THE OHIO STATE UNIVERSITY

Verification of the results from IDEA StatiCa for steel connections according to the U.S. design codes

Authors:

Baris Kasapoglu, Ph.D. student (Department of Civil, Environmental and Geodetic Engineering)

Rafael Arthur Giorjao, Ph.D. (Department of Materials Science and Engineering)

Ali Nassiri, Ph.D. (Department of Integrated Systems Engineering)

Halil Sezen, Ph.D. (Department of Civil, Environmental and Geodetic Engineering)

June 2021

Introduction:

In the field of structural and civil engineering, understanding the structural behavior and integrity of the building is critical to ensure the safety of its occupants. However, it is a challenge to analyze and determine the behavior of a complex structure when it is subjected to a variety of loading conditions using conventional analytical methods. Therefore, Finite Element Analysis (FEA) is a valuable tool for numerically modeling physical structures that are too complex for analytical solutions. The overarching objective of this report is to evaluate the FEA results obtained from the IDEA StatiCa software package for three groups of common steel connections used in the United States (i.e., simple, semi-rigid, and rigid connections), and compare them with available experimental data and the results calculated from another FEA software, ABAQUS. The beam-column joint response obtained from the IDEA StatiCa software is then compared with the design calculations performed following the requirements of the AISC 360, Specification for Structural Steel Building (2016), and AISC Steel Construction Manual (2017) codes.

This report includes four chapters. In Chapters 1–3, an experimentally validated connection design was chosen from the literature for each connection type as a base model. The code design checks and calculations were performed according to the U.S. building codes for each base model and its ten variations. Then, the results were compared with the IDEA StatiCa predictions. Additionally, the FEA results from the IDEA StatiCa were compared with those from ABAQUS. All of the required steps and details of all geometric and design checks according to the AISC design codes are included in appendices. The last chapter contains the overall evaluation of the IDEA StatiCa software in terms of its accuracy and compatibility with the requirements of the U.S. building codes for the steel connections.

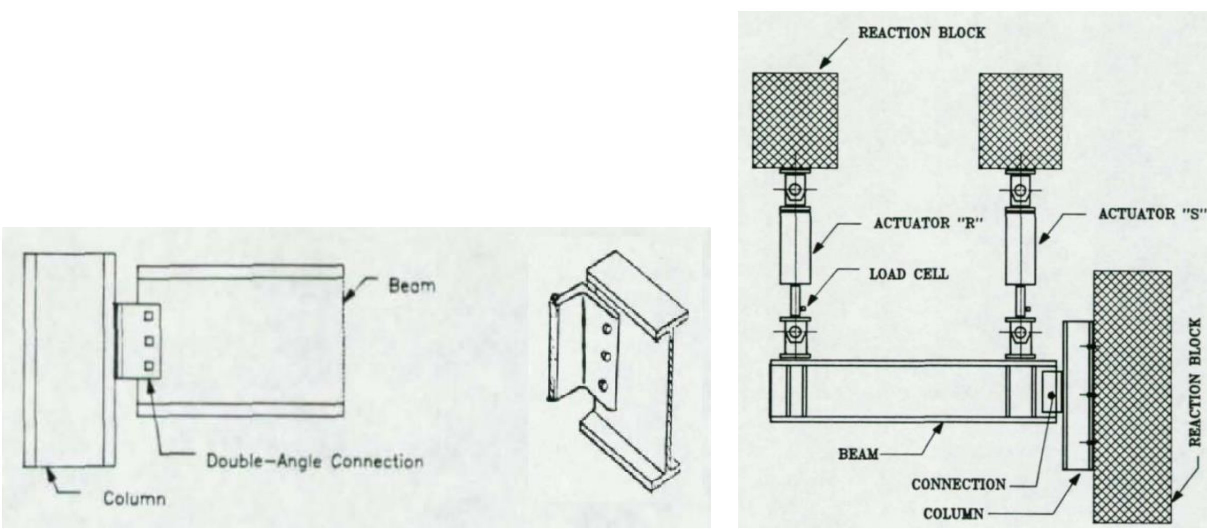
CHAPTER 1 SIMPLE CONNECTIONS

1.1. Introduction

In this study, the design strength capacities of ten simple, hinge or pin connection specimens were calculated following the requirements of the AISC 360 (2016) and AISC Construction Manual (2017). Four test specimens were selected from the experimental study performed by McMullin and Astaneh (1988) in the Department of Civil Engineering at the University of California, Berkeley. Six additional models were developed for verification purposes by modifying the parameters based on the available test specimens. Then, the baseline model was analyzed using ABAQUS (2020) and IDEA StatiCa (Version 20.1.3471.1) and the results were compared.

1.2. Experimental Study

Seven full-scale steel beam-column connection specimens were tested, and results were presented in McMullin and Astaneh (1988). Each connection specimen was bolted to the beam and welded to the column with double angle sections. Double angle connection used in these experiments and test set-up are shown in Figure 1.1(a). As shown in Figure 1.1(b), the main goal of these tests is to apply only shear force in the connection with very small bending or moment. To achieve this objective, the actuator S near the connection applies the shear force. The actuator R near the tip of the cantilever aims to keep the beam horizontal and limit the rotation (bending) of the connection. The properties of the seven double angle connection specimens are provided in Table 1.1.



(a) **(b)**

Figure 1.1: (a) Double angle connection specimens, and (b) test set-up (McMullin and Astaneh, 1988)

Table 1.1: Properties of double angle connections

Test No.	Number of Bolts	Bolt Size	Weld Size	Connection Length	Weld Length	Angle Size	Connection Detail
4	7	3/4	1/4	20.5	20.5	4 x 3.5 x 3/8	I
5	5	3/4	1/4	14.5	14.5	4 x 3.5 x 3/8	I
6	3	3/4	1/4	8.5	8.5	4 x 4 x 3/8	I
7	7	7/8	5/16	20.5	26.0	4 x 4 x 3/8	III
8	5	7/8	5/16	14.5	20.0	4 x 4 x 3/8	III
9	5	7/8	5/16	14.5	14.5	4 x 4 x 3/8	I
10	5	7/8	5/16	14.5	14.5	4 x 4 x 3/8	II

Beams and columns were made of ASTM A992 steel, and angles were manufactured from ASTM A36 steel. The material properties of the members are presented in Table 1.2. All bolts were A325 bolts with threads excluded from shear planes. The edge distance of bolts was 1.25 inches from the top and bottoms of angle sections, while the bolt spacing was 3.0 in. The weld size of each specimen was 0.25 inches and welded to the column using E-70XX electrodes with a nominal strength of 70 ksi.

Table 1.2: Material properties

Members	Strength (ksi)	
Columns and Beams (A992)	Yield Strength, F_y	50
	Ultimate Strength, F_u	65
Angles (A36)	Yield Strength, F_y	36
	Ultimate Strength, F_u	58
Bolts A325 (threads excluded)	Nominal Tensile Strength, F_{nt}	90
	Nominal Shear Strength, F_{nv}	68

Four double angle beam-column connection specimens, Tests No. 4, 5, 6 and 9, were selected out of the eight specimens tested (Table 1.1). The properties, failure modes, and shear capacities measured during the testing of these four specimens are provided in Table 1.3. Ultimate failure of all four specimens was due to weld failure, including in the heat affected zone (HAZ).

Table 1.3: Summary of test results

Test No.	Properties of Specimens					Test Results at Ultimate Load		Failure Mode
	Beam	Column	Double Angle (in.)	Number of Bolts	Bolt Diameter (in.)	Shear (kips)	Rotation (rad)	
4	W24x68	W10x77	4x3.5x3/8	7	3/4	230	0.0257	Weld sheared along its full length in the HAZ
5	W24x68	W10x77	4x3.5x3/8	5	3/4	205	0.0315	Weld cracked in HAZ of angle
6	W24x68	W10x77	4x3.5x3/8	3	3/4	117	0.0414	Weld cracked along the top length
9	W24x68	W10x77	4x4x3/8	5	7/8	192	0.0332	Weld cracks from top down

Shear force-rotation and moment-shear diagrams measured during the experiments (McMullin and Astaneh, 1988) are shown in Figures 1.2 through 1.5 for each of the four specimens modeled and analyzed in this report.

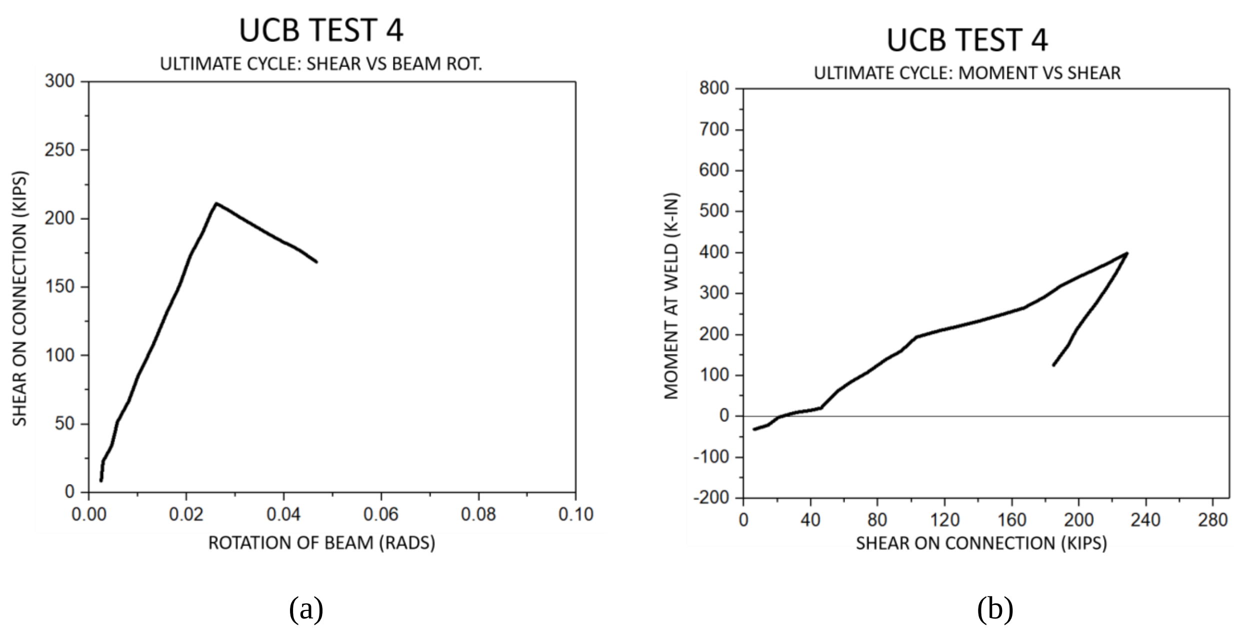
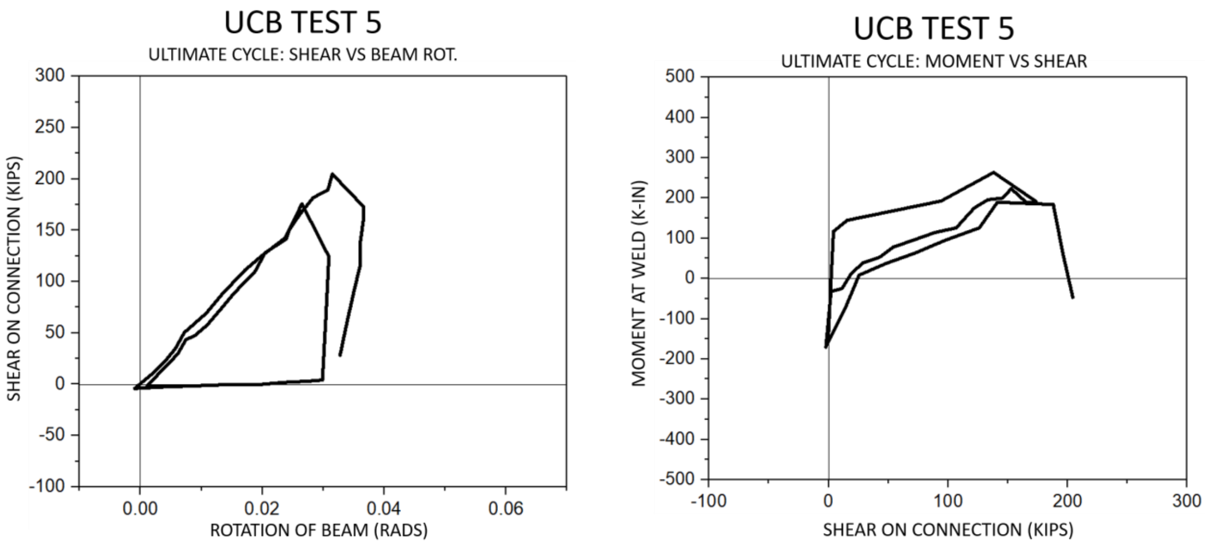
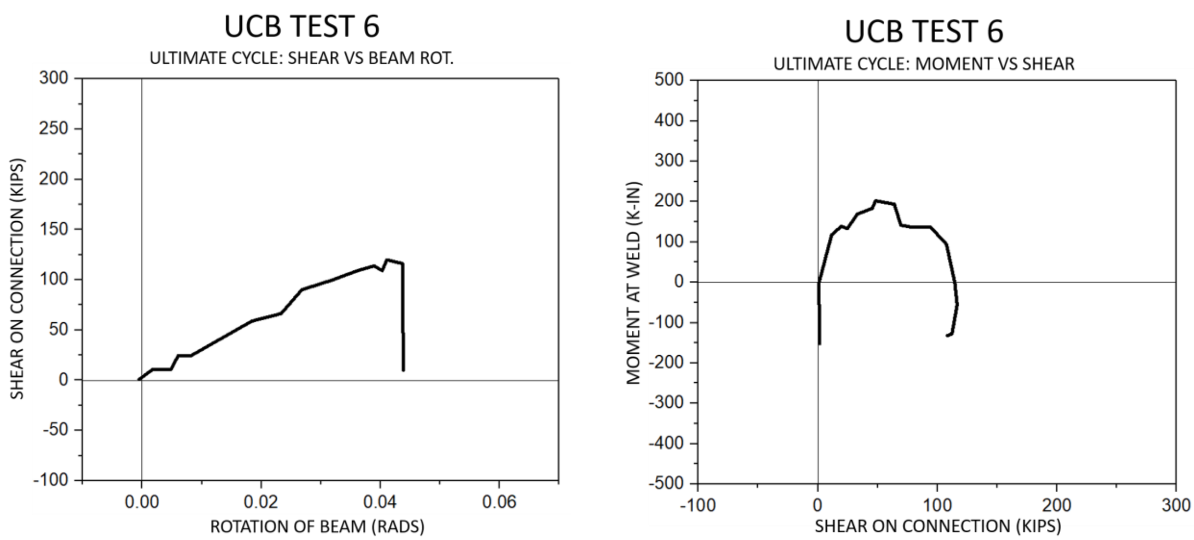


Figure 1.2: Test specimen No. 4: a) measured shear on connection-rotation of beam relationship, and b) moment at weld-shear on connection relationship



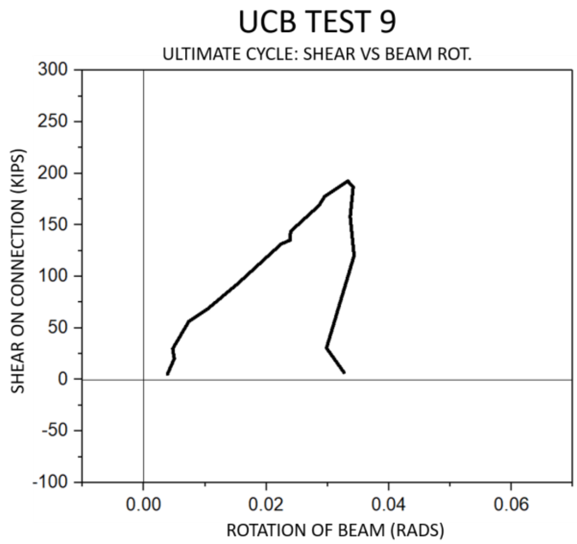
(a) (b)

Figure 1.3: Test specimen No. 5: a) measured shear on connection-rotation of beam relationship, and b) moment at weld-shear on connection relationship

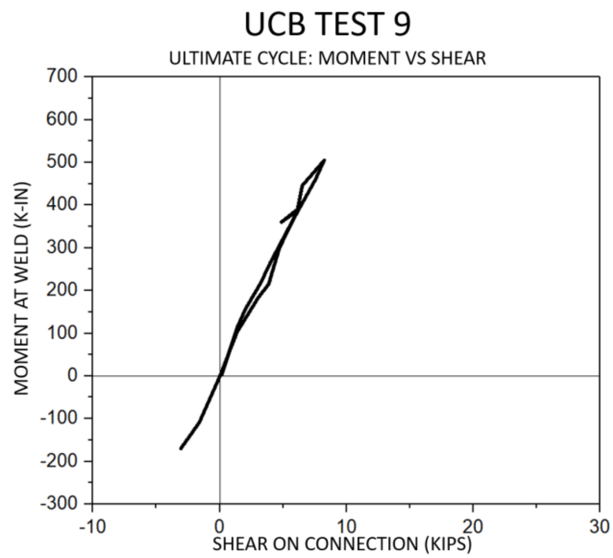


(a) (b)

Figure 1.4: Test specimen No. 6: a) measured shear on connection-rotation of beam relationship, and b) moment at weld-shear on connection relationship



(a)



(b)

Figure 1.5: Test specimen No. 9: a) measured shear on connection-rotation of beam relationship, and b) moment at weld-shear on connection relationship

1.2.1 Instrumentation

The instrumentation used in this experimental study are (see Figure 1.6):

- Three Linear Variable Displacement Transducers (LVDT)
- Three Linear Potentiometers (LP)
- Two load cells

LVDT 7 was used to measure the separation of the top of the angle relative to the column flange, while LVDT 5, 6, 8 and 9 measured the relative displacement between the column flange and the beam flange. The small rotations (less than 0.02 rad.) of the beam can be calculated with the following equation:

Rotation = (LVDT 5 + LVDT 6 + LVDT 8 + LVDT 9) / (2 x distance between LVDT centerlines) (1)

The deflection at the end of the beam was measured with LP #3. The deflection across from the actuator (actuator S in Figure 1.1) was measured using LP #4, while LP #10 was used to measure the displacement at the bolt line in the direction of the applied shear load. The rotation of the beam (larger than 0.01 rad.) can be calculated with the following equation:

Rotation = (LP #3 + LP #10) / separation (2)

