	no	
Are there beam on both sides of the column?	no	
Web fillet weld to end plate near tension bolts, t_{wt} (in)	0.375	
Web fillet weld to end plate away from tension bolts, t_{wv} (in)	0.375	
Continuity plate to column web double fillet size, t_{wcp} (in)	0.1875	
End-plate stiffener to beam flange double fillet size, t_{ws} (in)	0.375	
Weld strength, F_{EXX} (ksi)	70	
Column Continuity Plate Thickness, t_{cp} (in)	0	
Continuity Plate Width, w_{cp} (in)	0	
Column Web Doubler Plate Thickness, t_{dp} (in)	0	
Yield stress of continuity and doubler plate, $F_{yc\&dp}$ (ksi)	50	

Member Properties

Depth of Beam, d_{bm} (in)	23.90
Beam Flange Width b_{fb} (in)	8.99
Beam Flange Thickness t_{fb} (in)	0.68
Beam Web Thickness t_{wb} (in)	0.44
Area of Beam, A_b (in^2)	22.4
Moment of Inertia, I (in^4)	2100.0
Elastic Section Modulus, S_{xb} (in^3)	176.0
Beam Plastic Section Modulus, Z_{xb} (in^3)	200.0

Column Depth, d_c (in)	18.3
Column Flange Width, b_{fc} (in)	16.600
Column Flange Thickness t_{fc} (in)	2.850
Column Web Thickness, t_{wc} (in)	1.770
Area of Beam, A_c (in^2)	117.0
Column Plastic Section Modulus, Z_{xc} (in^3)	801.0

<u>Step 0 - Member Sizes</u>

beam flange slenderness, $b_f/2t_f$	6.6
Flange Compact Limit, λ_{hd}=0.30*sqrt(E/F_y)	7.22
Beam Flange is	OK
beam web slenderness, h/t_w	49.0
$P_y=F_yA_b$ (kips)	1120
For Compression: ϕ_c	0.9
$C_a=P_u/(\phi_c P_y)$	0.00
For C_a<0.125 , λ_{hd1}=2.45*sqrt(E/F_y)(1-0.93C_a)	59.0
For C_a>0.125 , λ_{hd2}=0.77*sqrt(E/F_y)(2.93-C_a)	54.3
Limit, λ_{hd3}=1.49*sqrt(E/F_y)	35.9
Controlling Limit, $\lambda_{hd}=\lambda_{hd1}$, λ_{hd2}, or λ_{hd3}	59.0
Beam Web is	OK
Column flange slenderness, $b_f/2t_f$	2.9
Flange Compact Limit, λ_{hd}=0.30*sqrt(E/F_y)	7.2
Column Flange is	OK
Column web slenderness, h/t_w	6.4
$P_y=F_yA_c$ (kips)	5850
For Compression: ϕ_c	0.9
$C_a=P_u/(\phi_c P_y)$	0.00
For C_a<0.125 , λ_{hd1}=2.45*sqrt(E/F_y)(1-0.93C_a)	59.0
For C_a>0.125 , λ_{hd2}=0.77*sqrt(E/F_y)(2.93-C_a)	54.3
Limit, λ_{hd3}=1.49*sqrt(E/F_y)	35.9
Controlling Limit, $\lambda_{hd}=\lambda_{hd1}$, λ_{hd2}, or λ_{hd3}	59.0
Column Web is	OK
Beam Flange Thickness Min, t_{bf} (in)	0.375
Beam Flange Thickness Max, t_{bf} (in)	0.75
Is Beam Flange OK?	OK
Beam Flange Width Min, b_{bf} (in)	6.0
Beam Flange Width Max, b_{bf} (in)	9.00
Is Beam Flange OK?	OK
Beam Depth Min, d (in)	13.8
Beam Depth Max, d (in)	24.0
Is Beam Depth OK?	OK
End-Plate Thickness Min, t_p (in)	0.5
End-Plate Thickness Max, t_p (in)	1.5
Is End-Plate Thickenss OK?	OK
End-Plate Width Min, b_p (in)	7.0
End-Plate Width Max, b_p (in)	10.8
Is End-Plate Width OK?	OK
Bolt Gage Min, g (in)	3.3
Bolt Gage Max, g (in)	6.0
Is Bolt Gage OK?	OK
Bolt Spacing Min, p_{fi} p_{fo} (in)	1.8
Bolt Spacing Max, p_{fi} p_{fo} (in)	5.5

Is Bolt Spacing OK?	OK
Beam Clear Span, $L_c = L - d_c$ (in)	341.7
Span-to-Depth Ratio, L_c / d_b	14.3
Span-to-Depth Limit	7.0
Is Span-to-Depth OK?	OK

Step 1.1 - Calculate Moment at Face of Column

C_{pr}=min {(F_y+F_u)/(2F_y), 1.2}	1.15
Probable Max Moment, $M_{pr}=C_{pr}R_yF_yZ_x$ (k-in)	12650
height of stiffener, $h_{st} = p_{ext}$ (in)	4
Length of end-plate stiffener, $L_{st}=h_{st}/\tan(30°)$ (in)	7.04
Dist to Plastic Hinge, $S_h=L_{st}+t_p$ (in)	8.29
Distance Between Plastic Hinges, $L_h=L-d_c-2S_h$ (in)	325.1
Shear, $V_u=2M_{pr}/L_h+V_{gravity}$ (kips)	77.8
Moment at Face, $M_f=M_{pr}+V_u S_h$ (k-in)	13295

Step 1.3 - Check Bolt Size

dist to first row, h_2 (in)	20.82
dist to second row, h_1 (in)	25.62
Nominal Stress for Bolts, F_{nt} (ksi)	113
for nonductile limit state, ϕ_n	0.90
Minimum Bolt Diameter d_{reqd}= sqrt(2*M_f/(π*ϕ_r*F_{nt}*(h_1+h_2))) (in)	1.339
Selected bolt diameter from input info	1.375
Is Bolt Diameter > Min?	OK