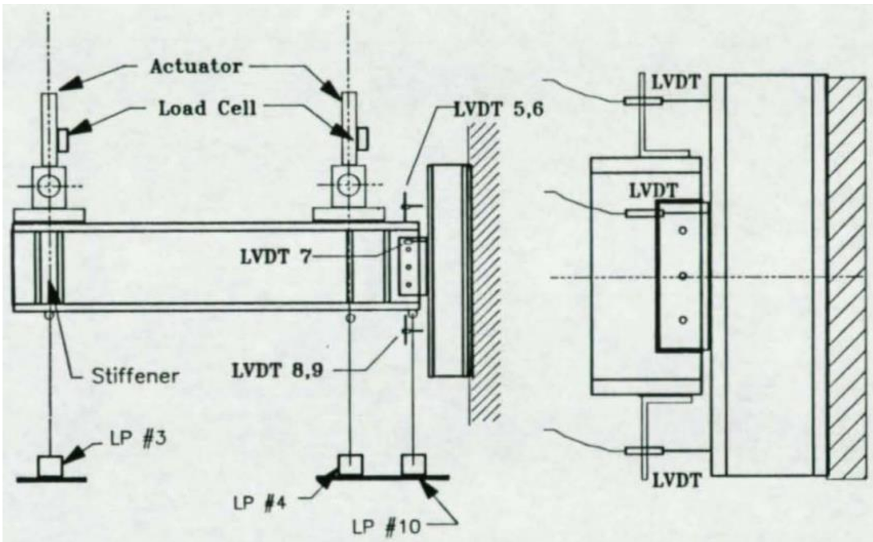
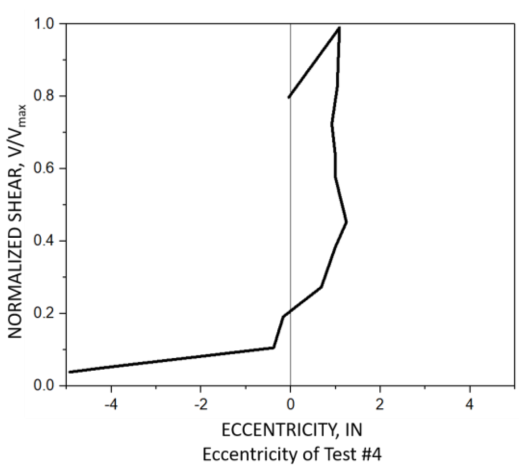


Figure 1.6: Diagram of the instrumentation used during the experiment (McMullin and Astaneh, 1988)

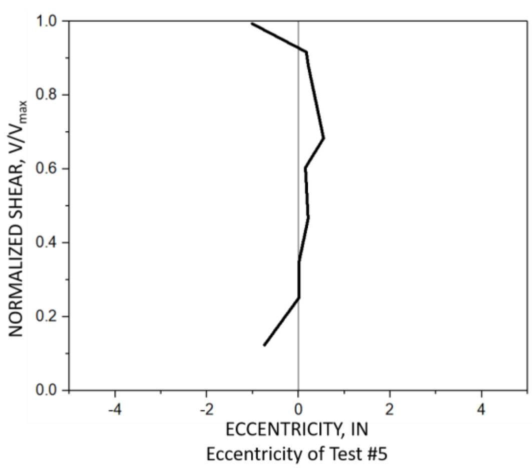
In this study, it is assumed that the moment at the weld of the connection is the moment transferred into column. If the location of the inflection point is known, the moment, M, transferred from the beam can be calculated by multiplying applied shear force, V, by the distance between the inflection point and the column, e.

$$M = V \cdot e \tag{3}$$

However, it is not possible to determine the location of the inflection point because of the complexity of the connection and loading. From the results obtained during testing, the actual location of the inflection point can be computed by static analysis for any load during the loading history and can be obtained as a function of the normalized shear, V/V_{max} on the connection.

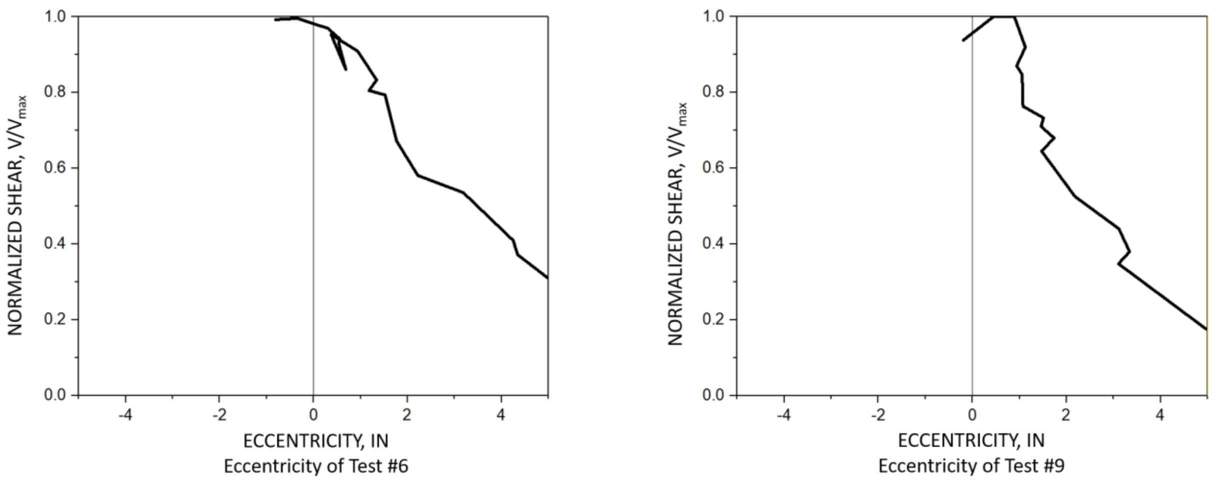


(a)



(b)

Figure 1.7: (a) Normalized shear-eccentricity of Test No 4, and (b) Normalized shear-eccentricity of Test No 5 (McMullin and Astaneh, 1988)



(a) (b)

Figure 1.8: (a) Normalized shear-eccentricity of Test No 6, and (b) Normalized shear-eccentricity of Test No 9 (McMullin and Astaneh, 1988)

1.3. Code Design Calculations and Comparisons

The design strength capacities (ϕR_n) of the connections were calculated following the requirements of AISC Specification for Structural Steel Buildings (AISC 360, 2016) and AISC Steel Construction Manual (AISC Manual, 2017). The nominal strength, R_n, and the corresponding resistance factor, ϕ, for each connection design limit state for load and resistance factored design (LRFD) are provided in Chapter J of AISC 360. The following 13 design checks were performed according to the LRFD design equations included in AISC 360 or AISC Manual.

- Bolt Shear Check (Eq. J3-1, AISC 360-16)
- Bolt Tensile Check (Eq. J3-1, AISC 360-16)
- Bolt Bearing on Beam (AISC 360-16, Eq. J3-6a)
- Bolt Tearout on Beam (AISC 360-16, Eq. J3-6c)
- Bolt Bearing on Angles (AISC 360-16, Eq. J3-6a)
- Bolt Tearout on Angles (AISC 360-16, Eq. J3-6c)
- Shear Rupture on Angles (Beam Side) (AISC 360-16, Eq. J4-4)
- Block Shear on Angles (Beam Side) (AISC 360-16, Eq. J4-5)
- Shear Yielding on Angles (AISC 360-16, Eq. J4-3)
- Shear Yielding on Beam (AISC 360-16, Eq. J4-3)
- Welds Rupture on Angles (Support Side) (Page 9-5, AISC Manual)
- Weld Capacity (Page 10–11, AISC Manual)
- Weld Capacity (no eccentricity) (AISC 360-16, Eq. J4-2)

1.3.1 LRFD Design Strength Capacities of four Test Specimens

The design strength capacities (ϕR_n) of the four selected test specimens were calculated by following the AISC LRFD code requirements, i.e., AISC 360-16 and AISC Manual. The properties of the four selected test specimens and their design strength capacities are provided in Tables 1.4 and 1.5, respectively. The detailed design strength calculations for Test No. 4 are provided in Appendix A.

The minimum yield stress, F_y, and the specified minimum tensile strength, F_u, of the materials of the test specimens are obtained from AISC Construction Manual (2017) Table 2-4. The clear distance between web fillets, T, the thickness of web of beam, t_w, the depth of beam, d, the distance from outer face of flange to web tow of fillet, k, and the thickness of flange of column, t_f, are obtained from Table 1-1 in AISC Construction Manual (2017). The lengths of the double angles, L, are provided in the experiment report (McMullin and Astaneh, 1988).

Out of the calculated design capacities for the four test specimens, the lowest shear capacity was selected in Table 1.5. Comparison of the calculated strengths in Table 1.5 shows that the design capacity of each of these four double angle specimens was controlled by weld failure according to AISC Manual, i.e., the lowest strengths are in the second row from the bottom.

Table 1.4: Properties of Test Specimens

Properties of Test Specimens		Test No. 4	Test No. 5	Test No. 6	Test No. 9
Beam	Type	W24x68	W24x68	W24x68	W24x68
	t_w (in.)	0.415	0.415	0.415	0.415
	d (in.)	23.7	23.7	23.7	23.7
	T (in.)	20.75	20.75	20.75	20.75
	k (in.)	17/16	17/16	17/16	17/16
	F_y (ksi)	50	50	50	50
	F_u (ksi)	65	65	65	65
Column	Type	W10x77	W10x77	W10x77	W10x77
	t_f (in.)	0.87	0.87	0.87	0.87
	F_y (ksi)	50	50	50	50
	F_u (ksi)	65	65	65	65
Double Angle	Dimension (in.)	4x3-1/2x3/8	4x3-1/2x3/8	4x3-1/2x3/8	4x4x3/8
	leg (in.)	4	4	4	4
	L (in.)	20.5	14.5	8.5	14.5
	F_y (ksi)	36	36	36	36
	F_u (ksi)	58	58	58	58
Bolts	Type	A325	A325	A325	A325
	Diameter	0.75	0.75	0.75	0.875
	Number	7	5	3	5
	Threads	Excluded	Excluded	Excluded	Excluded
	Spacing (in.)	3	3	3	3
	Edge dist. (in.)	1.25	1.25	1.25	1.25
Weld	Size (in.)	0.25	0.25	0.25	0.3125
	Length (in.)	20.5	14.5	8.5	14.5
	Electrode	E70XX	E70XX	E70XX	E70XX

Table 1.5: Calculated LRFD Design Strength Capacities of Test Specimens

Design Checks	Test No. 4	Test No. 5	Test No. 6	Test No. 9
Bolt Shear (kips)	315.6	225.4	135.2	306.5
Bolt Tensile (kips)	417.8	298.4	179.0	405.7
Bolt Bearing on Beam (kips)	254.9	182.1	109.3	212.4
Bolt Tearout on Beam (kips)	378.2	344.7	311.2	330.7
Bolt Bearing on Angles (kips)	411.1	293.6	176.2	342.6
Bolt Tearout on Angles (kips)	547.3	375.8	204.4	235.4
Shear Rupture on Angles (kips)	281.4	198.4	114.8	185.8
Block Shear on Angles (kips)	260.5	187.6	114.7	185.4
Shear Yielding on Angles (kips)	332.2	235.0	137.8	235.0
Shear Yielding on Beam (kips)	295.2	295.2	295.2	295.2
Minimum Thickness for Connecting Element Rupture Strength (in.)	0.21	0.21	0.21	0.26
Weld Capacity by AISC Manual (pp. 10-11), (kips)	186.8	114.6	48.1	126.6
Weld Capacity (no eccentricity) by AISC 360-16 Eq. J2.4 (kips)	228.3	161.5	94.7	201.9

1.3.2 LRFD Design Strength Capacities of Six Additional Connection Models

The design strength capacities (ϕR_n) of six additional models were calculated by following AISC LRFD code requirements. The properties of these six connections and their calculated design capacities are provided in Tables 1.6 and 1.7, respectively. Test No 4 was selected as a baseline model. The modified properties were bolted and italicized in Table 1.6.

Comparison of the calculated design capacities of the six additional models in Table 1.7 showed that, in all six additional specimens, the lowest shear capacity corresponds to the weld capacity calculated from the equations in the AISC Manual (2017).

Table 1.6: Properties of Test Specimens

		Model 1	Model 2	Model 3	Model 4	Model 5	Model 6
Beam	Type	W24x68	W24x68	W24x68	W24x68	W24x68	W24x68
	t_w (in.)	0.415	0.415	0.415	0.415	0.415	0.415
	d (in.)	23.7	23.7	23.7	23.7	23.7	23.7
	T (in.)	20.75	20.75	20.75	20.75	20.75	20.75
	k (in.)	17/16	17/16	17/16	17/16	17/16	17/16
	F_y (ksi)	50	50	50	50	50	50
	F_u (ksi)	65	65	65	65	65	65
Column	Type	W10x77	W10x77	W10x77	W10x77	W10x77	W10x77
	t_f (in.)	0.87	0.87	0.87	0.87	0.87	0.87
	F_y	50	50	50	50	50	50
	F_u	65	65	65	65	65	65
Double Angle	Dimen-sion (in.)	4x3.5x3/8	*4x3.5x1/2*	4x3.5x3/8	4x3.5x3/8	4x3.5x3/8	4x4x3/8
	leg (in)	4	4	4	4	4	4
	L (in.)	20.5	20.5	20.5	20.5	20.5	20.5
	F_y (ksi)	36	36	36	36	36	36
	F_u (ksi)	58	58	58	58	58	58
Bolts	Type	A325-X	A325-X	A325-X	*A325-N*	A325-X	A325-X
	Diame-ter (in.)	0.75	0.75	*0.5*	0.75	*1*	*0.875*
	Number	7	7	7	7	7	7
	Threads	Excluded	Excluded	Excluded	*Not Excluded*	Excluded	Excluded
	Spacing (in.)	3	3	3	3	3	3
	Edge distance (in.)	1.25	1.25	1.25	1.25	1.25	1.25
Weld	Size (in.)	*0.3125*	0.25	0.25	0.25	0.25	*0.3125*
	Length (in.)	20.5	20.5	20.5	20.5	20.5	20.5
	Electrode	E70XX	E70XX	E70XX	E70XX	E70XX	E70XX

Table 1.7: Calculated LRFD design strength capacities of test specimens

Design Checks	Model 1	Model 2	Model 3	Model 4	Model 5	Model 6
Bolt Shear (kips)	315.6	315.6	139.9	250.6	560.6	429.1
Bolt Tensile (kips)	417.8	417.8	185.2	417.8	741.9	568.0
Bolt Bearing on Beam (kips)	254.9	254.9	169.9	254.9	339.9	297.4
Bolt Tearout on Beam (kips)	378.2	378.2	417.8	378.2	328.7	354.8
Bolt Bearing on Angles (kips)	411.1	548.1	274.1	411.1	548.1	479.6
Bolt Tearout on Angles (kips)	547.3	729.7	407.4	547.3	467.4	514.5
Shear Rupture on Angles (kips)	281.4	375.2	315.7	281.4	238.6	264.3
Block Shear on Angles (kips)	260.5	347.1	264.5	260.5	247.1	258.3
Shear Yielding on Angles (kips)	332.2	442.8	332.2	332.2	332.2	332.2
Shear Yielding on Beam (kips)	295.2	295.2	295.2	295.2	295.2	295.2
Minimum Thickness for Connecting Element Rupture Strength (in.)	0.26	0.21	0.21	0.21	0.21	0.26
Weld Capacity by AISC Manual (pp. 10-11), (kips)	233.5	186.8	186.8	186.8	186.8	214.4
Weld Capacity (no eccentricity) by AISC 360-16 Eq. J2.4 (kips)	285.4	228.1	228.1	228.1	228.1	285.4

1.3.3 Calculated ASD Design Strength Capacities

According to allowable strength design (ASD), the allowable strength (R_n/Ω) is calculated by dividing the nominal strength, R_n by the safety factor, Ω. The allowable strength capacities of connection specimen Test No. 4 are calculated by following the AISC ASD code requirements. The properties of this test specimen were given in Table 1.4. The calculated ASD design strength capacities (R_n/Ω) of the specimen are provided in Table 1.8. The calculated lowest strength is the weld capacity (124.5 kips) for this specimen. For ASD design purposes, this capacity (124.5 kips) should be compared with the load demand calculated using the ASD design load combinations (AISC Manual, 2017). The detailed design strength calculations for Test No. 4 are provided in Appendix B.

Table 1.8: Calculated ASD design strength capacities of Test No. 4

Design Checks	Test No. 4
Bolt Shear (kips)	210.4
Bolt Tensile (kips)	278.5
Bolt Bearing on Beam (kips)	170.0
Bolt Tearout on Beam (kips)	252.2
Bolt Bearing on Angles (kips)	274.1
Bolt Tearout on Angles (kips)	364.9
Shear Rupture on Angles (kips)	187.6
Block Shear on Angles (kips)	173.7
Shear Yielding on Angles (kips)	221.5
Shear Yielding on Beam (kips)	196.8
Minimum Thickness for Connecting Element Rupture Strength (in.)	0.21
Weld Capacity (kips)	124.5
Weld Capacity (no eccentricity) by AISC 360-16 Eq. J2.4 (kips)	152.0

1.4. IDEA StatiCa Analysis

IDEA StatiCa checks four different failure scenarios of this steel connection type: (1) plate failure, (2) bolt failure, (3) weld failure, and (4) buckling. The selected four test specimens (Table 1.4) and six additional models (Table 1.6) were modeled in IDEA StatiCa and analyzed under a shear force, as shown in Figure 1.9. In the software, the location of the shear force can be arbitrarily selected. Two shear force locations were investigated: (1) in bolts, and (2) at the column face.

The shear force was applied incrementally on the vertical line connecting the bolts and on vertical welding material in different models until the connections reached their shear capacities in IDEA StatiCa. In this way, the shear capacities of the four tested specimens and the six additional models were computed as presented in Tables 1.9 and 1.10.

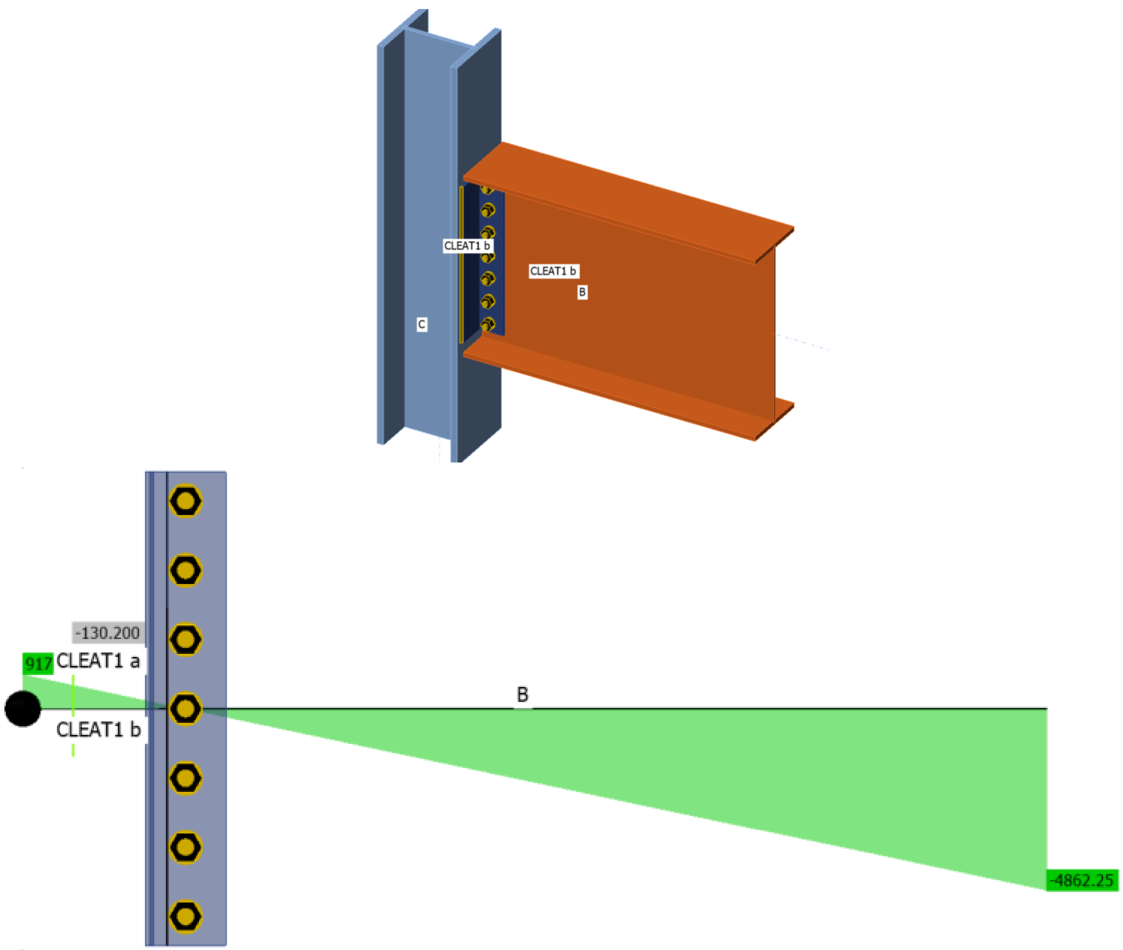


Figure 1.9: IDEA StatiCa model setup (top) and the wireframe model in bottom for the double angle connection specimen, Test No 4 (force applied on the centroid of the bolts group)

Table 1.9: Shear capacities of selected test specimens calculated by IDEA StatiCa

Strength Capacity by IDEA StatiCa	**Test No. 4**	**Test No. 5**	**Test No. 6**	**Test No. 9**
Strength by IDEA StatiCa - force applied on bolts (kips)	130.2	73.4	31.3	61.3
Failure mode - force applied on bolts	Plate failure (limit plastic strain, 5%)	Plate failure (limit plastic strain, 5%)	Plate failure (limit plastic strain, 5%)	Plate failure (limit plastic strain, 5%)
Strength by IDEA StatiCa when force is applied on welding (kips)	216.6	145.4	74.8	168.0
Failure mode - force applied on welding	Weld failure	Weld failure	Weld failure	Plate failure (limit plastic strain, 5%)

Table 1.10: Shear capacities of the six variation models calculated by IDEA StatiCa

Strength Capacity by IDEA StatiCa	Model 1	Model 2	Model 3	Model 4	Model 5	Model 6
Strength by IDEA StatiCa force applied on bolts (kips)	127.3	200.1	129.1	130.2	132.3	127.9
Failure mode force applied on bolts	Plate failure (limit plastic strain, 5%)	Weld failure	Bolt shear failure	Plate failure (limit plastic strain, 5%)	Plate failure (limit plastic strain, 5%)	Plate failure (limit plastic strain, 5%)
Strength by IDEA StatiCa force applied on welding (kips)	229.0	226.7	136.0	216.5	213.3	234.1
Failure mode force applied on welding	Plate failure (limit plastic strain, 5%)	Weld failure	Bolt shear failure	Weld failure	Bolt bearing failure	Bolt bearing failure

For a new user, modeling the first connection (Test No. 4) takes approximately 8–10 minutes. Since each of the other connections were modeled by modifying the first one, each took 2–3 minutes. The software completed the calculation for each connection in 5–7 seconds by a personal computer. The result screen pointing out the failure mode and the deformed shapes (deformation scale 10) of finite element models from IDEA StatiCa are shown in Appendix C.

1.5. ABAQUS Modeling and Analysis

The aim of this section is to compare the results from IDEA StatiCa with those from another commercial finite element code. In this study, ABAQUS software package (version 2020) was utilized. ABAQUS is a robust general-purpose FEA software package suitable for analyzing whole range of static, dynamic, and nonlinear problems.

In this study, Test No. 4 as described in Section 1.2 was chosen as a base model. Numerical simulations with almost identical conditions (i.e., in terms of material properties, boundary condition, and loading) were carried out using both IDEA StatiCa and ABAQUS. The model was initially designed in IDEA StatiCa and then the assembly (including beam, column, and double angles) was imported to ABAQUS using the IDEA StatiCa's viewer platform. Afterward, a simplified model for the bolt and weld were designed and added to the ABAQUS model (see Figure 1.10).

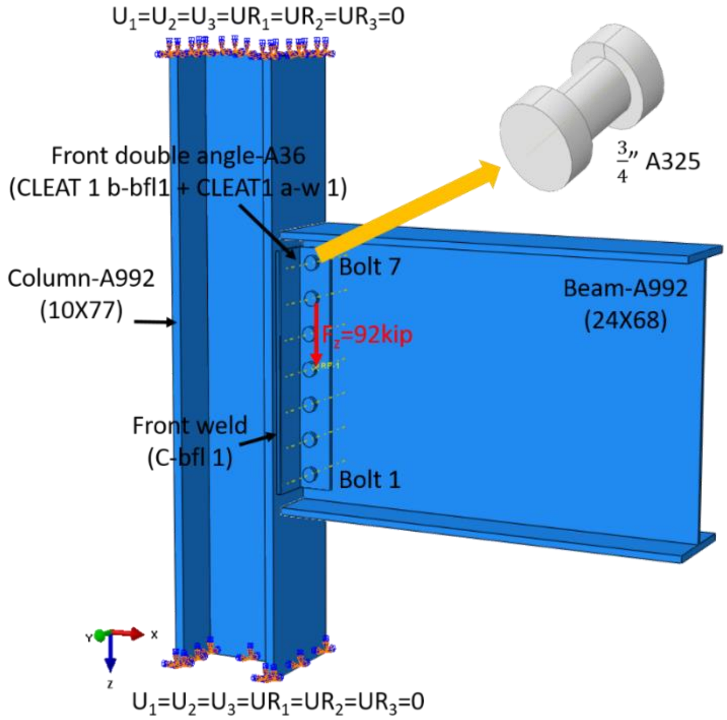


Figure 1.10: Model setup in ABAQUS

In ABAQUS, the element type was C3D8R (3D stress, 8-node linear brick, reduced integration), and a total of 293,294 elements were generated in the model (see Table 1.11 and Figure 1.11 for more details).

Table 1.11. Number of finite elements in the ABAQUS model

Item	Number of Elements
Column	185,902
Beam	19,430
Bolt	6,304
Weld	1,820
Double angle	40,194

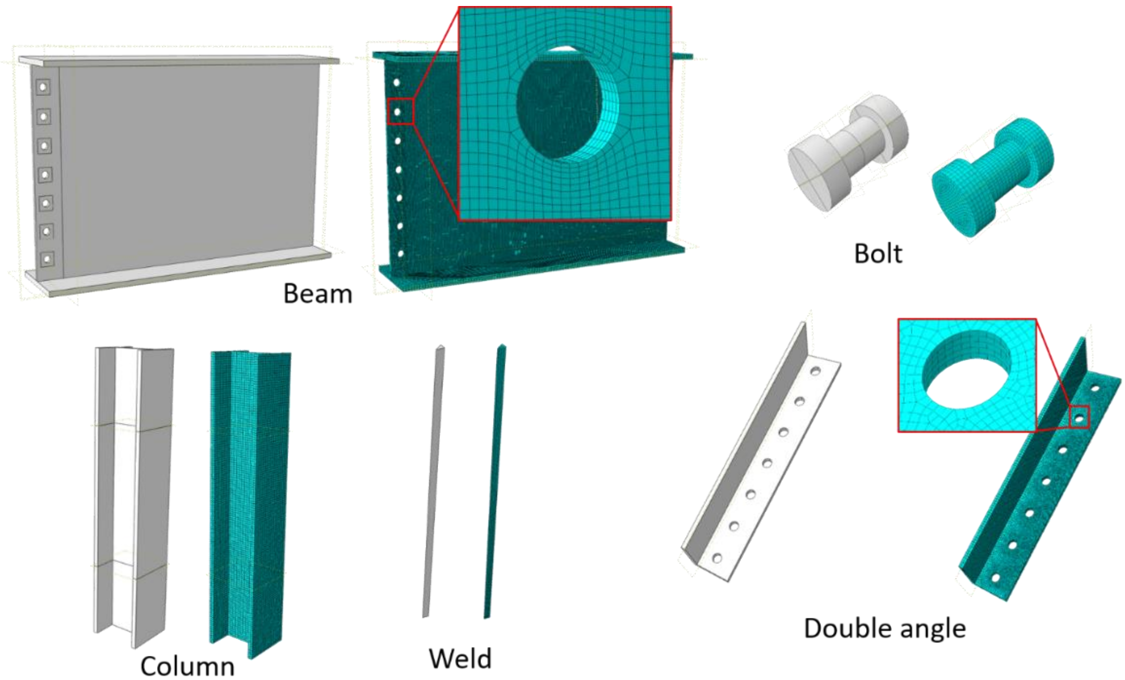


Figure 1.11: ABAQUS model mesh densities

As described in the previous section, different results can be achieved depending on the position of the acting vertical shear force. Therefore, two cases were defined and investigated using the ABAQUS model. In case 1, the vertical shear force of 130.2 kips was applied on the centroid of the bolt group (x = 7.045 in., x is the distance from the centerline of the column). In case 2, the vertical shear force of 216 kips was applied on the weld lines (x = 5.5 in.). It should be mentioned that in the second case, the beam and column were slightly shorter than the first case to mimic the experimental test. In both cases, top and bottom of the column were fixed as a boundary condition (see Figure 1.10). The contact between the parts was defined as surface-to-surface with finite sliding formulation. Friction was defined with a penalty method, and a Coulomb friction coefficient of $\mu = 0.3$ was used everywhere except between the column face and double angles in which the contact is assumed to be frictionless. Also, tie constraint was applied between the weld lines and the attaching parts (i.e., column and double angels).

The material behavior was modeled using a bi-linear plasticity in ABAQUS. Other parameters, including density, elastic modulus, and Poisson's ratio, were taken from the IDEA StatiCa materials library. The numerical simulations were carried out on two processors (Intel Xenon (R) CPU E5-2698 v4 @ 2.20GHz). Each simulation took approximately 155 minutes. Figure 1.12 depicts the comparison of the predicted von-Mises stresses between the IDEA StatiCa and ABAQUS models for both cases.

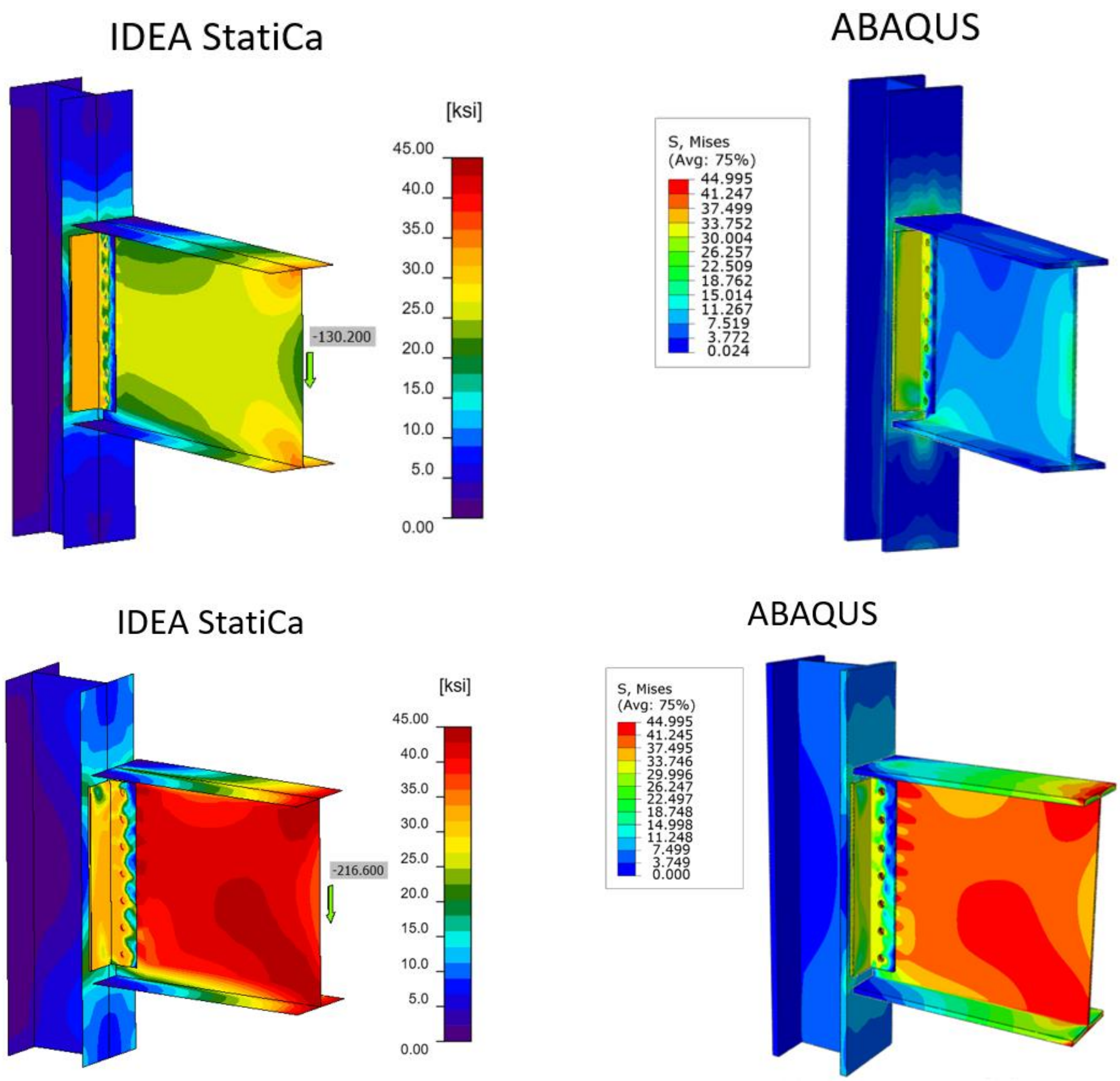


Figure 1.12: Predicted von Mises stress between IDEA StatiCa and ABAQUS models; case 1 (top row): shear load was applied on the centroid of the bolt group, and case 2 (bottom row): shear load was applied on the weld lines

1.6 Summary and Comparison of Results

1.6.1 Comparison of IDEA StatiCa and AISC Design Strength Capacities

Two different weld capacities were calculated for each test specimen following the AISC LRFD design requirements. For the same four test specimens (Table 1.3), two different weld capacities were calculated from the IDEA StatiCa models by applying the shear force at different locations. In all loading scenarios, it was found that the weakest component of the connections was welding. The controlling or smallest calculated strengths corresponding to the weld capacities are presented and compared with the ultimate welding shear capacity measured during the experiment in Table 1.12.

Weld capacities of the test specimens were computed in two different ways by following the AISC LRFD code requirements (AISC 360-16 and AISC Manual, 2017). For Test No. 4, if Equation J2.4 in AISC 360-16 is followed, the weld design capacity of the specimen is calculated as 228.3 kips. In this solution, no eccentricity is taken into account. To compare this approach with the IDEA StatiCa analysis, the vertical shear force was applied on the welding (parallel to the weld line) and the welding capacity of this specimen was calculated as 216.6 kips, which is very close the one calculated from Equation J2.4 in AISC 360-16 (228.3 kips in Table 1.12).

When the shear force is applied on the bolts (external vertical force parallel to the bolt line) in the IDEA StatiCa model, the connection capacity was computed as 130.2 kips. If the welding capacity is calculated by following the LRFD weld strength equation (Page 10-11 of AISC Construction Manual, 2017), which considers the eccentricity of the loading on the support side, the welding capacity of the specimen is calculated as 186.8 kips (Line 1 in Table 1.12). However, conservatively this AISC LRFD equation does not account for the eccentricity resulting from the gap between the bolts and welding. It is believed that this assumption is the reason for the difference between the results calculated from IDEA StatiCa and LRFD strength equation in the AISC Manual (2017).