Step 1.5 - Check End-Plate Thickness

Effective end-plate width, b_{peff}=min(b_p,b_{bf}+1) (in)	10.0
distance to yield line, s=1/2*sqrt(b_p*g)	3.62

$$Y=\frac{b_p}{2}\left[h_1\left(\frac{1}{p_{fo}}\right)+h_2\left(\frac{1}{p_{fi}}+\frac{1}{s}\right)-\frac{1}{2}\right]+\frac{2}{g}\left[h_1\left(p_{ext}\right)+h_2\left(s+p_{fi}\right)\right]+\frac{g}{4}$$

new equation that is more conservative than 358

End Plate Yield Parameter, $Y_{p\text{-}new}$ (in)	228.9

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

from AISC 358-16. Note that h1=h2 and h0=h1

End Plate Yield Parameter, $Y_{p\text{-}old}$ (in)	250.9
End Plate Yield Parameter to use, $Y_p = Y_{p\text{-}new}$ (in)	228.9
for ductile limit state, ϕ_d	1.00
Req'd end plate, t_{pReqd}=sqrt(1.11*M_f/(ϕ_d*F_{py}*Y))	1.085
Selected end plate thickness from input info	1.250
Is $t_p > t_{pReqd}$	OK

Step 1.7 - Calculate Beam Flange Force

Flange force, $F_{fu}=M_f / (d-t_{bf})$ (kips)	573

Step 1.10 - Stiffener Checks

Stiffener Geometry

Minimum stiffener thickness, $t_{s\text{-}min}=t_{wb}(F_y/F_{ys})$ (in)	0.440
Is $t_s > t_{s\text{-}min}$	OK
height of stiffener, $h_{st} = p_{ext}$ (in)	4.06
Stiffener slenderness limit, $(h/t)_{limit}$=0.56 sqrt(E/F_{ys})	13.49
Stiffener slenderness, h_{st}/t_s	8.13
Is $h_{st}/t_s < (h/t)_{limit}$?	OK

Stiffener Welds

Stiffener to end-plate is CJP	OK	if stiffener, ts<=3/8" then can use double fillet
Stiffener to beam flange demand, $R_u = 0.6\ F_{ys}\ t_s$ (k/in)	15.00	
Stif. to beam strength, R_n=(2 welds)0.6$F_{EXX}\ t_{ws}$/sqrt(2) (k/in)	22.27	
for fillet welds ϕ	0.75	
ϕR_n	16.71	
Is $\phi R_n > T_u$	OK	

Step 1.11 - Bolt Shear Rupture Strength

Required Shear Force From Step 1.1, V_u (kips)	78
Number of bolts resisting shear (compression side), n_b	4
Nominal Shear Strength of Bolts, F_{nv} (ksi)	68
Area of bolt, A_{bolt} (in^2)	1.48
For nonductile limit state, ϕ_n	0.90
$\phi_n\ R_n = \phi_n\ n_b\ F_{nv}\ A_{bolt}$ (kips)	363
Is $\phi R_n > V_u$?	OK

Step 1.12 - Bearing / Tear-Out

(tF_u) = min {$t_p\ F_{up}$, $t_{fc}\ F_u$} (k/in)	81.3
Inner Bolts at Compression Side	
clear distance, $L_c=p_{fo}+t_f+p_{fi}-d_b$ (in)	3.37
Tear-Out Strength, 1.2 L_c (t F_u) (kips/bolt)	328.3
Bearing Strength, 2.4 d_b (t F_u) (kips/bolt)	268.1
Strength, R_{n1} = min (Tear-Out, Bearing) (kips/bolt)	268.1
Outer Bolts at Compression Side	
clear distance, $L_c=p_{ext}-p_{fo}-(d_b+16)/2$ (in)	1.28
Tear-Out Strength, 1.2 L_c (t F_u) (kips/bolt)	124.9
Bearing Strength, 2.4 d_b (t F_u) (kips/bolt)	268.1
Strength, R_{n2} = min (Tear-Out, Bearing) (kips)	124.9
Total Resistance	
$R_n = 2R_{n1} + 2R_{n2}$ (kips)	786.1
For nonductile limit state, ϕ_n	0.90
Design Bearing / Tear-Out Strength, ϕR_n (k-in)	707
Required Shear, Vu from Step 1.1 (kips)	78
Is $\phi R_n > V_u$?	OK

Step 1.13 - Weld Design

Beam Flange to End Plate	
Flange to End Plate is CJP	OK
Beam Web to End Plate Weld Near Tension Bolts	
Web Tension, $T_u=F_y*t_{wb}$ (k/in)	22.00
$R_{nFillet}$ = 2 welds*1.5*0.6 $F_{EXX}\ t_{wt}$/sqrt(2) (k/in)	33.41
for fillet welds ϕ	0.75
ϕR_n	25.06
Is $\phi R_n > T_u$	OK
Beam Web to End Plate Weld Away from Tension Bolts	
Height of weld, $L_{wv}=d_{bm}-t_{fb}-p_{fi}-6-t_{fb}$ (in)	14.48
$R_n = 0.6\ F_{EXX}\ t_{wv}$/sqrt(2) 2$L_{wv}$ (kips)	322.5
for fillet welds ϕ	0.75
ϕR_n	241.9
Is $\phi R_n > V_u$	OK