Table 1.12: Comparison of measured shear capacities with those calculated from AISC LRFD design equations and IDEA StatiCa analysis

Strength Capacities	Test No. 4	Test No. 5	Test No. 6	Test No. 9
Strength by IDEA StatiCa - force applied on bolts (kips)	130.2	73.4	31.3	61.3
Strength by AISC Manual - force applied on bolts (kips)	186.8	114.6	48.1	126.6
Strength by IDEA StatiCa - force applied on welding (kips)	216.6	145.4	74.8	168.0
Strength by AISC 360-16 Eq. J2.4 - force applied on welding (kips)	228.3	161.5	94.7	201.9
Ultimate Shear Measured During Experiments (kips)	230	205	117	192

Table 1.13: Comparison of shear capacities of six additional models from AISC LRFD design equations and IDEA StatiCa analysis

Strength Capacities	Model 1	Model 2	Model 3	Model 4	Model 5	Model 6
Strength by IDEA StatiCa - force applied on bolts (kips)	127.3	200.1	129.1	130.2	132.3	127.9
Strength by AISC Manual - force applied on bolts (kips)	233.5	186.8	139.9	186.8	186.8	214.4
Strength by IDEA StatiCa - force applied on welding (kips)	229.0	226.7	136.0	216.5	213.3	234.1
Strength by AISC 360-16 Eq. J2.4 - force applied on welding (kips)	285.4	228.1	139.9	228.1	228.1	285.4

1.6.2 Comparison of IDEA StatiCa and ABAQUS Results

The comparison between the IDEA StatiCa and ABAQUS results were summarized in Tables 1.14–1.16 for case 1, and Tables 1.17–1.19 for case 2. In these tables:

$\boldsymbol{F_y}$**:** Yield strength

$\boldsymbol{\sigma_{Ed}}$ **:** Resultant Equivalent stress

$\boldsymbol{\varepsilon_{pl}}$**:** Plastic Strain

Check status Ok: pass the AISC requirements

$\boldsymbol{F_t}$**:** Tension force

$\boldsymbol{V}$**:** Resultant of shear forces in bolt

$\emptyset\boldsymbol{R_{n,bearing}}$**:** Bolt bearing resistance

$\boldsymbol{Ut_t}$**:** Utilization in tension

$\boldsymbol{Ut_s}$**:** Utilization in shear

$\boldsymbol{F_n}$**:** Force in weld critical element

$\emptyset\boldsymbol{R_n}$**:** Weld resistance

Table 1.14. Specified yield strengths and calculated stress, strain, and plates check status (case 1)

IDEA StatiCa					ABAQUS				
Item	F_y (ksi)	σ_{Ed} (ksi)	ε_{pl} (%)	**Check Status**	**Item**	F_y (ksi)	σ_{Ed} (ksi)	ε_{pl} (%)	**Check Status**
C-bfl 1	50	25.9	0	Ok	**Column**	50	37.75	0	Ok
C-tfl 1	50	4.7	0	Ok					
C-w 1	50	9.2	0	Ok					
B-bfl 1	50	36.0	0	Ok	**Beam**	50	45.00	0	Ok
B-tfl 1	50	36.0	0	Ok					
B-w 1	50	45.1	0.2	Ok					
CLEAT 1 a-bfl1	36	32.7	1.1	Ok	**Front double angle**	36	32.40	12.7	Not Ok!
CLEAT 1 a-w1	36	33.8	4.9	Ok					
CLEAT 1 b-bfl1	36	32.7	1.1	Ok	**Back double angle**	36	32.40	12.7	Not Ok!
CLEAT 1 b-w1	36	33.8	4.9	Ok					

Table 1.15. Calculated tension force, shear force, and bolt bearing resistance when the external load is applied on the bolt line (case 1)

	IDEA StatiCa					ABAQUS				
Item	F_t ($kips$)	V ($kips$)	$\emptyset R_{n,bearing}$ ($kips$)	Ut_t (%)	Ut_s (%)	F_t ($kips$)	V ($kips$)	$\emptyset R_{n,bearing}$ ($kips$)	Ut_t (%)	Ut_s (%)
B1	3.10	9.49	16.60	10.4	57.2	3.18	9.64	36.40	9.1	45.9
B2	4.24	9.51	36.28	14.2	53.2	2.47	9.64	36.40	7.1	45.9
B3	4.94	9.46	36.28	16.6	52.9	2.67	9.44	36.40	7.6	45.1
B4	5.63	9.40	36.28	18.9	52.6	5.70	9.53	36.40	16.3	45.4
B5	6.42	9.24	36.28	21.6	51.7	5.01	9.11	36.40	14.3	43.4
B6	7.17	9.10	36.28	24.1	50.9	8.67	8.91	36.40	24.8	42.4
B7	6.88	9.05	36.28	23.1	50.6	10.37	8.92	36.40	29.6	42.5

Table 1.16. Calculated force in weld critical element, weld resistance, and welds check status (case 1)

	IDEA StatiCa				ABAQUS			
Item	F_n ($kips$)	$\emptyset R_n$ ($kips$)	Ut (%)	***Status***	F_n ($kips$)	$\emptyset R_n$ ($kips$)	Ut (%)	***Status***
C-bfl 1	3.39	4.09	82.9	Ok	4.09	4.13	99.1	Ok
C-bfl 1	3.39	4.09	82.8	Ok	4.09	4.13	99.1	Ok

Table 1.17. Specified yield strengths and calculated stress, strain, and plates check status (case 2)

IDEA StatiCa					ABAQUS				
Item	F_y (ksi)	σ_{Ed} (ksi)	ε_{pl} (%)	**Check Status**	**Item**	F_y (ksi)	σ_{Ed} (ksi)	ε_{pl} (%)	**Check Status**
C-bfl 1	50	23.3	0	Ok	**Column**	50	19.03	0	Ok
C-tfl 1	50	4.7	0	Ok					
C-w 1	50	9.6	0	Ok					
B-bfl 1	50	45.0	0	Ok	**Beam**	50	45.00	3.6	Ok
B-tfl 1	50	45.0	0	Ok					
B- w 1	50	46.1	3.9	Ok					
CLEAT 1 a-bfl1	36	32.8	1.3	Ok	**Front double angle**	36	32.40	1.2	Ok
CLEAT 1 a-w1	36	32.4	0.2	Ok					
CLEAT 1 b-bfl1	36	32.8	1.3	Ok	**Back double angle**	36	32.40	1.2	Ok
CLEAT 1 b-w1	36	32.4	0.2	Ok					

Table 1.18. Calculated tension force, shear force, and bolt bearing resistance when the external load is applied on the weld line (case 2)

	IDEA StatiCa					ABAQUS				
Item	F_t ($kips$)	V ($kips$)	$\emptyset R_{n,bearing}$ ($kips$)	Ut_t (%)	Ut_s (%)	F_t ($kips$)	V ($kips$)	$\emptyset R_{n,bearing}$ ($kips$)	Ut_t (%)	Ut_s (%)
B1	7.31	15.92	36.28	24.5	89.1	7.32	15.76	36.40	24.7	88.1
B2	3.79	16.01	36.28	12.7	89.6	3.51	15.95	36.40	11.9	89.2
B3	2.80	15.67	36.28	9.4	87.6	2.83	15.16	36.40	10.1	84.8
B4	2.60	15.55	36.28	8.7	87	2.75	14.43	36.40	8.8	80.6
B5	2.49	15.64	36.28	8.4	87.5	2.33	14.66	36.40	7.8	82.1
B6	2.64	16.03	36.28	8.9	89.7	2.65	14.97	36.40	8.5	83.7
B7	4.31	16.81	36.28	14.5	94	5.63	16.56	36.40	17.9	92.6

Table 1.19. Calculated force in weld critical element, weld resistance, and welds check status (case 2)

	IDEA StatiCa				ABAQUS			
Item	F_n ($kips$)	$\emptyset R_n$ ($kips$)	Ut (%)	***Status***	F_n ($kips$)	$\emptyset R_n$ ($kips$)	Ut (%)	***Status***
C-bfl 1	3.87	3.87	100	Not OK!	4.1	4.12	99.5	Ok
C-bfl 1	3.87	3.87	100	Not OK!	4.1	4.12	99.5	Ok

In general, there was good agreement between the results of two software packages. In case 1, in which load was applied on the centroid of the bolt group, more deformation was observed on the double angles in the ABAQUS model. Also, the maximum predicted stress on the beam, column, and weld lines was slightly higher in the ABAQUS model. In addition, slightly different stress distribution was observed on the beam in the ABAQUS model. While applying the load on the bolt group is not common in traditional finite element software, such discrepancy could be associated with different contact formulations or element types (i.e., solid element in ABAQUS versus shell element in IDEA StatiCa). Also, due to the nature of the tie constraint, larger stresses were obtained on the column in the ABAQUS model. In case 2, in which load was applied on the weld lines, much better agreement was observed between the two models. In both models, it was found the weakest component of the connections was weld lines. This is also consistent with the LRFD code design checks (Section 1.6.1). Stress distributions for each case can be seen in Appendix D.

CHAPTER 2 SEMI-RIGID CONNECTIONS

2.1. Introduction

In this chapter, the design strength capacities of ten semi-rigid connection specimens were calculated following the requirements of the AISC 360 (2016) and AISC Construction Manual (2017). These specimens were selected from the experimental study performed by Azizinamini et al. (1985) in the Department of Civil Engineering at University of South Carolina. All specimens were analyzed using IDEA StatiCa while one of them was analyzed using ABAQUS (2020). Then, the results were compared.

2.2. Experimental Study on Semi-Rigid Connections

Several semi-rigid connections comprised of double angles and top and seat beam flanges were subjected to static and cyclic loadings to investigate their moment-rotation behavior. A pair of specimens was tested at the same time as shown in Figure 2.1. One side of the beam sections were bolted to the column and the other side was supported by roller-type seats. The vertical movement of the stub column was allowed by roller guides attached to the top and bottom of the column. The hydraulic actuator was used to apply the load on the column and the connection transferred the load to the beams.

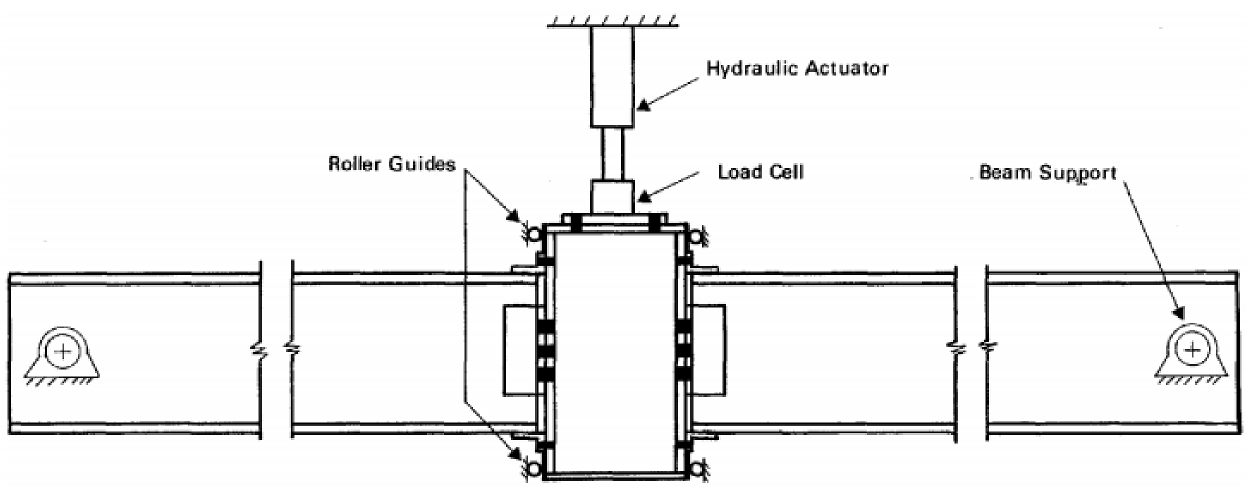


Figure 2.1: Test set-up used by Azizinamini et al. (1985)

The moment values were calculated from the actuator load cell readings. To obtain the corresponding rotations, the displacements measured by the linear variable differential transducers (LVDT) were converted to relative rotations between the end of the beam and the flange of the column (Figure 2.2).

In this study, ten specimens subjected to static loading were selected to be analyzed. The properties of these ten semi-rigid connection specimens are presented in the Table 2.1. All connections were bolted to beam and column. The first four beam specimens were framed to W14x38 beam section while W8x21 were used for the other six beam sections. The column section of W12x96 was used for all ten specimens. The members and connections were made of ASTM A36 steel while the fasteners were ASTM A325 with ¾ in. diameter bolts. The material properties of the members are provided in Table 2.2.

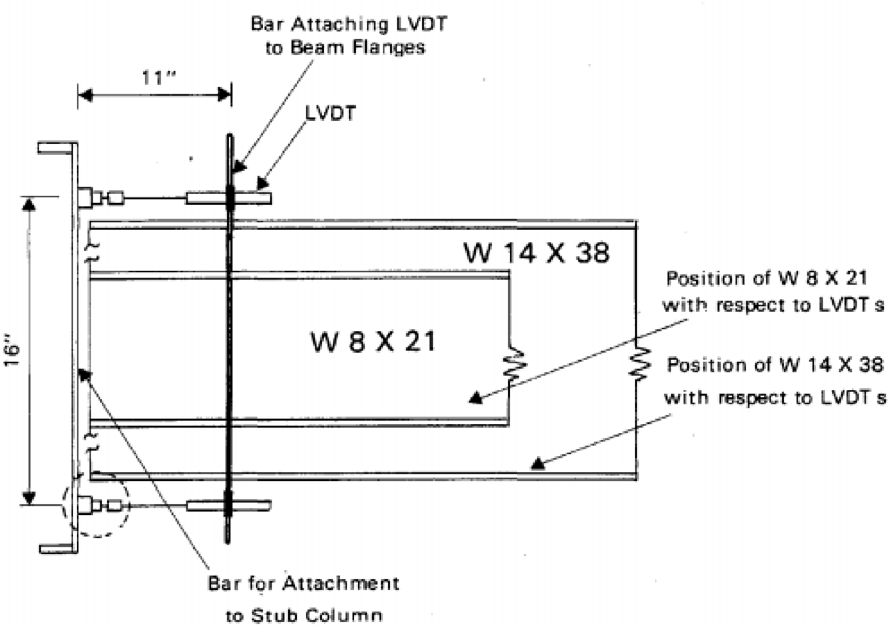


Figure 2.2: Measurements of horizontal displacements using the LVDT apparatus

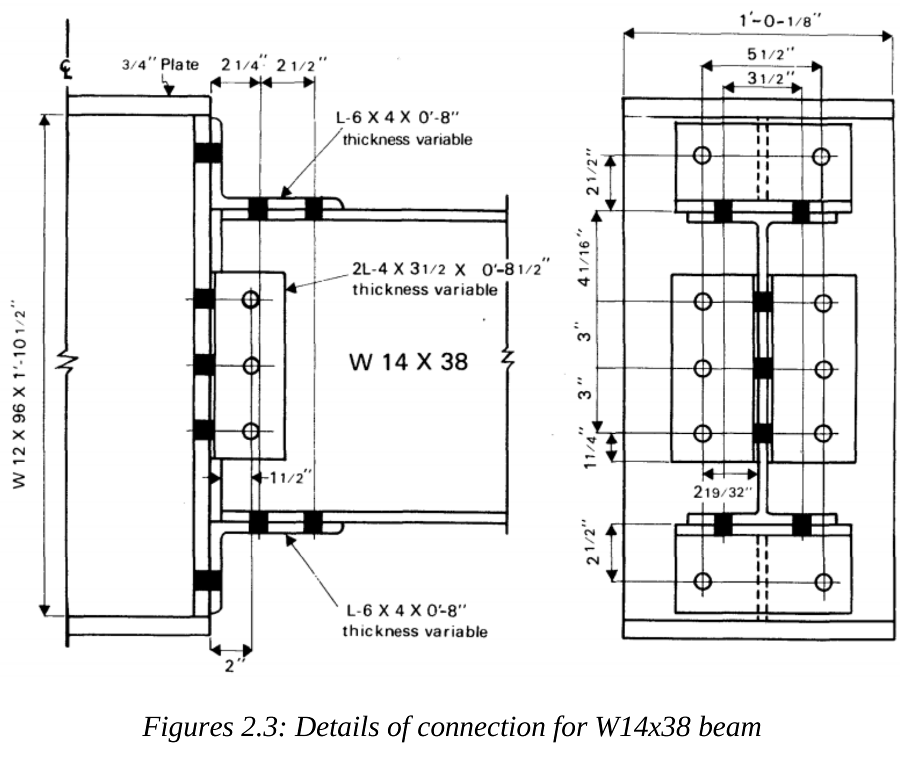
Table 2.1: Properties of semi-rigid connection specimens

Specimen Number	Beam Section	Top and Bottom Flange Angles				Web Angle		
		Angle	Length (in.)	Gage in leg on column Flange (in.)	Bolt Spacing in Leg on Column Flange (in.)	Angle	Length (in.)	Number of the bolt
14S1	W14x38	L6x4x3/8	8	2.5	5.5	2L4x3.5x1/4	8.5	3
14S2	W14x38	L6x4x1/2	8	2.5	5.5	2L4x3.5x1/4	8.5	3
14S3	W14x38	L6x4x3/8	8	2.5	5.5	2L4x3.5x1/4	5.5	2
14S4	W14x38	L6x4x3/8	8	2.5	5.5	2L4x3.5x3/8	8.5	3
8S1	W8x21	L6x3.5x5/16	6	2	3.5	2L4x3.5x1/4	5.5	2
8S2	W8x21	L6x3.5x3/8	6	2	3.5	2L4x3.5x1/4	5.5	2
8S3	W8x21	L6x3.5x5/16	8	2	3.5	2L4x3.5x1/4	5.5	2
8S4	W8x21	L6x6x3/8	6	4.5	3.5	2L4x3.5x1/4	5.5	2
8S5	W8x21	L6x4x3/8	8	2.5	5.5	2L4x3.5x1/4	5.5	2
8S6	W8x21	L6x4x5/16	6	2.5	3.5	2L4x3.5x1/4	5.5	2

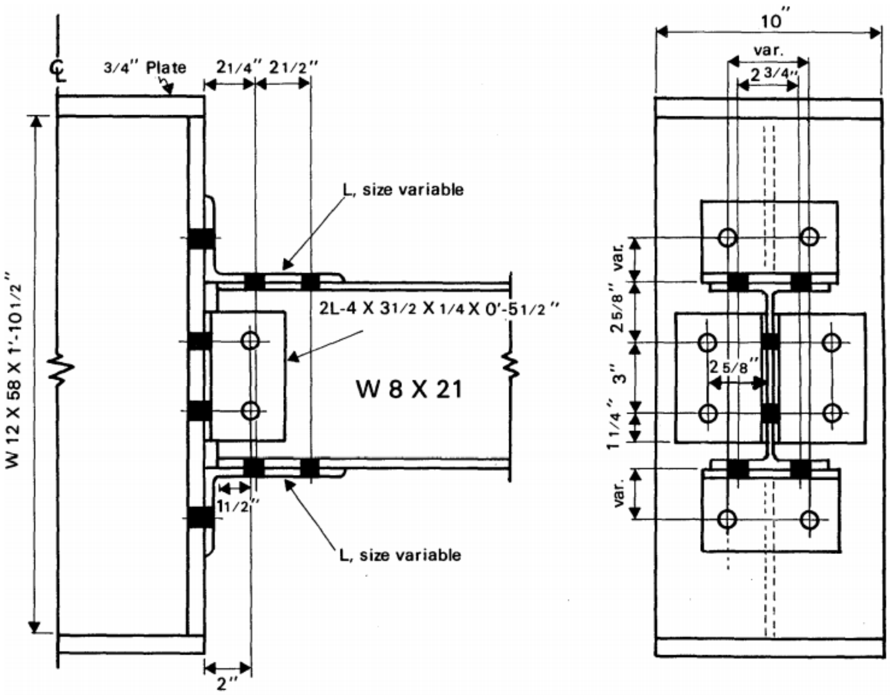
Table 2.2: Material properties of semi-rigid test specimens

Members	Strength	ksi
Columns, Beams and Angles (A36)	Yield Strength, F_y	36
	Ultimate Strength, F_u	58
Bolts A325 (threads excluded)	Nominal Tensile Strength, F_{nt}	90
	Nominal Shear Strength, F_{nv}	68

The bolt spacing was 3.0 in. while the edge distance of bolts was 1.25 inches from the top and bottom of double angle sections. The longitudinal bolt spacing and the edge distance of top and seat angles on beam side were 2.5 in. and 1.25 in., respectively while those varied on the angles attached to the column flange. Similarly, transfer bolt spacing, and edge distance of top and seat angles varied as provided in Table 2.1. The geometric details of the connections are shown in Figures 2.3 and 2.4. The summary of the test results measured during static loading are presented in Table 2.3 and Figures 2.5 through 2.9. The blue line shows the resistance determined by AISC traditional calculation (Chapter 2.3) and the orange line shows the resistance determined by IDEA StatiCa (Chapter 2.4).