Column-Side Design

Step 2.1 - Check Column Flange Flexural Yielding

distance to yield line, s=1/2*sqrt(b_{cf}*g)	4.56
c=$p_{fi}+t_{fb}+p_{fo}$ (in)	4.81

$$Y_c = \frac{b_{cf}}{2}\left[h_1\left(\frac{1}{s}\right)+h_0\left(\frac{1}{s}\right)\right]+\frac{2}{g}\left[h_1\left(s+\frac{3c}{4}\right)+h_0\left(s+\frac{c}{4}\right)+\frac{c^2}{2}\right]+\frac{g}{2}$$

note that h1=h2 and h0=h1

Column Flange Yield Parameter if Unstiffened, $Y_{c\text{-}us}$ (in)	218.7
$p_{si} = p_{fi}+t_{fb}/2-t_{cp}/2$ (in)	2.40
$p_{so}= p_{fo}+t_{fb}/2-t_{cp}/2$ (in)	2.40

$$Y_c = \frac{b_{cf}}{2}\left[h_1\left(\frac{1}{s}+\frac{1}{p_{si}}\right)+h_0\left(\frac{1}{s}+\frac{1}{p_{so}}\right)\right]+\frac{2}{g}\left[h_1(s+p_{si})+h_0(s+p_{so})\right]$$

note that h1=h2 and h0=h1

Column Flange Yield Parameter if Stiffened, $Y_{c\text{-}s}$ (in)	374.3
Column Flange Yield Parameter, Y_c = $Y_{c\text{-}us}$ or $Y_{c\text{-}s}$ (in)	218.7
for ductile limit state, ϕ_d	1.00
Req'd column flange, t_{pReqd}=sqrt(1.11*M_f/(ϕ_d*F_y*Y_c))	1.162
Column flange thickness	2.850
Is $t_p > t_{pReqd}$	OK

Step 2.2 - Calculate Column Flange Force Design Strength

for ductile limit state, ϕ_d	1.00
Design Moment Strength, $\phi_d M_{cf}=\phi_d F_y Y_c t_{cf}^2$ (k-in)	88809
Design Force Strength, $\phi_d R_n=(\phi_d M_{cf})/(d_b-t_{bf})$ (kips)	3824.7

Step 2.3 - Check Local Column Web Yielding

Required Flange Force from Step 1.7, F_{fu} (kips)	573
C_t=0.5 if beam within d of top of column, else C_t=1.0	1.0
Fillet distance, k_c (in)	3.44
Nominal Strength, $R_n=C_t(6k_c+t_{bf}+2t_p)F_y t_{cw}$ (kips)	2108.1
for ductile limit state, ϕ_d	1.00
Design Strength, $\phi_d R_n$ (kips)	2108.1
Is $\phi_d R_n > F_{fu}$?	OK

Step 2.4 - Check Column Web Buckling

Required Flange Force from Step 1.7, F_{fu} (kips)	573
Is dist from beam flange to top col. < $d_c/2$?	no
clear dist between column flange fillets, h=d-2k_c (in)	11.42
If close to top of col, $R_{n1}=24t_{cw}^3$sqrt(EF_y)/h (kips)	14032.9
If not close to top of col, $R_{n2}=12t_{cw}^3$sqrt(EF_y)/h (kips)	7016.5
Nominal Strength, $R_n=R_{n1}$ or R_{n2} (kips)	14032.9
for column web buckling, ϕ	0.75
Design Strength, ϕR_n (kips)	10524.7
Is $\phi_d R_n > F_{fu}$?	OK

Step 2.5 - Check Column Web Crippling

Required Flange Force from Step 1.7, F_{fu} (kips)	573	
Is dist from beam flange to top col. < $d_c/2$?	no	
Term X_1=sqrt(EF_yt_{cf}/t_{cw}) (ksi)	1528	
Beam flange CJP reinforcing fillet from Sec. 6.9.7(2), t_{wr} (in)	0.313	
Dimension, $N=t_{bf}+t_{wr}+2t_p$ (in)	3.493	
If not close to top of col, $R_{n1}=0.8t_{cw}^2[1+3(N/d_c)(t_{cw}/t_{cf})^{1.5}]X_1$ (kips)	4902.8	
If close to top of col, $R_{n2}=0.4t_{cw}^2[1+3(N/d_c)(t_{cw}/t_{cf})^{1.5}]X_1$ (kips)	2451.4	
If close to top of col, $R_{n3}=0.4t_{cw}^2[1+(4N/d_c-0.2)(t_{cw}/t_{cf})^{1.5}]X_1$ (kips)	2442.8	
N/d_c to compare to 0.2 to decide which Rn	0.19	
Nominal Strength, $R_n=R_{n1}$, R_{n2} or R_{n3} (kips)	4902.8	
for column web crippling, ϕ	0.75	
Design Strength, ϕR_n (kips)	3677.1	
Is $\phi_d R_n > F_{fu}$?	OK	

Step 2.6 - Check Continuity Plate Size

Required Compression Strength		
Required Flange Force from Step 1.7, F_{fu} (kips)	573	
Min ϕR_n from step 2.2, 2.3, 2.4, 2.5 (kips)	2108.1	
Required strength, $F_{su}=F_{fu}$-min(ϕR_n) (kips)	-1535.5	
Compression Strength of Effective Section		
min plate thickness of AISC 341 E3.6f(2), $t_{cp\text{-}min}$ (in)	0.340	
Is $t_{cp} > t_{cp\text{-}min}$?	No Good	No continuity plates unecessary
Effective length, KL=0.75h (in)	8.565	AISC 360 J10.8
Is dist from beam flange to top col. < $d_c/2$?	no	similar limit as for web buckling and crippling
Effective web width if not close to top of column, $w_{w1}=25t_{wc}$ (in)	44.25	
Effective web width if close to top of column, $w_{w2}=12t_{wc}$ (in)	21.24	
Effective web width, $w_w=w_{w1}$ or w_{w2} (in)	44.25	
Moment of Inertia effective section, I_{cp} (in4)	20.45	
Area of effective section, A_{cp} (in2)	78.32	
Radius of Gyration, r_{cp}=sqrt(I_{cp}/A_{cp}) (in)	0.51	
Slenderness, KL/r_{cp}	16.76	
Is KL/r_{cp} < 25?	OK	AISC 360 J10.8 reference J4.4. If no, need to use Chapter
Compression Strength Assuming KL/r_{cp}<25, $P_n=F_{yc\&dp}A_{cp}$ (kips)	3916.1	AISC 360 J4.4
for continuity plate compression, ϕ	0.90	AISC 360 J4.4
Design Strength, ϕR_n (kips)	3524.5	
Is $\phi_d R_n > F_{su}$?	OK	
Continuity Plate Welds to Column Flange		
Continuity plate weld to flange is CJP	OK	
Continuity Plate Welds to Column Web		
k_{det} for column (in)	4.13	
Contact length along column web, $L_{cpw}=d_c-2(k_{det}+1.5)$ (in)	7.1	1.5" is Seismic clip per AWS D1.8 4.1
k_1 for column (in)	2.13	
Contact length along column flange, $L_{cpf}=(w_{cp}+t_{cw}/2)-(k_1+0.5)$ (in)	-1.74	
Design Strength of CP contact area in tension, $R_{u\text{-}a}=0.9(2L_{cpf})t_{cp}F_{yc\&dp}$ (kips)	0.0	AISC 341 E3.6f(3)(a)
Des. Strng of CP contact area in shear, $R_{u\text{-}b}=0.9(2L_{cpw})t_{cp}(0.6F_{yc\&dp})$ (kips)	0.0	AISC 341 E3.6f(3)(b)
Design Strength of Panel Zone from Step 2.7, $R_{u\text{-}c}$ (kips)	1479.5	AISC 341 E3.6f(3)(c)
Number of beam flanges transmitting force to continuity plate, N_b	1	
Expected Strength of Flanges, $R_{u\text{-}d}=R_yF_yt_{bf}b_{fb}\,N_b$ (kips)	336.2	AISC 341 E3.6f(3)(d)
Continuity plate weld to web demand, R_u=min{$R_{u\text{-}a}$, $R_{u\text{-}b}$, $R_{u\text{-}c}$, $R_{u\text{-}d}$} (kips)	0.0	AISC 341 E3.6f(3)
$R_n = 0.6\,F_{EXX}\,[t_{wcp}/\text{sqrt}(2)]\,4L_{cpw}$ (kips)	157.0	welds for both continuity plates
for fillet welds ϕ	0.75	
ϕR_n	117.8	
Is $\phi R_n > R_u$	OK	

Step 2.7 - Check Panel Zone

Panel Zone Strength

Number of beams, N_b	1	
Panel Zone Shear, $V_{upz} = F_{fu} - V_c - P_{ub}/2$ (kips)	572.6	
Column squash load, $P_y = F_y A_c$ (kips)	5850	
Column compression, $P_u = P_{uc} + V_u$ (kips)	78	
Ratio of P_u/P_y (kips)	0.01	
Panel zone thickness, $t_{pz} = t_{cw} + 2t_{dp}$ (in)	1.77	
$R_n = 0.6F_y\, d_c\, t_{pz}\, (1+3b_{cf}\, t_{cf}^2 / (d_{bm}\, d_c\, t_{pz}))$ (kips)	1479.5	AISC 360 J10.6 - see assumptions
for panel zone, ϕ	1.00	AISC 341 E3.6e(1)
Design Strength, ϕR_n (kips)	1479.5	
Is $\phi_d R_n > F_{su}$?	OK	
Min Panel Zone Thickness		
$d_z = d_{bm} - 2t_{fb}$ (in)	22.54	
$w_z = d_c - 2t_{cf}$ (in)	12.6	
$t_{min} = (d_z + w_z)/90$ (in)	0.390	
Is $t_{dp} > t_{min}$?	No Good	No doubler plates included
Is $t_{cw} > t_{min}$?	OK	

Doubler Plate Welds Not Designed

Step 2.8 - Strong Column Weak Beam (not listed in steps, but should be - Tell Tom)

V_u from Step 1.1	77.8
$M_v = V_u(S_h + d_c/2)$ (k-in)	1356.9
Number of Beams, N_b	1
$\Sigma M_{pb}^* = N_b(M_{pr} + M_v)$ (k-in)	14007
Column Compression from Step 2.7, P_u (kips)	77.8
Number of Columns, N_c	2
$\Sigma M_{pc}^* = N_c Z_c (F_y - P_u/A_c)$ (k-in)	79035
$\Sigma M_{pc}^* / \Sigma M_{pb}^*$	5.64
Is $\Sigma M_{pc}^* / \Sigma M_{pb}^* > 1.0$?	OK

Eight Bolt Extended Stiffened End Plate Design

Test Specimens 3 and 4

<u>Input Information</u>

Beam Compression, P_{ub} (kips)	0.0	
Beam Shear due to gravity, $V_{gravity}$ (kips)	0.0	
Column Compression from Gravity Loads, P_{uc} (kips)	0.0	
Column Shear, V_c (kips)	0.0	
Width of End Plate, b_p (in)	14	
Extension of End Plate Above Flange, p_{ext} (in)	7.375	height of stiffener, h_s, is equal to p_{ext}
Thickness of End Plate, t_p (in)	1.5	
Thickness of End-Plate Stiffener, t_s (in)	0.625	
End plate and stiffener yield stress, F_{yp} and F_{ys} (ksi)	50	
End plate ultimate stress, F_{up} (ksi)	65	
Bolt Diameter, d_b (in)	1.375	
Bolt Grade	A490	
Threads included (N) or excluded (X) from shear plane?	N	
Gage, g (in)	5	
Exterior dist to bolt row, p_{fo} (in)	1.875	
Interior dist to bolt row, p_{fi} (in)	1.875	
Interior bolt row spacing, p_b (in)	3.75	
Beam Section	W36x150	
Column Section	W14x398	*Column must be W36 or Smaller*
Yield stress of beam and column, F_y (ksi)	50	
Ultimate stress of beam and column, F_u (ksi)	65	
Yield Stress Ratio, R_y	1.1	
Story Height, H (in)	150	
Bay Width, L (in)	360	
Distance from top of beam to top of column or "no" if not close, d_{top} (in)	no	For column web checks
Are there beam on both sides of the column?	no	For SCWB and panel zone
Web fillet weld to end plate near tension bolts, t_{wt} (in)	0.5	
Web fillet weld to end plate away from tension bolts, t_{wv} (in)	0.5	
Continuity plate to column web double fillet size, t_{wcp} (in)	0.5	
End-plate stiffener to beam flange double fillet size, t_{ws} (in)	0.4375	
Weld strength, F_{EXX} (ksi)	70	
Column Continuity Plate Thickness, t_{cp} (in)	0	
Continuity Plate Width, w_{cp} (in)	0	
Column Web Doubler Plate Thickness, t_{dp} (in)	0	
Yield stress of continuity and doubler plate, $F_{yc\&dp}$ (ksi)	50	