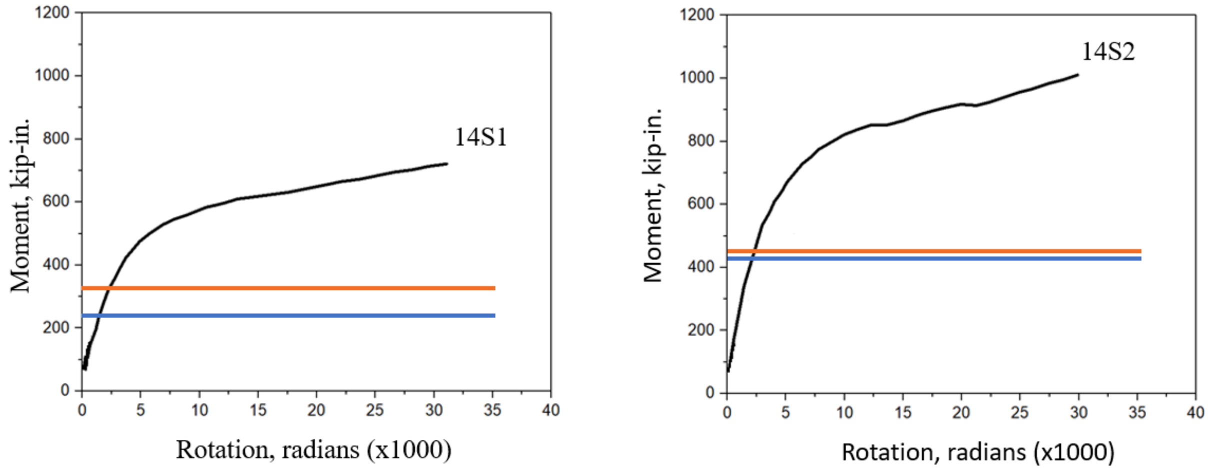
Figures 2.3: Details of connection for W14x38 beam



Figures 2.4: Details of connection for W8x21 beam

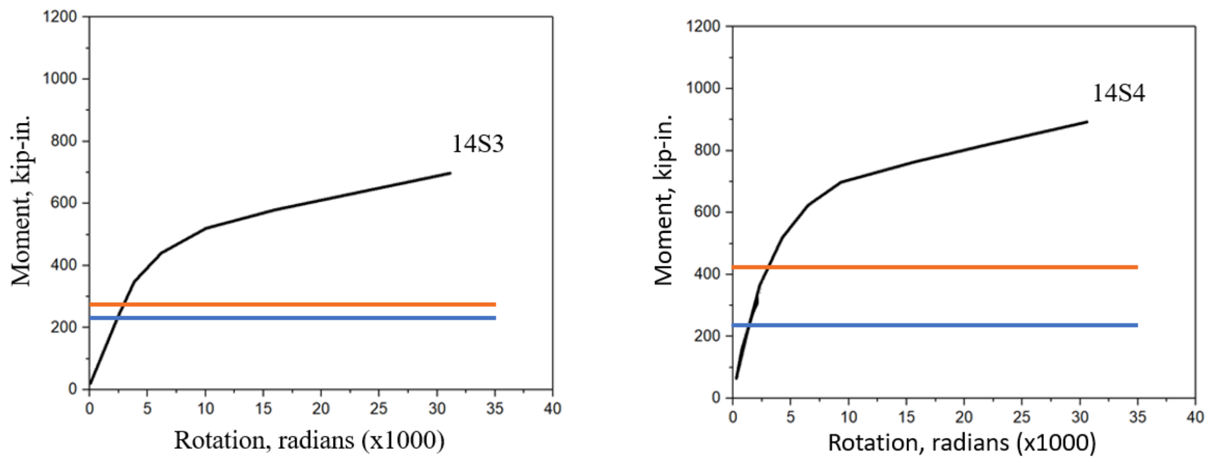
Table 2.3: Summary of test results

Specimen Number	Initial Slope of M-ϕ Curve (k-in./radian)	Slope of Secant Line to M-ϕ Curve at $4.0X10^{-3}$ radians (k-in./radian)	Moment at $4.0X10^{-3}$ radians (k-in.)	Slope of M-ϕ Curve at $24X10^{-3}$ radians (k-in./radian)	Moment at $24X10^{-3}$ radians (k-in.)	Remarks
14S1	$195.0X10^{+3}$	$108.7X20^{+3}$	435	$5.8X10^{+3}$	688	
14S2	295.0	151.8	607	12.6	(947)	Major slip at $12X10^{-3}$ & $20X10^{-3}$ radians
14S3	115.9	88.8	355	7.2	652	
14S4	221.9	124.0	496	8.3	822	
8S1	66.7	44.3	177	4.1	329	
8S2	123.4	69.0	276	1.5	(384)	Major slip at $16X10^{-3}$ radians
8S3	104.7	64.3	257	4.0	422	
8S4	15.3	14.4	57.5	2.2	165	
8C5	76.7	47.9	191.5	2.7	337	
8S6	39.5	30.0	120	3.2	244	
8S7	48.0	40.8	163	3.2	381	



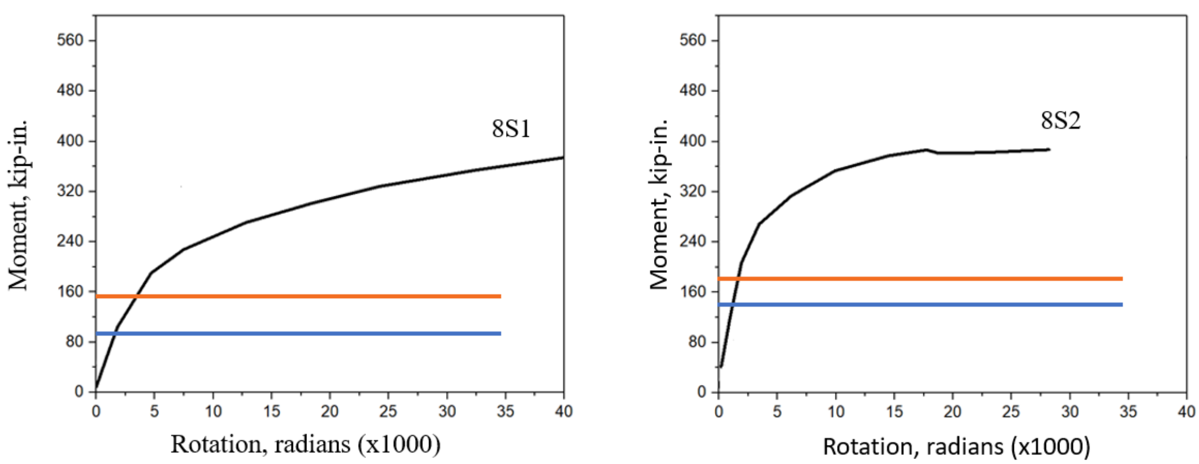
(a) (b)

Figure 2.5: Moment-rotation relationship of Test No: a) 14S1 and b) 14S2



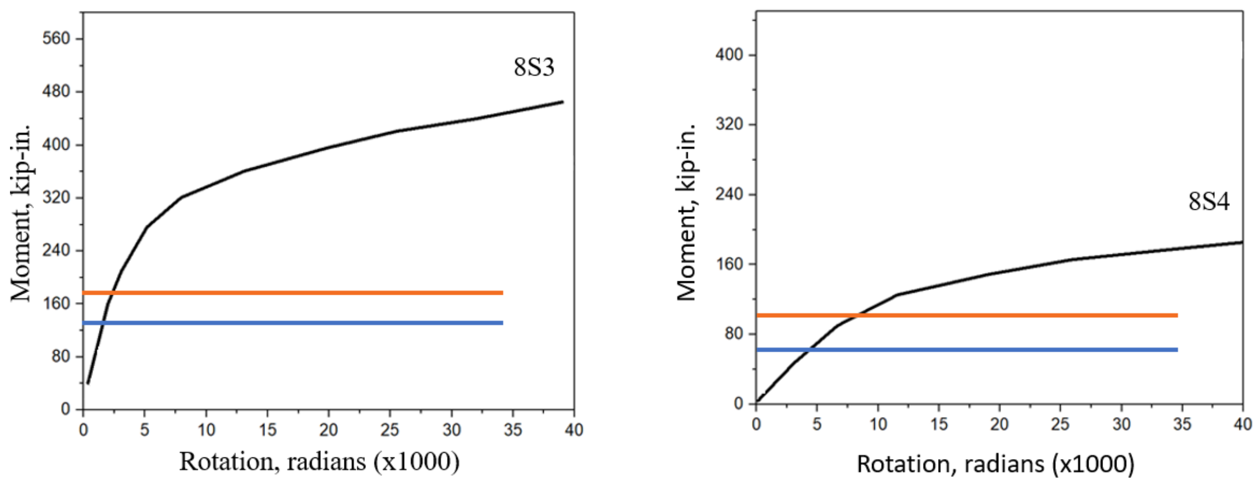
(a) (b)

Figure 2.6: Moment-rotation relationship of Test No: a) 14S3 and b) 14S4



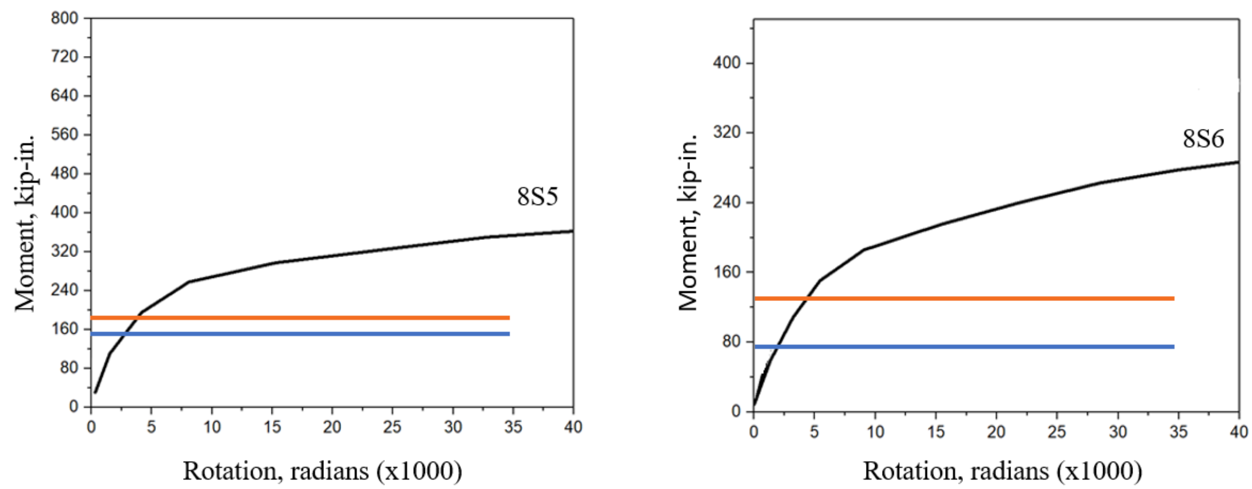
(a) (b)

Figure 2.7: Moment-rotation relationship of Test No: a) 8S1 and b) 8S2



(a) (b)

Figure 2.8: Moment-rotation relationship of Test No: a) 8S3, and b) 8S4



(a) (b)

Figure 2.9: Moment-rotation relationship of Test No: a) 8S5, and b) 8S6

2.3. Code Design Calculations and Comparisons

The design strength capacities (ϕR_n) of the connections were calculated following the requirements of AISC 360 (2016) and AISC Manual (2017). The nominal strength, R_n, and the corresponding resistance factor, ϕ for each connection design LRFD limit state are provided in Chapter J of AISC 360. It is assumed that the top and seat angles provide moment resistance, and the double web angle is used for shear resistance for the connection conservatively.

2.3.1 Design strength capacity of double web-angles

The following 14 design checks were performed according to the LRFD design equations included in AISC 360 or AISC Manual for the design strength capacity of double web-angle.

1. Angle (Beam side)
    a. Bolts shear — Eq. J3-1, AISC 360-16
    b. Bolt bearing and tearout — Eq. J3-6, AISC 360-16
    c. Shear yielding — Eq. J4-3, AISC 360-16
    d. Shear rupture — Eq. J4-4, AISC 360-16
    e. Block shear — Eq. J4-5, AISC 360-16
2. Angle (Column side)
    a. Bolts shear — Eq. J3-1, AISC 360-16
    b. Bolt bearing and tearout — Eq. J3-6, AISC 360-16
    c. Shear yielding — Eq. J4-3, AISC 360-16
    d. Shear rupture — Eq. J4-4, AISC 360-16
    e. Block shear — Eq. J4-5, AISC 360-16
    f. Resulting tension capacity due prying action — Part 9, AISC Manual
3. Beam
    a. Bolt bearing and tearout — Eq. J3-6, AISC 360-16
    b. Shear yielding — Eq. J4-3, AISC 360-16
4. Column
    a. Bolt bearing and tearout — Eq. J3-6, AISC 360-16

The design strength capacities (ϕR_n) of double web angles for the ten specimens calculated by following the AISC LRFD code requirements are provided in Table 2.4.

Table 2.4: LRFD design strength capacities (ϕR_n) of double web angles in ten specimens

Design Checks (LRFD)	**Design Strength Capacity (kips)**									
Specimen Number	**14S1**	**14S2**	**14S3**	**14S4**	**8S1**	**8S2**	**8S3**	**8S4**	**8S5**	**8S6**
Angles (Beam side)										
- Bolt Shear	135.24	135.24	89.58	134.4	89.58	89.58	89.58	89.58	89.58	89.58
- Bolt Bearing and Tearout	117.45	117.45	67.70	160.30	67.70	67.70	67.70	67.70	67.70	67.70
- Shear Yielding	91.80	91.80	64.80	145.80	64.80	64.80	64.80	64.80	64.80	64.80
- Shear Rupture	76.73	76.73	55.46	124.8	55.46	55.46	55.46	55.46	55.46	55.46
- Block Shear	*71.53*	*71.53*	59.56	125.8	59.56	59.56	59.56	59.56	59.56	59.56
Angles (Column side)										
- Bolt Shear	135.24	135.24	89.58	134.4	89.58	89.58	89.58	89.58	89.58	89.58
- Bolt Bearing and Tearout	117.45	117.45	61.17	150.5	61.17	61.17	61.17	61.17	61.17	61.17
- Shear Yielding	91.80	91.80	64.80	145.80	64.80	64.80	64.80	64.80	64.80	64.80
- Shear Rupture	76.73	76.73	55.46	124.8	55.46	55.46	55.46	55.46	55.46	55.46
- Block Shear	71.53	71.53	52.10	114.6	52.10	52.10	52.10	52.10	52.10	52.10
Beam										
- Bolt Bearing and Tearout	72.81	72.81	*48.55*	*72.82*	*39.15*	*39.15*	*39.15*	*39.15*	*39.15*	*39.15*
- Shear Yielding	94.41	94.41	94.41	94.41	44.71	44.71	44.71	44.71	44.71	44.71
Column										
- Bolt Bearing	211.41	211.41	281.90	422.80	281.90	281.90	281.90	281.90	281.9	281.9

Out of the calculated design capacities for the ten test specimens, the lowest shear capacities are shown in bold and italic in Table 2.4. According to the results, the design capacity of two double web-angles (in specimens 14S1 and 14S2) were controlled by block shear of the bolts on the angle attached to the beam while bearing and tearout of the bolts on the beam controlled the shear design capacities of the other eight specimens. The detailed design strength calculations for Test 14S1 are provided in Appendix E.

2.3.2 Design strength capacity of the top and bottom seat-angles

The following 16 design checks were performed according to the LRFD equations included in AISC 360 or AISC Manual for the design strength capacity of the top- and seat-angle.

1. Top- and Seat-Angle (Beam Side)
    a. Tension yielding — Eq. J4-1, AISC 360-16
    b. Tension rupture — Eq. J4-2, AISC 360-16
    c. Compression — Sec. J4.4, AISC 360-16
    d. Bolts shear — Eq. J3-1, AISC 360-16

e. Bolt bearing and tearout — Eq. J3-6, AISC 360-16
f. Block shear — Eq. J4-5, AISC 360-16

2. Top- and Seat-Angle (Column Side)
   a. Shear yielding — Eq. J4-3, AISC 360-16
   b. Shear rupture — Eq. J4-4, AISC 360-16
   c. Tension capacity due prying action — Page 9-10, AISC Manual
3. Beam
   a. Bolt bearing and tearout — Eq. J3-6, AISC 360-16
   b. Flexural Strength — Sec. F13.1, AISC 360-16
   c. Block shear — Eq. J4-5, AISC 360-16
4. Column
   a. Panel web shear — Eq. J10-9, AISC 360-16
   b. Flange local bending — Eq. J10-1, AISC 360-16
   c. Web local yielding — Eq. J10-2, AISC 360-16
   d. Web local crippling — Eq. J10-4, AISC 360-16

The design strength capacities (ϕR_n) of top and bottom seat angles in ten specimens are calculated and provided in Table 2.5. The values highlighted in bold and italic in Table 2.5 show the lowest shear capacities in the ten test specimens considered. According to the results, the design capacities of all top and seat angles were controlled by tension capacity due prying action on the angle side bolted to the column. The design capacity all top and seat angles were controlled by tension capacity due prying action.

Table 2.5: The design strength capacities (ϕR_n) of top and seat-angle for ten specimens

Design Checks (LRFD)	**Design Strength Capacity (kips)**									
Specimen Number	**14S1**	**14S2**	**14S3**	**14S4**	**8S1**	**8S2**	**8S3**	**8S4**	**8S5**	**8S6**
Top and Seat Angles (Beam Side)										
- Tension Yielding	97.20	129.60	97.20	97.20	60.75	72.90	81.00	72.90	97.20	60.75
- Tension Rupture	101.78	135.94	101.78	101.78	57.77	69.33	84.96	69.33	101.78	57.77
- Compression	93.13	129.60	93.13	93.13	57.12	69.84	76.16	69.84	93.13	57.12
- Bolts Shear	90.16	89.58	90.16	90.16	90.16	90.16	90.16	90.16	90.16	90.16
- Bolt Bearing and Tearout	99.03	122.34	99.03	99.03	76.46	79.52	76.46	99.03	99.03	99.03
- Block Shear	67.25	117.84	67.25	67.25	63.46	64.43	63.46	76.15	76.15	63.46
Top and Seat Angles (Column Side)										
- Shear Yielding	64.80	86.40	64.80	64.80	40.50	48.60	54.00	48.60	64.80	40.50
- Shear Rupture	61.07	81.56	61.07	61.07	34.66	41.60	50.98	41.60	61.07	34.66
- Tension Capacity Due Prying Action	***13.73***	***25.01***	***13.73***	***13.73***	***9.00***	***13.25***	***12.47***	***4.84***	***13.62***	***6.72***
Beam										
- Bolt Bearing and Tearout	120.97	120.97	120.97	120.97	118.76	112.23	118.76	118.76	118.76	118.76
- Flexural Strength	166.05	166.05	166.05	166.05	55.08	55.08	55.08	55.08	55.08	55.08
- Block Shear	124.46	124.57	124.46	124.46	83.70	83.70	83.70	83.70	83.70	83.70
Column										
- Panel Web Shear	135.79	135.79	135.79	135.79	85.38	85.38	85.38	85.38	85.38	85.38
- Flange Local Bending	164.03	164.03	164.03	164.03	82.94	82.94	82.94	82.94	82.94	82.94
- Web Local Yielding	163.35	168.30	163.35	163.35	88.45	88.45	88.45	88.45	88.45	88.45
- Web Local Crippling	257.31	264.00	257.31	257.31	112.81	112.81	112.81	112.81	112.81	112.81

The moment capacities of the specimens were calculated by multiplying by the tension capacity (which is identical at the top and bottom angles) by moment arm which is set equal to the distance from the center of compression to bolt-row in tension (gage in leg on column flange + beam depth + a half of the thickness of seated-angle) as provided in Table 2.6. This definition may provide slightly larger moment arm because the compressive force is likely to be applied above the bolt-row.