Member Properties

Depth of Beam, d_{bm} (in)	35.90
Beam Flange Width b_{fb} (in)	12.00
Beam Flange Thickness t_{fb} (in)	0.94
Beam Web Thickness t_{wb} (in)	0.625
Area of Beam, A_b (in^2)	44.3
Moment of Inertia, I (in^4)	9040.0
Elastic Section Modulus, S_{xb} (in^3)	504.0
Beam Plastic Section Modulus, Z_{xb} (in^3)	581.0
Column Depth, d_c (in)	18.3
Column Flange Width, b_{fc} (in)	16.6
Column Flange Thickness t_{fc} (in)	2.85
Column Web Thickness, t_{wc} (in)	1.77
Area of Column, A_c (in^2)	117.0
Column Plastic Section Modulus, Z_{xc} (in^3)	801.0

Step 0 - Member Sizes

beam flange slenderness, $b_f/2t_f$	6.4
Flange Compact Limit, λ_{hd}=0.30*sqrt(E/F_y)	7.22
Beam Flange is	OK
beam web slenderness, h/t_w	51.9
$P_y=F_yA_b$ (kips)	2215
For Compression: ϕ_c	0.9
$C_a=P_u/(\phi_c P_y)$	0.00
For C_a<0.125 , λ_{hd1}=2.45*sqrt(E/F_y)(1-0.93C_a)	59.0
For C_a>0.125 , λ_{hd2}=0.77*sqrt(E/F_y)(2.93-C_a)	54.3
Limit, λ_{hd3}=1.49*sqrt(E/F_y)	35.9
Controlling Limit, $\lambda_{hd}=\lambda_{hd1}$, λ_{hd2}, or λ_{hd3}	59.0
Beam Web is	OK
Column flange slenderness, $b_f/2t_f$	2.9
Flange Compact Limit, λ_{hd}=0.30*sqrt(E/F_y)	7.2
Column Flange is	OK
Column web slenderness, h/t_w	6.4
$P_y=F_yA_c$ (kips)	5850
For Compression: ϕ_c	0.9
$C_a=P_u/(\phi_c P_y)$	0.00
For C_a<0.125 , λ_{hd1}=2.45*sqrt(E/F_y)(1-0.93C_a)	59.0
For C_a>0.125 , λ_{hd2}=0.77*sqrt(E/F_y)(2.93-C_a)	54.3
Limit, λ_{hd3}=1.49*sqrt(E/F_y)	35.9
Controlling Limit, $\lambda_{hd}=\lambda_{hd1}$, λ_{hd2}, or λ_{hd3}	59.0
Column Web is	OK

Beam Flange Thickness Min, t_{bf} (in)	0.5625
Beam Flange Thickness Max, t_{bf} (in)	1.00
Is Beam Flange OK?	OK
Beam Flange Width Min, b_{bf} (in)	7.5
Beam Flange Width Max, b_{bf} (in)	12.3
Is Beam Flange OK?	OK
Beam Depth Min, d (in)	18.0
Beam Depth Max, d (in)	36.0
Is Beam Depth OK?	OK
End-Plate Thickness Min, t_p (in)	0.8
End-Plate Thickness Max, t_p (in)	2.5
Is End-Plate Thickenss OK?	OK
End-Plate Width Min, b_p (in)	9.0
End-Plate Width Max, b_p (in)	15.0
Is End-Plate Width OK?	OK
Bolt Gage Min, g (in)	5.0
Bolt Gage Max, g (in)	6.0
Is Bolt Gage OK?	OK
Bolt Spacing Min, p_{fi} p_{fo} (in)	1.6
Bolt Spacing Max, p_{fi} p_{fo} (in)	2.0
Is Bolt Spacing OK?	OK

Beam Clear Span, $L_c = L - d_c$ (in)	341.7
Span-to-Depth Ratio, L_c / d_{bm}	9.5
Span-to-Depth Limit	7.0
Is Span-to-Depth OK?	OK

Step 1.1 - Calculate Moment at Face of Column

C_{pr}=min {$(F_y+F_u)/(2F_y)$, 1.2}	1.15
Probable Max Moment, $M_{pr}=C_{pr}R_yF_yZ_x$ (k-in)	36748
height of stiffener, $h_{st} = p_{ext}$ (in)	7.38
Length of end-plate stiffener, $L_{st}=h_{st}/\tan(30°)$ (in)	12.77
Dist to Plastic Hinge, $S_h=L_{st}+t_p$ (in)	14.27
Distance Between Plastic Hinges, $L_h=L-d_c-2S_h$ (in)	313.2
Shear, $V_u=2M_{pr}/L_h+V_{gravity}$ (kips)	234.7
Moment at Face, $M_f=M_{pr}+V_u S_h$ (k-in)	40098