Table 2.6: Design moment calculations for the ten semi-rigid connection specimens

Specimen Number	**14S1**	**14S2**	**14S3**	**14S4**	**8S1**	**8S2**	**8S3**	**8S4**	**8S5**	**8S6**
Tension Capacity due Prying Action (kips)	13.73	25.01	13.73	13.73	9.00	13.25	12.47	4.84	13.62	6.72
Beam Depth, *d* (in.)	14.10	14.10	14.10	14.10	8.28	8.28	8.28	8.28	8.28	8.28
Gage in leg on column flange, *g* (in.)	2.5	2.5	2.5	2.5	2.0	2.0	2.0	4.5	2.5	2.5
Flange angle thickness, *t* (in.)	3/8	1/2	3/8	3/8	5/16	3/8	5/16	3/8	3/8	5/16
Moment arm, z ($d + g + t/2$) (in.)	16.79	16.85	16.79	16.79	10.44	10.47	10.44	12.97	10.97	10.94
Moment Capacity, $M = T \cdot d$ (kips-in.)	230.49	421.42	230.49	230.49	93.93	138.69	130.14	62.76	149.38	73.49

2.3.3 ASD Design Strength Capacities of Test No. 14S1

According to allowable strength design (ASD), the allowable strength (R_n/Ω) is calculated by dividing the nominal strength, R_n by the safety factor, Ω. The strength of connection specimen Test No. 14S1 is calculated by following the AISC ASD code requirements. The properties of this test specimen were given in Table 2.1. The calculated ASD strength capacities (R_n/Ω) of the specimen including double web angle, and top and seat angles are provided in Tables 2.7 and 2.8, respectively. The calculated lowest strength of the double web angles in Test No. 14S1 is 48.54 kips due to the bolt bearing and tearout on the beam. The detailed design strength calculations for Test No. 14S1 are provided in Appendix F.

Table 2.7: ASD strength capacity (ϕR_n) of double web angle for Test No. 14S1

Design Checks (ASD)	**Design Strength Capacity (kips)**
Angle (Beam side)	
- Bolt Shear	90.18
- Bolt Bearing and Tearout	78.30
- Shear Yielding	61.20
- Shear Rupture	51.16
- Block Shear	55.87
Angle (Column side)	
- Bolt Shear	90.18
- Bolt Bearing and Tearout	78.30
- Shear Yielding	61.20
- Shear Rupture	51.16
- Block Shear	55.87
Beam	
- Bolt Bearing and Tearout	***48.54***
- Shear Yielding	62.93
Column	
- Bolt Bearing	140.94

Table 2.8: Design strength capacity (ϕR_n) of top and seat angles in specimen Test No. 14S1

Design Checks (ASD)	Design Strength Capacity (kips)
Top and Seat Angle (Beam Side)	
- Tension Yielding	64.67
- Tension Rupture	67.85
- Compression	61.96
- Bolts Shear	60.12
- Bolt bearing and tearout	78.30
- Block Shear	48.04
Top- and Seat-Angle (Column Side)	
- Shear Yielding	43.20
- Shear Rupture	40.72
- Tension Capacity Due Prying Action	***9.15***
Beam	
- Bolt Bearing and Tearout	80.66
- Flexural Strength	110.48
- Block Shear	82.98
Column	
- Web Panel Zone Shear	90.35
- Flange local Bending	109.13
- Web Local Yielding	108.90
- Web Local Crippling	171.54

The calculated lowest strength of top and seat angle for Test No. 14S1 is 9.15 kips because of the tension capacity due prying action on the angle bolted to column flange. The moment capacity of the connection (153.63 kips-in.) can be calculated by multiplying by the tension capacity of the angle (9.15 kips) by the moment arm (16.79 in.).

2.4. IDEA StatiCa Analysis

2.4.1 Moment capacity analysis using IDEA StatiCa

The ten test specimens were modeled in IDEA StatiCa with and without web angles, and analyzed under a shear force applied a certain distance away from the column. The distance was selected to be equal to the one between the column centerline and the beam support. The beam support is assumed to be at 120 in. away from the column centerline for the first four specimens while it was 72 in. for the other six specimens (this beam support is at the right side of the beam while the left side of the beam is bolted to the column as shown in Figure 2.1). The shear force was applied incrementally until the connection models reached their capacities in IDEA StatiCa. All specimens fail because of that the top angles attached to the column exceed the plastic strain limit which is defined as 5% by the software. The calculated moment capacities of the connection specimens having and not web angles are shown in Tables 2.9 and 2.10, respectively.

Table 2.9: Moment capacities of the specimens which have web angles calculated using IDEA StatiCa

Specimen Number	Shear force (kips)	Distance (in.)	Moment (kips-in.)
14S1	2.66	120	319.20
14S2	3.75	120	450.00
14S3	2.33	120	279.60
14S4	3.52	120	422.40
8S1	2.13	72	153.36
8S2	2.65	72	190.80
8S3	2.42	72	174.24
8S4	1.54	72	110.88
8S5	2.55	72	183.60
8S6	1.79	72	128.88

Table 2.10: Moment capacities of the specimens which don't have web angles calculated using IDEA StatiCa

Specimen Number	Shear force (kips)	Distance (in.)	Moment (kips-in.)
14S1	1.77	120	212.40
14S2	2.88	120	345.60
14S3	1.76	120	211.20
14S4	1.76	120	211.20
8S1	1.5	72	108.00
8S2	2.03	72	146.16
8S3	1.79	72	128.88
8S4	0.91	72	65.52
8S5	1.92	72	138.24
8S6	1.16	72	83.52

The screenshots from IDEA StatiCa showing the failure modes, and deformed shapes (deformation scale 10) of finite element models are shown in Appenix G.

2.4.2 Moment-rotation analysis

Moment-rotation analysis for Test No. 14S1 was performed using IDEA StatiCa. To generate the test condition, the mechanical properties of A36 steel provided in the test report were used (see Reference 6). The mean values of the yielding and ultimate strength of the material were reported in Azizinamini et al. (1985) as 40.65 ksi and 68.43 ksi, respectively. These are the properties used for the materials used in IDEA StatiCa models. The resistance factors were set to be equal 1.0 and moment-rotation analysis was performed by selecting stiffness analysis option (Figure 2.10).

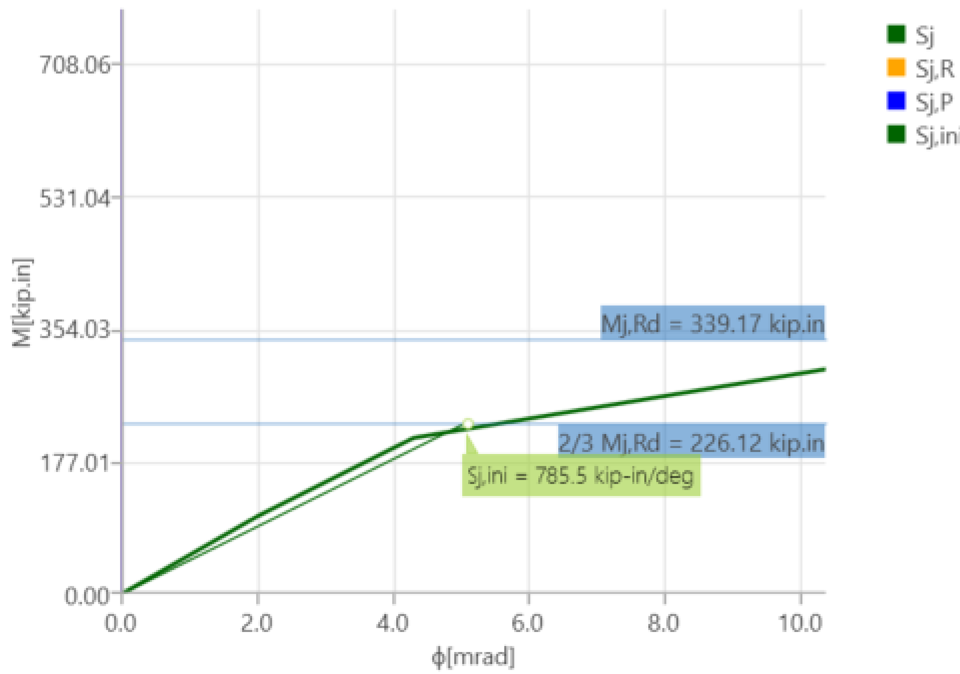


Figure 2.10: Moment-rotation relationship for Test No. 14S1 computed by IDEA StatiCa

2.5. ABAQUS Analysis

In this section, the output results from IDEA StatiCa were compared to those from ABAQUS (2020) software package. In this study, Test No. 14S1 specimen, as described in Table 2.1, was chosen as a base model. Numerical simulations with almost identical conditions (i.e., in terms of material properties, boundary conditions, and loading) were carried out using both IDEA StatiCa and ABAQUS. The model was initially designed in IDEA StatiCa and then the assembly (including beam, column, web angles, and top and seat angles) was imported to ABAQUS using the IDEA StatiCa's viewer platform. Afterward, a simplified model for the bolt was designed and added to the ABAQUS model (see Figure 2.11).

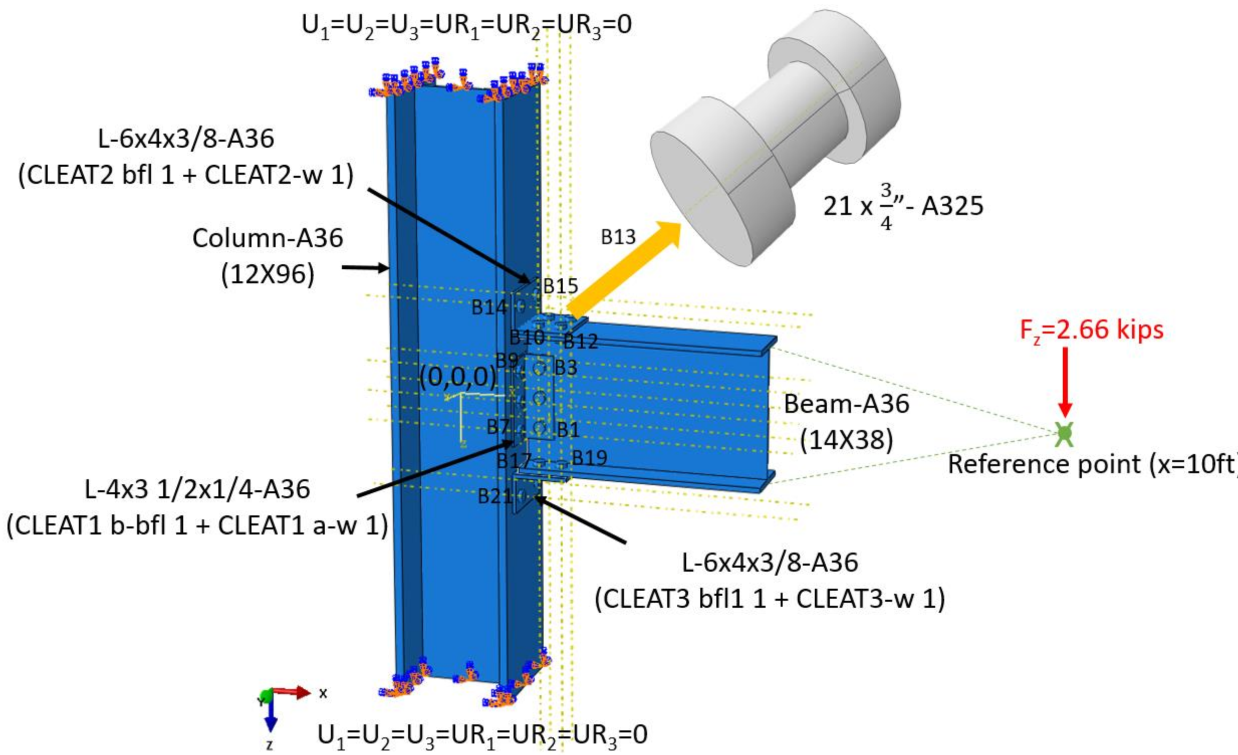


Figure 2.11: Semi-rigid connection model setup in ABAQUS

In ABAQUS, the element type was C3D8R (3D stress, 8-node linear brick, reduced integration), and a total of 562,377 elements were generated in the model. More details are provided in Table 2.11 and Figure 2.12.

Table 2.11. Number of elements in the ABAQUS model

Item	Number of Elements
Column	69,167
Beam	167,574
L-6x4x3/8	60,550
L-4x31/2x1/4	33,619
Bolt	6,538

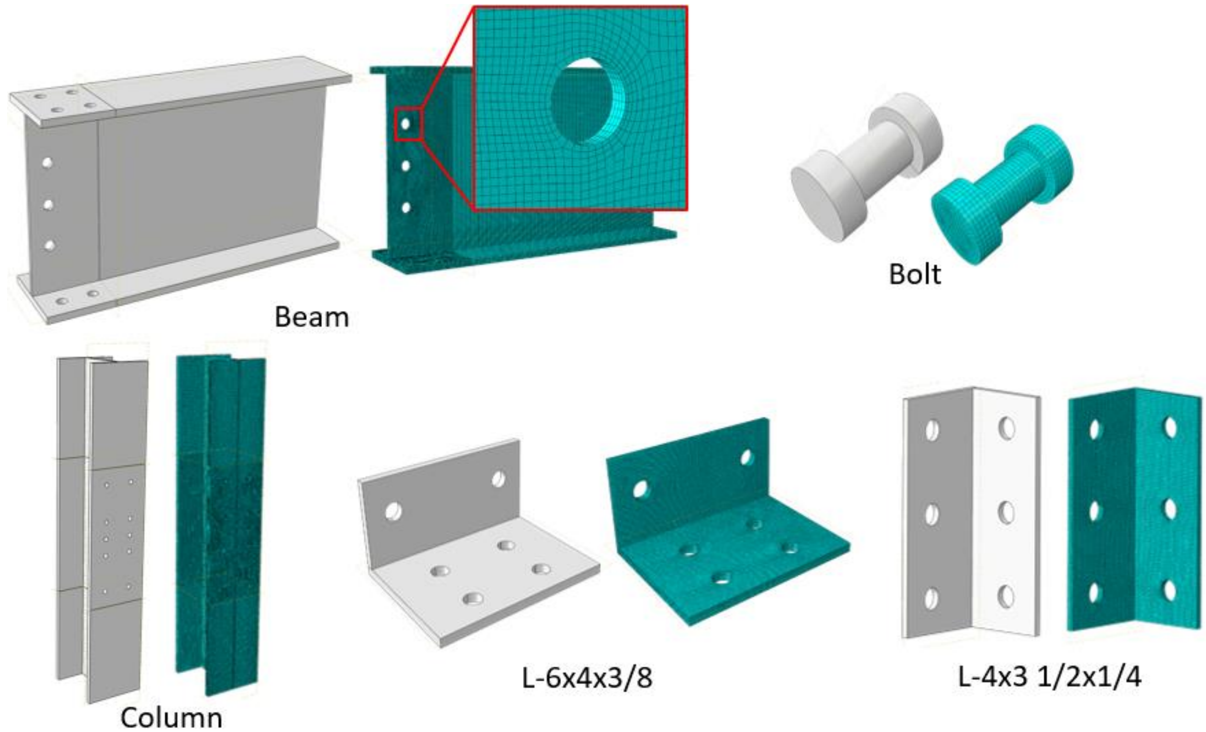


Figure 2.12: ABAQUS model mesh densities

In the ABAQUS model, the vertical force of 2.66 kips was applied on a reference point (or node) that was defined 10 ft away from the centerline of the column (i.e., x = 10 ft). Then, the coupling constraint (i.e., structural distributing) was defined to connect this reference point to the end section of the beam. The top and bottom of the column were fixed as a boundary condition (see Figure 2.11). The contact between all parts including column to all angles was defined as surface-to-surface with finite sliding formulation. Friction was defined with a penalty method, and a Coulomb friction coefficient of $\mu = 0.3$ was used everywhere except between the column face and each angle, in which the contact was assumed to be frictionless.

The material behavior was modeled using a bi-linear plasticity in ABAQUS. Other parameters including density, elastic modulus, and Poisson's ratio were exactly taken from the IDEA StatiCa materials library. The numerical simulations were carried out on four processors (Intel Xenon (R) CPU E5-2698 v4 @ 2.20GHz) and each simulation took approximately 535 minutes to finish. Figure 2.13 compares the calculated von-Mises stresses in IDEA StatiCa and ABAQUS. Figure 2.14 also shows the side view in which the deformation scale factor of ten was applied to models in both sotfware.

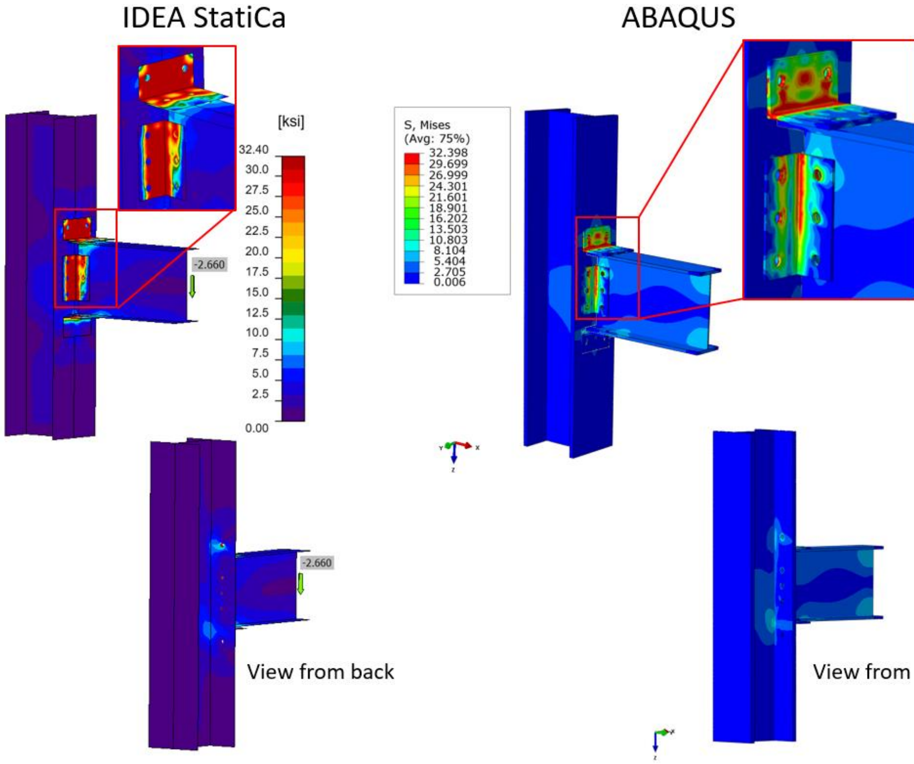


Figure 2.13: Comparison of the predicted von-Mises stress between IDEA StatiCa and ABAQUS

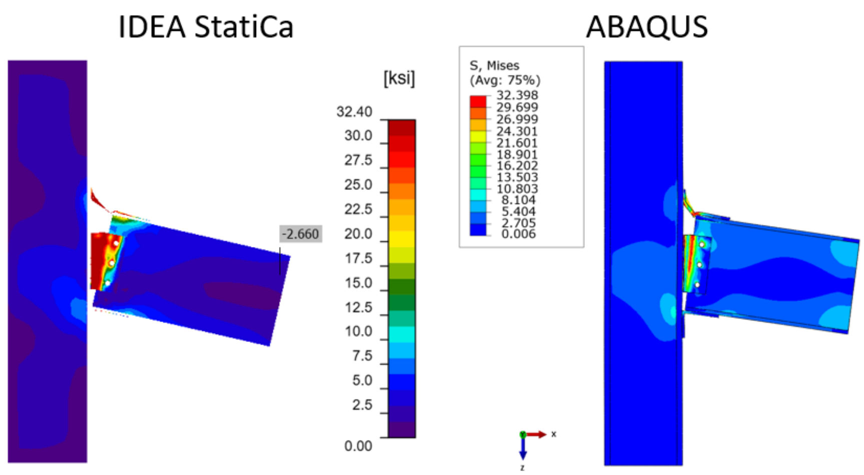


Figure 2.14: Side view comparison between IDEA StatiCa and ABAQUS with deformation scale factor of ten

2.6 Summary and Comparison of Results

2.6.1 Comparison of connection capacities from IDEA StatiCa analysis, AISC design codes, and Experiments

The design strength capacities (ϕR_n) of the ten semi-rigid connections were calculated using AISC 360 and AISC Manual (2017). The moment capacities of the specimens were calculated using a conservative approach assuming moment is carried by top and seat angles while web angles resist shear force only. The smallest calculated strengths were determined, and the moment capacities of the connections were obtained corresponding to these controlling strengths.