Step 1.3 - Check Bolt Size

dist to second row, h_1 (in)	41.055
dist to second row, h_2 (in)	37.31
dist to first row, h_3 (in)	32.62
dist to first row, h_4 (in)	28.87
Nominal Stress for Bolts, F_{nt} (ksi)	113
for nonductile limit state, ϕ_n	0.90
Minimum Bolt Diameter d_{reqd}= sqrt(2*M_f/(π*ϕ_n*F_{nt}*($h_1+h_2+h_3+h_4$))) (in)	1.340
Selected bolt diameter from input info	1.375
Is Bolt Diameter > Min?	OK

Step 1.5 - Check End-Plate Thickness

distance from top bolt to edge of plate, $d_e=p_{ext}-p_b-p_{fo}$ (in)	1.75	
Effective end-plate width, bp = min (bp, bbf+1) (in)	13.00	
distance to yield line, $s=1/2*\mathrm{sqrt}(b_p\,g)$ (in)	4.03	

$$Y=\frac{b_p}{2}\left[h_2\left(\frac{1}{P_{fo}}\right)+h_3\left(\frac{1}{P_{fi}}\right)+h_4\left(\frac{1}{s}\right)-\frac{1}{2}\right]+\frac{2}{g}\left[h_1\left(0.5P_b+0.5d_e\right)+h_2\left(0.25P_b+P_{fo}\right)+h_3\left(P_b+P_{fi}+0.5d_e\right)+h_4\left(s+0.5P_b\right)\right]+\frac{5g}{4}$$

End Plate Yield Parameter, $Y_{p\text{-}new}$ (in)	532.1	New Equation - more conservative

$$Y_p=\frac{b_p}{2}\left[h_1\left(\frac{1}{2d_e}\right)+h_2\left(\frac{1}{p_{fo}}\right)+h_3\left(\frac{1}{p_{fi}}\right)+h_4\left(\frac{1}{s}\right)\right]$$
$$+\frac{2}{g}\left[h_1\left(d_e+\frac{3p_b}{4}\right)+h_2\left(p_{fo}+\frac{p_b}{4}\right)+h_3\left(p_{fi}+\frac{3p_b}{4}\right)+h4\left(s+\frac{p_b}{4}\right)\right]+g$$

from AISC 358-16

End Plate Yield Parameter, $Y_{p\text{-}old}$ (in)	605.6	
End Plate Yield Parameter to use, $Y_p = Y_{p\text{-}new}$ (in)	605.6	
for ductile limit state, ϕ_d	1.00	
Req'd end plate, $t_{pReqd}=\mathrm{sqrt}(1.11*M_f/(\phi_d*F_{py}*Y))$	1.293	
Selected end plate thickness from input info	1.500	
Is $t_p > t_{pReqd}$	OK	

Step 1.7 - Calculate Beam Flange Force

Flange force, $F_{fu}=M_f / (d-t_{bf})$ (kips)	1147

Step 1.10 - Stiffener Checks

Stiffener Geometry

Minimum stiffener thickness, $t_{s\text{-}min}=t_{wb}(F_y/F_{ys})$ (in)	0.625
Is $t_s > t_{s\text{-}min}$	OK
height of stiffener, $h_{st} = p_{ext}$ (in)	7.38
Stiffener slenderness limit, $(h/t)_{limit}=0.56\ \mathrm{sqrt}(E/F_{ys})$	13.49
Stiffener slenderness, h_{st}/t_s	11.80
Is $h_{st}/t_s < (h/t)_{limit}$?	OK

Stiffener Welds

Stiffener to end-plate is CJP	OK
Stiffener to beam flange demand, $R_u = 0.6\ F_{ys}\ t_s$ (k/in)	18.75
Stif. to beam strength, R_n=(2 welds)$0.6F_{EXX}\ t_{ws}/\mathrm{sqrt}(2)$ (k/in)	25.99
for fillet welds ϕ	0.75
ϕR_n	19.49
Is $\phi R_n > T_u$	OK

Step 1.11 - Bolt Shear Rupture Strength

Required Shear Force From Step 1.1, V_u (kips)	235
Number of bolts resisting shear (compression side), n_b	8
Nominal Shear Strength of Bolts, F_{nv} (ksi)	68
Area of bolt, A_{bolt} (in^2)	1.48
For nonductile limit state, ϕ_n	0.90
$\phi_n\ R_n = \phi_n\ n_b\ F_{nv}\ A_{bolt}$ (kips)	727
Is $\phi R_n > V_u$?	OK

Step 1.12 - Bearing / Tear-Out

$(tF_u) = \min\{t_p F_{up}, t_{fc} F_u\}$ (k/in)	97.5
Inner Bolts at Compression Side	
clear distance, $L_c = p_b - (d_b + 1/16)$ (in)	2.31
Tear-Out Strength, 1.2 L_c (t F_u) (kips/bolt)	270.6
Bearing Strength, 2.4 d_b (t F_u) (kips/bolt)	321.8
Strength, R_{n1} = min (Tear-Out, Bearing) (kips/bolt)	270.6
Outer Bolts at Compression Side	
clear distance, $L_c = p_{ext} - p_{fo} - (d_b + 16)/2$ (in)	1.03
Tear-Out Strength, 1.2 L_c (t F_u) (kips/bolt)	120.7
Bearing Strength, 2.4 d_b (t F_u) (kips/bolt)	321.8
Strength, R_{n2} = min (Tear-Out, Bearing) (kips)	120.7
Total Resistance	
$R_n = 6R_{n1} + 2R_{n2}$ (kips)	1864.7
For nonductile limit state, ϕ_n	0.90
Design Bearing / Tear-Out Strength, ϕR_n (k-in)	1678
Required Shear, Vu from Step 1.1 (kips)	235
Is $\phi R_n > V_u$?	OK