First, the same specimens were modeled in IDEA StatiCa and analyzed under a shear force applied 120 in. away from the column centerline for the first four specimens (14S1, 14S2, 14S3, 14S4); and 72 in. away for the other six specimens (8S1, 8S2, 8S3, 8S4, 8S5, 8S6). The shear force was increased incrementally until the connections reached their capacities. The moment capacities of the connections were obtained by multiplying by the distance between the shear force application point and column centerline, and the ultimate shear force reached in the incremental loading. In the second part, the web angles were removed out of the specimens, and the moment capacities of top and seated connections were obtained by following the same procedure to eliminate the resistance of web angles on the moment capacities of the specimens in IDEA analysis. The results were compared in Table 2.12.

Table 2.12: Comparison of the moment capacities of the specimens calculated from AISC design equations and IDEA StatiCa analysis

Specimen Number	**AISC LRFD Design Strength Moment Capacities (kips-in.)**	**IDEA StatiCa Analysis of the specimens which have web angles, Moment Capacities (kips-in.)**	**IDEA StatiCa Analysis of the specimens which don't have web angles, Moment Capacities (kips-in.)**
14S1	230.49	319.20	212.40
14S2	421.42	450.00	345.60
14S3	230.49	279.60	211.20
14S4	230.49	422.40	211.20
8S1	93.93	151.92	108.00
8S2	138.69	190.80	146.16
8S3	130.14	174.24	128.88
8S4	62.76	110.88	65.52
8S5	149.38	184.32	138.24
8S6	73.49	128.88	83.52

The moment-rotation relationship of the Test No. 14S1 was calculated from IDEA StatiCa analysis using the mean measured material properties (the mean values of the material strengths (F_u, F_y) tested in the lab and provided in the test report) of the tested specimens measured in the experimental study. The calculated response is compared with the moment-rotation relationship provided in the test report (Figure 2.15).

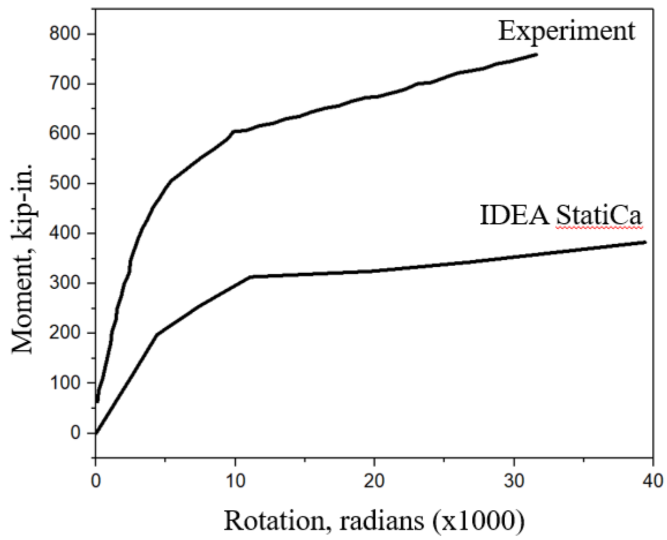


Figure 2.16: Comparison of the moment-rotation relationships of Test No. 14S1 measured during the experiment and calculated from IDEA StatiCa

2.6.2 Comparison of IDEA StatiCa and ABAQUS Results

The comparison between the IDEA StatiCa and ABAQUS results were summarized in Tables 2.13 and 2.14. In general, there was good agreement between the results of two software packages. However, more deformation was captured on the web angeles, top, and bottom flanges in the IDEA StatiCa model. Also, the stress distributions on the web angles were slightly different between the two models. This is most likely due to the fact that in the ABAQUS model solid elements with reduced integration were utilized. In both models, it was found that the weakest component of the assembly was the top flange in tension under the applied shear force pointing downward, which introduces tension in the top flange. Stress distribution for each part can be seen in Appendix H.

Table 2.13. Specified yield strengths and calculated stress, strain, and plates check status

IDEA StatiCa					ABAQUS				
Item	F_y (ksi)	σ_{Ed} (ksi)	ε_{pl} (%)	**Check Status**	**Item**	F_y (ksi)	σ_{Ed} (ksi)	ε_{pl} (%)	**Check Status**
C-bfl 1	36	13.7	0	Ok	**Column**	36	15.624	0	Ok
C-tfl 1	36	1.2	0	Ok					
C-w 1	36	8.6	0	Ok					
B-bfl 1	36	17.2	0	Ok	**Beam**	36	32.398	0	Ok
B-tfl 1	36	32.4	0	Ok					
B- w 1	36	29.3	0	Ok					
CLEAT 1 a-bfl1	36	33.3	3.1	Ok	**L-4 x 3 1/2 x 1/4 (Front)**	36	32.398	0.8	Ok
CLEAT 1 a-w1	36	33	2.1	Ok					
CLEAT 1 b-bfl1	36	33.2	2.6	Ok	**L-4 x 3 1/2 x 1/4 (Back)**	36	32.398	0.8	Ok
CLEAT 1 b-w1	36	33	2.1	Ok					
CLEAT 2 -bfl1	36	33.9	5	Ok	**L-6 x 4 x 3/8 (Top)**	36	32.398	1.8	Ok
CLEAT 2 -w1	36	33.8	4.7	Ok					
CLEAT 3 -bfl1	36	26	0	Ok	**L-6 x 4 x 3/8 (Bottom)**	36	32.398	0.1	Ok
CLEAT 3 -w1	36	32.5	0.3	Ok					

Table 2.14. Calculated Tension Force, Shear Force, and Bolt Bearing Resistance

	IDEA StatiCa					**ABAQUS**				
Item	F_t $(kips)$	V $(kips)$	$\emptyset R_{n,bearing}$ $(kips)$	Ut_t (%)	Ut_s (%)	F_t $(kips)$	V $(kips)$	$\emptyset R_{n,bearing}$ $(kips)$	Ut_t (%)	Ut_s (%)
B1	2.28	1.083	24.359	7.7	8.9	2.169	1.025	26.364	7.2	7.8
B2	3.777	2.250	21.318	12.7	21.1	3.401	2.035	23.146	11.4	17.6
B3	5.633	3.905	18.959	18.9	41.2	5.736	3.621	20.486	19.2	35.4
B4	8.397	2.179	16.596	28.2	13.1	7.536	2.023	17.895	25.2	11.3
B5	14.791	1.841	17.469	49.6	10.5	14.483	1.705	18.869	48.6	9.0
B6	19.713	1.486	19.580	66.2	7.6	18.751	1.526	21.132	62.9	7.2
B7	8.411	2.179	16.585	28.2	13.1	7.555	2.022	17.879	25.3	13.3
B8	14.821	1.842	17.456	49.7	10.5	14.491	1.705	18.859	48.6	9.0
B9	19.731	1.490	19.580	66.2	7.6	18.751	1.526	21.132	62.9	7.2
B10	12.306	3.391	29.293	41.3	15.0	11.89	3.556	31.615	39.9	15.8
B11	12.276	3.390	29.293	41.2	15.0	11.89	3.556	31.615	39.9	15.8
B12	0.233	3.069	16.285	0.8	18.8	0.450	3.456	17.575	1.5	19.6
B13	0.233	3.069	16.285	0.8	18.8	0.450	3.456	17.575	1.5	19.6
B14	23.861	3.134	22.136	80.1	14.2	23.259	3.222	23.752	78.1	13.5
B15	23.868	3.137	22.138	80.1	14.2	23.259	3.222	23.752	78.1	13.5
B16	2.476	6.092	29.370	8.3	27	2.569	5.957	31.559	8.6	26.4
B17	2.475	6.092	29.370	8.3	27	2.569	5.957	31.559	8.6	26.4
B18	0.423	6.318	29.370	1.4	28	0.445	6.152	31.559	1.5	27.2
B19	0.424	6.318	29.370	1.4	28	0.445	6.152	31.559	1.5	27.2
B20	0.358	1.575	29.370	1.2	7	0.258	1.456	31.559	0.9	6.4
B21	0.364	1.579	29.370	1.2	7	0.258	1.456	31.559	0.9	6.4

CHAPTER 3 RIGID CONNECTIONS

3.1. Introduction

In this chapter, the design strength capacities of ten rigid connection specimens were calculated following the requirements of the AISC 360 (2016) and AISC Construction Manual (2017). The baseline specimen was selected from the experimental study performed by Sato et al. (2007) in the Department of Structural Engineering at University of California, San Diego. The baseline specimen and nine additional variation models were analyzed using IDEA StatiCa while the baseline specimen was also analyzed using ABAQUS (2020). The results were then compared at the end of the chapter.

3.2. Experimental Study on Rigid Connections

Three full-scale bolted flange plate (BFP) moment connections were subjected to cycling testing at the University of California, San Diego. All specimens met the requirement of AISC Seismic Provisions for Structural Steel Buildings for the beam-column connections of special moment frames. The lateral bracing distance for the specimens was determined in accordance with this provision. The vertical displacements were applied by a hydraulic actuator at the tip of the beam as shown in Figure 3.1.

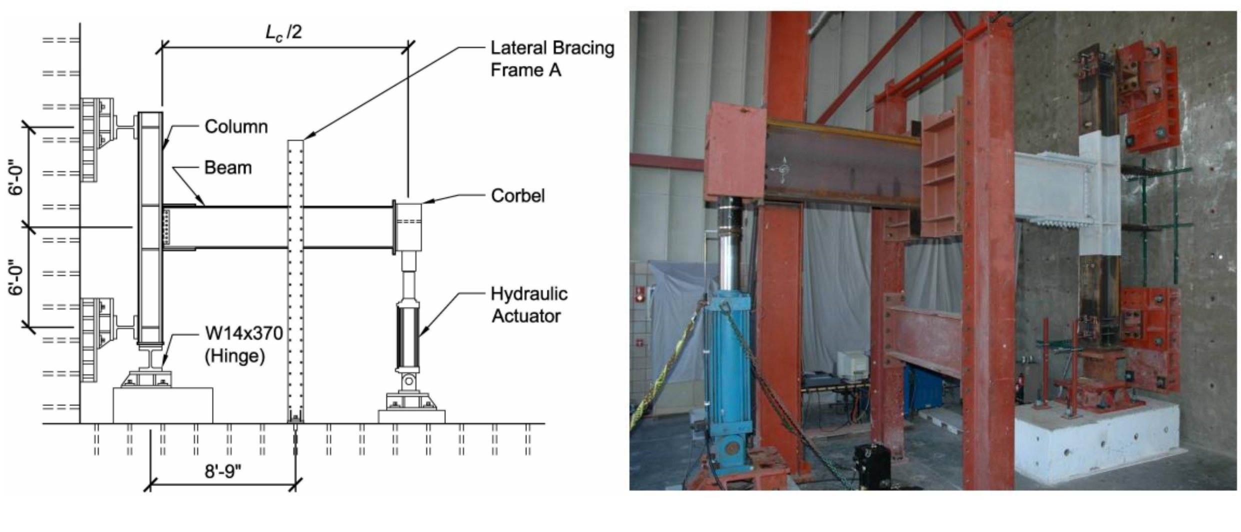


Figure 3.1: Test Setup: (a) schematic; (b) photo (Sato et al., 2007)

The loading began at 0.375% drift and the displacement magnitude was increased until the specimen failed. the applied load was measured by the load cell mounted on the actuator. The transducer L1 in Figure 3.2 measured the total displacement of the beam tip while the column horizontal movement was recorded by L5 and L6. The average shear deformation of the column panel zone was measured by L9 and L10 (Figure 3.2). The moment-rotation relationships at column face were obtained for all specimens using the data measured by these instruments.

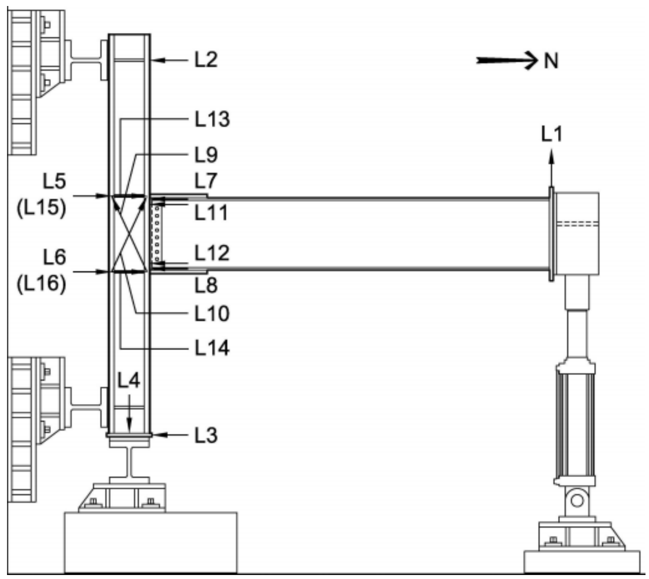


Figure 3.2: Displacement transducer locations (Sato et al., 2007)

In this study, the Specimen No. BFP was selected as a baseline model. For this specimen, the loading was applied at approximately 177.5 in. away from the column face. The details of this connection are shown in Figures 3.3 and 3.4.

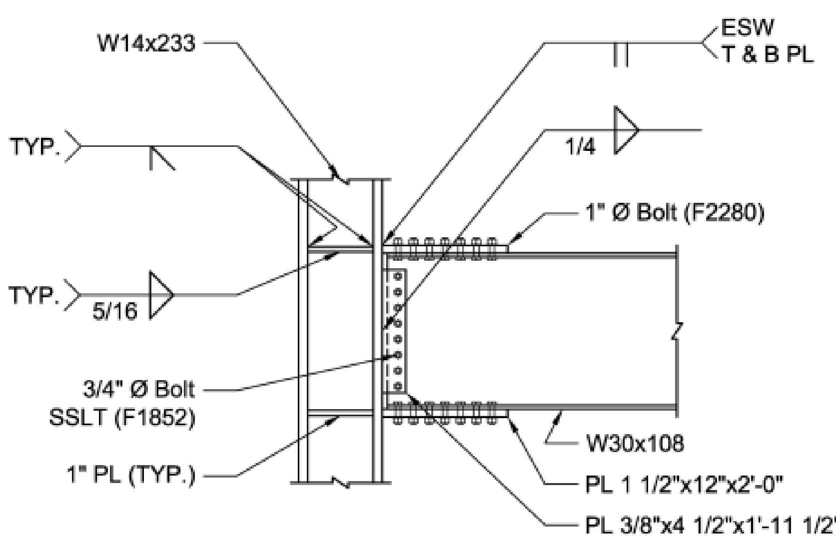


Figure 3.3: Moment connection details for the specimen No. BFP (Sato et al., 2007)

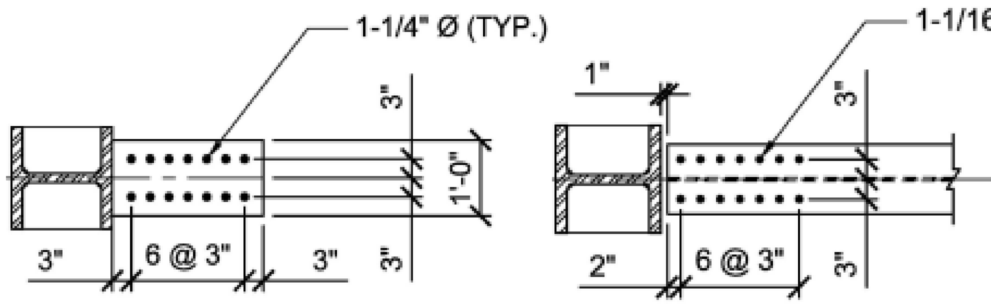


Figure 3.4: Bolt schedule details for the specimen No. BFP: (a) flange plate; and (b) beam flange (Sato et al., 2007)

All bolts were A325 bolts with threads excluded from shear planes. The beam and column sections were made of A992 steel while all plates were made of A572 Gr. 50 steel. The material properties of the members obtained by Colorado Metallurgical Services (CMS) and Certified Mill Test Reports are presented in Table 3.1.

Table 3.1: Steel mechanical properties

Member	Steel Grade	Yield Strength (ksi)	Tensile Strength (ksi)
Column	A992	51.5 (57.0)	76.5 (75.5)
Beam	A992	52.0 (57.0)	77.5 (75.0)
Plate	A572 Gr. 50	60.5 (63.0)	87.5 (85.3)

Note: Values in parentheses are based on Certified Mill Test Reports, others from testing by CMS.

The flange plates were welded to the flange of the column using electroslag welding (ESW) process. Two Arcmatic 105-VMC 3/32 in. diameter electrodes were used. This electrode has a specified minimum Charpy-V Notch Toughness of 15 ft-lbs at -20°F. Flux (FES72) was added by hand per the fabricator’s standard procedure.

The Specimen No. BFP failed by beam flange net section fracture when the interstory drift angle of 0.06 radians was achieved during testing. The applied load-beam tip displacement and the moment at column face-beam rotation relationships are provided in Figures 3.5 and 3.6. The fracture location and beam bottom flange net section fracture are shown in Figures 3.7 and 3.8.