Step 1.13 - Weld Design

Beam Flange to End Plate	
Flange to End Plate is CJP	OK
Beam Web to End Plate Weld Near Tension Bolts	
Web Tension, $T_u = F_y * t_{wb}$ (k/in)	31.25
$R_{nFillet}$ = 2 welds*1.5*0.6 F_{EXX} t_{wt}/sqrt(2) (k/in)	44.55
for fillet welds ϕ	0.75
ϕR_n	33.41
Is $\phi R_n > T_u$	OK
Beam Web to End Plate Weld Away from Tension Bolts	
Required Shear Force From Step 1.1, V_u (kips)	235
Height of weld, $L_{wv} = d_{bm} - t_{fb} - p_b - p_{fi} - 6 - t_{fb}$ (in)	22.40
R_n = 0.6 F_{EXX} t_{wv}/sqrt(2) 2L_{wv} (kips)	665.1
for fillet welds ϕ	0.75
ϕR_n	498.8
Is $\phi R_n > V_u$	OK

Column-Side Design

Step 2.1 - Check Column Flange Flexural Yielding

distance to yield line, $s=1/2*\mathrm{sqrt}(b_{cf}*g)$	4.56
$c=p_{fi}+t_{fb}+p_{fo}$ (in)	4.69

$$Y_c = \frac{b_{cf}}{2}\left[h_1\left(\frac{1}{s}\right)+h_4\left(\frac{1}{s}\right)\right] + \frac{2}{g}\left[h_1\left(p_b+\frac{c}{2}+s\right)+h_2\left(\frac{p_b}{2}+\frac{c}{4}\right)+h_3\left(\frac{p_b}{2}+\frac{c}{2}\right)+h_4(s)\right]+\frac{g}{2}$$

Column Flange Yield Parameter if Unstiffened, $Y_{c\text{-}us}$ (in)	457.9
$p_{si} = p_{fi}+t_{fb}/2-t_{cp}/2$ (in)	2.35
$p_{so}= p_{fo}+t_{fb}/2-t_{cp}/2$ (in)	2.35

$$Y_c = \frac{b_{cf}}{2}\left[h_1\left(\frac{1}{s}\right)+h_2\left(\frac{1}{p_{so}}\right)+h_3\left(\frac{1}{p_{si}}\right)+h_4\left(\frac{1}{s}\right)\right] + \frac{2}{g}\left[h_1\left(s+\frac{p_b}{4}\right)+h_2\left(p_{so}+\frac{3p_b}{4}\right)+h_3\left(p_{si}+\frac{p_b}{4}\right)+h_4\left(s+\frac{3p_b}{4}\right)+p_b^2\right]+g$$

Column Flange Yield Parameter if Stiffened, $Y_{c\text{-}s}$ (in)	680.6
Column Flange Yield Parameter, $Y_c = Y_{c\text{-}us}$ or $Y_{c\text{-}s}$ (in)	457.9
for ductile limit state, ϕ_d	1.00
Req'd column flange, $t_{pReqd}=\mathrm{sqrt}(1.11*M_f/(\phi_d*F_y*Y_c))$	1.394
Column flange thickness	2.850
Is $t_p > t_{pReqd}$	OK

Step 2.2 - Calculate Column Flange Force Design Strength

for ductile limit state, ϕ_d	1.00
Design Moment Strength, $\phi_d M_{cf}=\phi_d F_y Y_c t_{cf}^2$ (k-in)	185973
Design Force Strength, $\phi_d R_n=(\phi_d M_{cf})/(d_b-t_{bf})$ (kips)	5319.6

Step 2.3 - Check Local Column Web Yielding

Required Flange Force from Step 1.7, F_{fu} (kips)	1147
C_t=0.5 if beam within d of top of column, else C_t=1.0	1.0
Fillet distance, k_c (in)	3.44
Nominal Strength, $R_n=C_t(6k_c+t_{bf}+2t_p)F_y t_{cw}$ (kips)	2175.3
for ductile limit state, ϕ_d	1.00
Design Strength, $\phi_d R_n$ (kips)	2175.3
Is $\phi_d R_n > F_{fu}$?	OK

Step 2.4 - Check Column Web Buckling

Required Flange Force from Step 1.7, F_{fu} (kips)	1147
Is dist from beam flange to top col. < $d_c/2$?	no
clear dist between column flange fillets, $h=d-2k_c$ (in)	11.42
If close to top of col, $R_{n1}=24t_{cw}^3\mathrm{sqrt}(EF_y)/h$ (kips)	14032.9
If not close to top of col, $R_{n2}=12t_{cw}^3\mathrm{sqrt}(EF_y)/h$ (kips)	7016.5
Nominal Strength, $R_n=R_{n1}$ or R_{n2} (kips)	14032.9
for column web buckling, ϕ	0.75
Design Strength, ϕR_n (kips)	10524.7
Is $\phi_d R_n > F_{fu}$?	OK

Step 2.5 - Check Column Web Crippling

Required Flange Force from Step 1.7, F_{fu} (kips)	1147	
Is dist from beam flange to top col. < $d_c/2$?	no	
Term X_1=sqrt($EF_y t_{cf}/t_{cw}$) (ksi)	1528	
Beam flange CJP reinforcing fillet from Sec. 6.9.7(2), t_{wr} (in)	0.313	
Dimension, $N=t_{bf}$+twr+$2t_p$ (in)	4.253	
If not close to top of col, $R_{n1}=0.8t_{cw}^2[1+3(N/d_c)(t_{cw}/t_{cf})^{1.5}]X_1$ (kips)	5136.3	
If close to top of col, $R_{n2}=0.4t_{cw}^2[1+3(N/d_c)(t_{cw}/t_{cf})^{1.5}]X_1$ (kips)	2568.1	
If close to top of col, $R_{n3}=0.4t_{cw}^2[1+(4N/d_c-0.2)(t_{cw}/t_{cf})^{1.5}]X_1$ (kips)	2598.5	
N/d_c to compare to 0.2 to decide which Rn	0.23	
Nominal Strength, $R_n=R_{n1}$, R_{n2} or R_{n3} (kips)	5136.3	
for column web crippling, ϕ	0.75	
Design Strength, ϕR_n (kips)	3852.2	
Is $\phi_d R_n > F_{fu}$?	OK	

Step 2.6 - Check Continuity Plate Size

Required Compression Strength		
Required Flange Force from Step 1.7, F_{fu} (kips)	1147	
Min ϕR_n from step 2.2, 2.3, 2.4, 2.5 (kips)	2175.3	
Required strength, $F_{su}=F_{fu}$-min(ϕR_n) (kips)	-1028.3	
Compression Strength of Effective Section		
min plate thickness of AISC 341 E3.6f(2), $t_{cp\text{-}min}$ (in)	0.470	
Is $t_{cp} > t_{cp\text{-}min}$?	No Good	No continuity plates
Effective length, KL=0.75h (in)	8.565	AISC 360 J10.8
Is dist from beam flange to top col. < $d_c/2$?	no	
Effective web width if not close to top of column, $w_{w1}=25t_{wc}$ (in)	44.25	
Effective web width if close to top of column, $w_{w2}=12t_{wc}$ (in)	21.24	
Effective web width, $w_w=w_{w1}$ or w_{w2} (in)	44.25	
Moment of Inertia effective section, I_{cp} (in4)	20.45	
Area of effective section, A_{cp} (in2)	78.32	
Radius of Gyration, r_{cp}=sqrt(I_{cp}/A_{cp}) (in)	0.51	
Slenderness, KL/r_{cp}	16.76	
Is KL/r_{cp} < 25?	OK	AISC 360 J10.8 reference J4.4.
Compression Strength Assuming KL/r_{cp}<25, $P_n=F_{yc\&dp}A_{cp}$ (kips)	3916.1	AISC 360 J4.4
for continuity plate compression, ϕ	0.90	AISC 360 J4.4
Design Strength, ϕR_n (kips)	3524.5	
Is $\phi_d R_n > F_{su}$?	OK	
Continuity Plate Welds to Column Flange		
Continuity plate weld to flange is CJP	OK	

Continuity Plate Welds to Column Web

k_{det} for column (in)	4.13	
Contact length along column web, $L_{cpw}=d_c-2(k_{det}+1.5)$ (in)	7.1	1.5" is Seismic clip per AWS D1.8 4.1
k_1 for column (in)	2.13	
Contact length along column flange, $L_{cpf}=(w_{cp}+t_{cw}/2)-(k_1+0.5)$ (in)	-1.74	
Design Strength of CP contact area in tension, $R_{u\text{-}a}=0.9(2L_{cpf})t_{cp}F_{yc\&dp}$ (kips)	0.0	AISC 341 E3.6f(3)(a)
Des. Strng of CP contact area in shear, $R_{u\text{-}b}=0.9(2L_{cpw})t_{cp}(0.6F_{yc\&dp})$ (kips)	0.0	AISC 341 E3.6f(3)(b)
Design Strength of Panel Zone from Step 2.7, $R_{u\text{-}c}$ (kips)	1309.8	AISC 341 E3.6f(3)(c)
Number of beam flanges transmitting force to continuity plate, N_b	1	
Expected Strength of Flanges, $R_{u\text{-}d}=R_yF_yt_{bf}b_{fb}\ N_b$ (kips)	620.4	AISC 341 E3.6f(3)(d)
Continuity plate weld to web demand, $R_u=\min\{R_{u\text{-}a}, R_{u\text{-}b}, R_{u\text{-}c}, R_{u\text{-}d}\}$ (kips)	0.0	AISC 341 E3.6f(3)
$R_n = 0.6\ F_{EXX}\ [t_{wcp}/\mathrm{sqrt}(2)]\ 4L_{cpw}$ (kips)	418.7	welds for both continuity plates
for fillet welds ϕ	0.75	
ϕR_n	314.1	
Is $\phi R_n > R_u$	OK	

Step 2.7 - Check Panel Zone

Panel Zone Strength

Number of beams, N_b	1	
Panel Zone Shear, $V_{upz} = N_b\,F_{fu} - V_c - P_{ub}/2$ (kips)	1147.0	
Column squash load, $P_y=F_yA_c$ (kips)	5850	
Column compression, $P_u=P_{uc}+V_u$ (kips)	235	
Ratio of P_u/P_y (kips)	0.04	
Panel zone thickness, $t_{pz}=t_{cw}+t_{dp}$ (in)	1.77	
$R_n = 0.6F_y\ d_c\ t_{pz}\ (1+3b_{cf}\ t_{cf}^{\ 2}\ /\ (d_{bm}\ d_c\ t_{pz}))$ (kips)	1309.8	AISC 360 J10.6 - see assumptions
for panel zone, ϕ	1.00	AISC 341 E3.6e(1)
Design Strength, ϕR_n (kips)	1309.8	
Is $\phi_d R_n > V_{upz}$?	OK	

Min Panel Zone Thickness

$d_z=d_{bm}-2t_{fb}$ (in)	34.02	
$w_z=d_c-2t_{cf}$ (in)	12.6	
$t_{min}=(d_z+w_z)/90$ (in)	0.518	
Is $t_{dp}>t_{min}$?	No Good	No doubler plates
Is $t_{cw}>t_{min}$?	OK	

Doubler Plate Welds Not Designed

Step 2.8 - Strong Column Weak Beam

V_u from Step 1.1	234.7
$M_v=V_u(S_h+d_c/2)$ (k-in)	5497.7
Number of Beams, N_b	1
$\Sigma M_{pb}{}^*=N_b(M_{pr}+M_v)$ (k-in)	42246
Column Compression from Step 2.7, P_u (kips)	235
Number of Columns, N_c	2
$\Sigma M_{pc}{}^*=N_cZ_c(F_y-P_u/A_c)$ (k-in)	76886
$\Sigma M_{pc}{}^*/\Sigma M_{pb}{}^*$	1.82
Is $\Sigma M_{pc}{}^*/\Sigma M_{pb}{}^*>1.0$?	OK