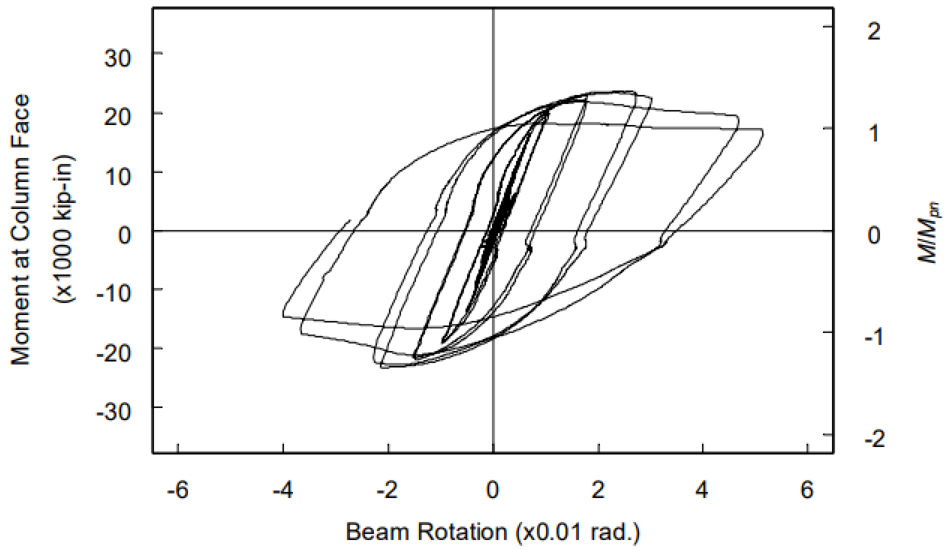


Figure 3.5: Applied load-beam tip displacement (Sato et al., 2007)

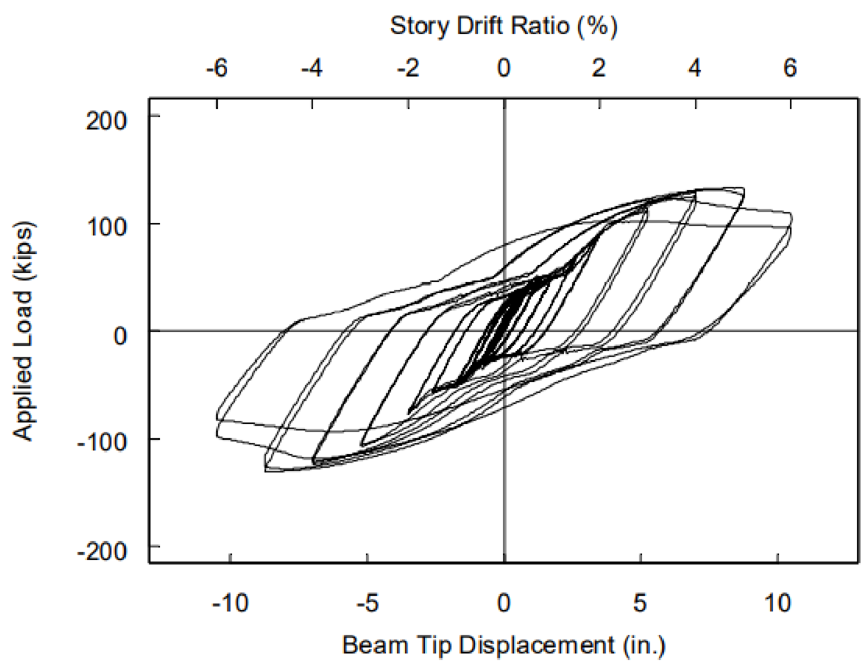


Figure 3.6: Moment at column face-beam rotation (Sato et al., 2007)

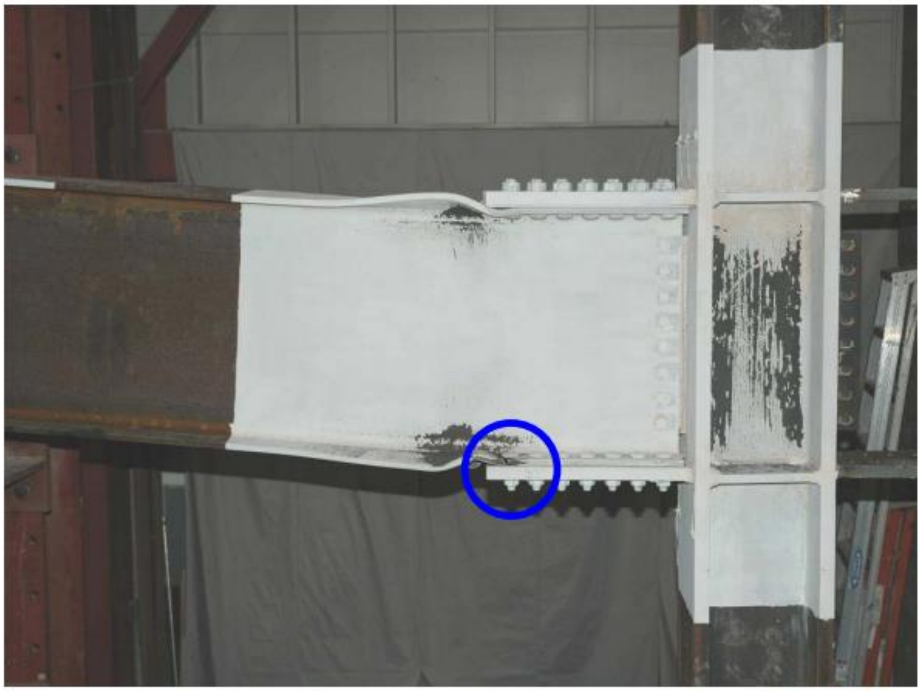


Figure 3.7: Fracture location (Sato et al., 2007)

Figure 3.8: Beam bottom flange net section fracture on 2nd cycle at +6% drift

3.3. Code Design Calculations and Comparisons

The design strength capacities (ϕR_n) of ten rigid connections were calculated following the requirements of AISC Specification for Structural Steel Buildings (AISC 360, 2016) and AISC Steel Construction Manual (AISC Manual, 2017). The nominal strength, R_n, and the corresponding resistance factor, ϕ, for each connection design limit state for load and resistance factored design (LRFD) are provided in Chapter J of AISC 360.

The Specimen No. BFP was selected as a baseline model from the experimental study and nine additional variation models were generated by changing only one parameter at a time from the baseline model. The properties of the baseline and nine additional variation models are shown in Table 3.2. The changing parameters were bolded and italicized.

Table 3.2: Properties of the ten specimens

Speci-men No	Beam	Column	Single Web Plate				Flange Plates		
			Web Plate Geometry (in.)	Weld Size (in.)	Bolt Schedule	Bolt Dia. (in.)	Flange Plates (in.)	Bolt Dia. (in.)	Bolt Schedule
BFP	W30x108	W14x233	3/8x4.5x23.5	1/4	8x1	3/4	1.5x12x24	1	7x2
Model 1	W30x108	W14x233	***1/4x4.5x23.5***	1/4	8x1	3/4	1.5x12x24	1	7x2
Model 2	W30x108	W14x233	3/8x4.5x23.5	1/4	8x1	***1/2***	1.5x12x24	1	7x2
Model 3	W30x108	W14x233	3/8x4.5x23.5	1/2	***6x1***	3/4	1.5x12x24	1	7x2
Model 4	W30x108	***W14x311***	3/8x4.5x23.5	1/4	8x1	3/4	1.5x12x24	1	7x2
Model 5	W30x108	***W14x370***	3/8x4.5x23.5	1/4	8x1	3/4	1.5x12x24	1	7x2
Model 6	W30x108	W14x233	3/8x4.5x23.5	1/4	8x1	3/4	***1x12x24***	1	7x2
Model 7	W30x108	W14x233	3/8x4.5x23.5	1/4	8x1	3/4	1.5x12x24	***3/4***	7x2
Model 8	W30x108	W14x233	3/8x4.5x23.5	1/4	8x1	3/4	1.5x12x24	1	***5x2***
Model 9	W30x108	W14x233	3/8x4.5x23.5	1/4	8x1	3/4	1.5x12x24	1	***3x2***

3.3.1 Design strength capacity of single web plates

The following eight design checks were performed according to the LRFD design equations included in AISC 360 or AISC Manual for the design strength capacities of single web plate.

1. Web Plate
    a. Bolts shear — Eq. J3-1, AISC 360-16
    b. Bolt bearing and tearout — Eq. J3-6, AISC 360-16
    c. Shear yielding — Eq. J4-3, AISC 360-16
    d. Shear rupture — Eq. J4-4, AISC 360-16
    e. Block shear — Eq. J4-5, AISC 360-16
    f. Weld shear — Eq. 8-2, AISC Manual
2. Beam
    a. Bolts shear — Eq. J3-1, AISC 360-16
    b. Bolt bearing and tearout — Eq. J3-6, AISC 360-16

The design strength capacities (ϕR_n) of single web plates of the ten specimens calculated by following AISC LRFD code requirements are provided in Table 3.3. The lowest shear capacities were bolded and italicized. Out of the calculated design capacities for the ten test specimens, the design capacity of model 2 was controlled by shear rupture while bolt shear led to failure for the other eight specimens.

Table 3.3: The design strength capacities (ϕR_n) of sing web-plates of the ten specimens

Design Checks (LRFD)	**Design Strength Capacity (kips)**									
Specimen Number	**BFP**	**Model 1**	**Model 2**	**Model 3**	**Model 4**	**Model 5**	**Model 6**	**Model 7**	**Model 8**	**Model 9**
Web Plate										
- Bolt shear	***180.32***	180.32	***80.14***	135.24	***180.32***	***180.32***	***180.32***	***180.32***	***180.32***	***180.32***
- Bolt bearing and tearout	263.25	175.50	175.50	197.44	263.25	263.25	263.25	263.25	263.25	263.25
- Shear yielding	264.30	176.20	264.30	196.82	264.30	264.30	264.30	264.30	264.30	264.30
- Shear rupture	181.06	***120.71***	202.07	***134.41***	181.06	181.06	181.06	181.06	181.06	181.06
- Block shear	191.00	127.66	209.44	144.88	191.00	191.00	191.00	191.00	191.00	191.00
- Weld shear	261.70	261.70	261.70	194.90	261.70	261.70	261.70	261.70	261.70	261.70
Beam										
- Bolt bearing and tearout	382.59	382.59	255.06	286.94	382.59	382.59	382.59	382.59	382.59	382.59
- Shear yielding	487.72	487.72	487.23	804.96	487.72	487.72	487.72	487.72	487.72	487.72

3.3.2 Design strength capacity of flange plates

The following 13 design checks were performed according to the LRFD design equations included in AISC 360 or AISC Manual for the design strength capacities of flange plates.

1. Flange Plate
    a. Bolts shear — Eq. J3-1, AISC 360-16
    b. Bolt bearing and tearout — Eq. J3-6, AISC 360-16
    c. Tensile yielding — Eq. J4-3, AISC 360-16
    d. Tensile rupture — Eq. J4-4, AISC 360-16
    e. Block shear — Eq. J4-5, AISC 360-16
    f. Compression — Sec. J4-4, AISC 360-16
2. Beam
    a. Bolt bearing and tearout — Eq. J3-6, AISC 360-16
    b. Flexural — Sec. F13.1, AISC 360-16

    c. Block shear — Eq. J4-5, AISC 360-16

3. Column
    a. Panel web shear — Eq. J10-9, AISC 360-16
    b. Flange local bending — Eq. J10-1, AISC 360-16
    c. Web local yielding — Eq. J10-2, AISC 360-16
    d. Web local crippling — Eq. J10-4, AISC 360-16

The design strength capacities (ϕR_n) of flange plates of ten specimens calculated by following AISC LRFD code requirements are provided in Table 3.4. The lowest shear capacities were shown in bold and italic.

Table 3.4: The design strength capacities (ϕR_n) of flange plates of ten specimens

Design Checks (LRFD)	**Design Strength Capacity (kips)**									
Specimen Number	**BFP**	**Model 1**	**Model 2**	**Model 3**	**Model 4**	**Model 5**	**Model 6**	**Model 7**	**Model 8**	**Model 9**
Flange Plate										
- Bolt shear	692.44	692.44	692.44	692.44	692.44	692.44	692.44	**389.84**	496.6	**296.76**
- Bolt bearing and tearout	2,402.60	2,402.60	2,402.60	2,402.60	2,402.60	2,402.60	1,605.20	2,760.60	1,748.00	1,088.1
- Tensile yielding	810.00	810.00	810.00	810.00	810.00	810.00	540.00	810.00	810.00	810.00
- Tensile rupture	704.00	704.00	704.00	704.00	704.00	704.00	469.30	665.60	704.00	704.00
- Block shear	1,517.30	1,517.30	1,517.30	1,517.30	1,517.30	1,517.30	1,011.60	1,718.40	1,199.30	881.20
- Compression	810.00	810.00	810.00	810.00	810.00	810.00	540.00	810.00	810.00	810.00
Beam										
- Bolt bearing and tearout	1,128.18	1,128.18	1,128.18	1,128.18	1,128.18	1,128.18	1,128.18	933.66	1297.50	461.27
- Block shear	668.95	668.95	668.95	668.95	668.95	***668.95***	668.95	770.64	507.59	346.42
- Flexural	1,297.50	1,297.50	1,297.50	1,297.50	1,297.50	1,297.50	1,297.50	1,297.50	794.72	1,297.50
Column										
- Web panel zone shear	***462.24***	***462.24***	***462.24***	***462.24***	***651.00***	802.28	***462.24***	462.24	***462.24***	462.24
- Flange local bending	832.05	832.05	832.05	832.05	1436.51	1990.01	832.05	832.05	832.05	832.05
- Web local yielding	700.85	700.85	700.85	700.85	1113.90	1477.40	700.85	700.85	700.85	700.85
- Web local crippling	1,193.12	1,193.12	1,193.12	1,193.12	2,053.80	2,832.00	1,193.12	1,193.12	1,193.12	1,193.12

Out of the calculated design capacities for the ten test specimens, the design capacity of seven specimens was controlled by web panel zone shear, two specimens were controlled by bolt shear and one specimen was controlled by block shear. The moment capacities of the specimens were calculated by multiplying by the controlling design capacity by the moment arm as provided in Table 3.5. The moment arm is equal to the depth of the beam for bolt shear while it is equal to the summation of the depth of the beam and the thickness of the plate for web panel zone shear and block shear strengths (BFP, models 1, 2, 3, 4, 5, 6 and 8). The detailed design strength calculations for Test No. BFP are provided in Appendix I.

Table 3.5: The moment capacities of the ten rigid connection specimens

Specimen number	Governing strength (kips)	Length of the moment arm (in.)	Moment capacity (kips-in.)
BFP	462.24	31.33	14,481.98
Model 1	462.24	31.33	14,481.98
Model 2	462.24	31.33	14,481.98
Model 3	462.24	31.33	14,481.98
Model 4	651.00	31.33	20,395.83
Model 5	668.95	29.83	19,954.78
Model 6	462.24	30.83	14,250.86
Model 7	389.84	29.83	11,628.93
Model 8	462.24	31.33	14,481.98
Model 9	296.76	29.83	8,852.351

3.3.3 Calculated ASD Design Strength Capacities of Test No. BFP

According to allowable strength design (ASD), the allowable strength (R_n/Ω) is calculated by dividing the nominal strength, R_n by the safety factor, Ω. The allowable strength capacities of connection specimen Test No. BFP are calculated by following the AISC ASD code requirements. The properties of this test specimen were given in Table 3.2. The calculated ASD design strength capacities (R_n/Ω) of the specimen for the web plate and flange plates are provided in Tables 3.6 and 3.7, respectively.

Table 3.6: The design strength capacity (ϕR_n) of the web-plate for Test No. BFP

Design Checks (ASD)	Design Strength Capacity (kips)
Web Plate	
- Bolt shear	***120.22***
- Bolt bearing and tearout	175.52
- Shear yielding	176.20
- Shear rupture	120.71
- Block shear	127.34
- Weld capacity	174.46
Beam	
- Bolt bearing and tearout	227.60
- Shear yielding	325.20

Table 3.7: The design strength capacity (ϕR_n) of the flange plates for Test No. BFP

Design Checks (ASD)	**Design Strength Capacity (kips)**
Flange Plate	
- Bolt shear	461.58
- Bolt bearing and tearout	1601.76
- Tensile yielding	538.92
- Tensile rupture	469.30
- Block shear	1011.53
- Compression	538.92
Beam	
- Bolt bearing and tearout	752.12
- Block shear	445.97
- Flexural strength	863.27
Column	
- Web panel zone shear	***307.54***
- Flange local bending	553.59
- Web local yielding	467.23
- Web local crippling	795.42

The calculated lowest strength of the web plate for Test No. BFP is 101.01 kips due to the bolt shear failure while the controlling strength of the flange plate is 307.54 kips because of web panel zone shear failure. The moment capacities of the specimen can be calculated by multiplying by the governing strength of the flange plate (307.54 kips) by the distance of the moment arm (31.33 in.) which is equal to the depth of the beam (29.83 in.) plus one plate thickness (1.5 in.). The detailed design strength calculations for Test No. BFP are provided in Appendix J.

3.4. IDEA StatiCa Analysis

3.4.1 Moment capacity analysis using IDEA StatiCa

The ten rigid steel connection specimens were modeled in IDEA StatiCa and analyzed under a shear force applied at 177.5 in. away from the column centerline as in the test report. The shear force was increased incrementally until the connections reached their capacities in IDEA StatiCa. The calculated maximum moment capacities are shown in Table 3.8.

Table 3.8: LRFD moment capacities of the specimens calculated by IDEA StatiCa

Specimen	IDEA StatiCa			
	Shear force (kips)	Moment Arm (in.)	Moment (kips-in.)	Failure Mode
BFP	96.70	177.50	17,164.25	Beam flange failure (limit plastic strain, 5%)
Model 1	96.05	177.50	17,048.88	Bolt shear failure on web plate
Model 2	96.00	177.50	17,040.00	Bolt shear failure on web plate
Model 3	96.10	177.50	17,057.75	Bolt shear failure on web plate
Model 4	100.20	177.50	17,785.50	Beam flange failure (limit plastic strain, 5%)
Model 5	100.40	177.50	17,821.00	Beam flange failure (limit plastic strain, 5%)
Model 6	89.70	177.50	15,921.75	Flange plate failure (limit plastic strain, 5%)
Model 7	64.00	177.50	11,360.00	Bolt shear failure on flange plate
Model 8	92.00	177.50	16,330.00	Beam flange failure (limit plastic strain, 5%)
Model 9	61.00	177.50	10,827.50	Bolt shear failure on flange plate

The IDEA StatiCa screenshots showing the failure modes as well as the deformed shapes of finite element models are shown in Appendix K.

3.4.2 Moment-rotation analysis

Moment-rotation analysis for Test No. BFP was performed using IDEA StatiCa. To be able to generate the test condition, the mean values of yielding and tensile strength of the materials measured by CMS and Certified Mill Test Reports were used (Table 3.9). The mean values defined in IDEA StatiCa and the resistance factors were adjusted to 1.0. Then, the moment-rotation analysis was performed by choosing stiffness analysis option (Figure 3.9).

Table 3.9: Mean values of measured material properties

Member	Steel Grade	Yield Strength (ksi)	Tensile Strength (ksi)
Column	A992	54.25	76
Beam	A992	54.5	76.25
Plate	A572 Gr. 50	61.75	86.4

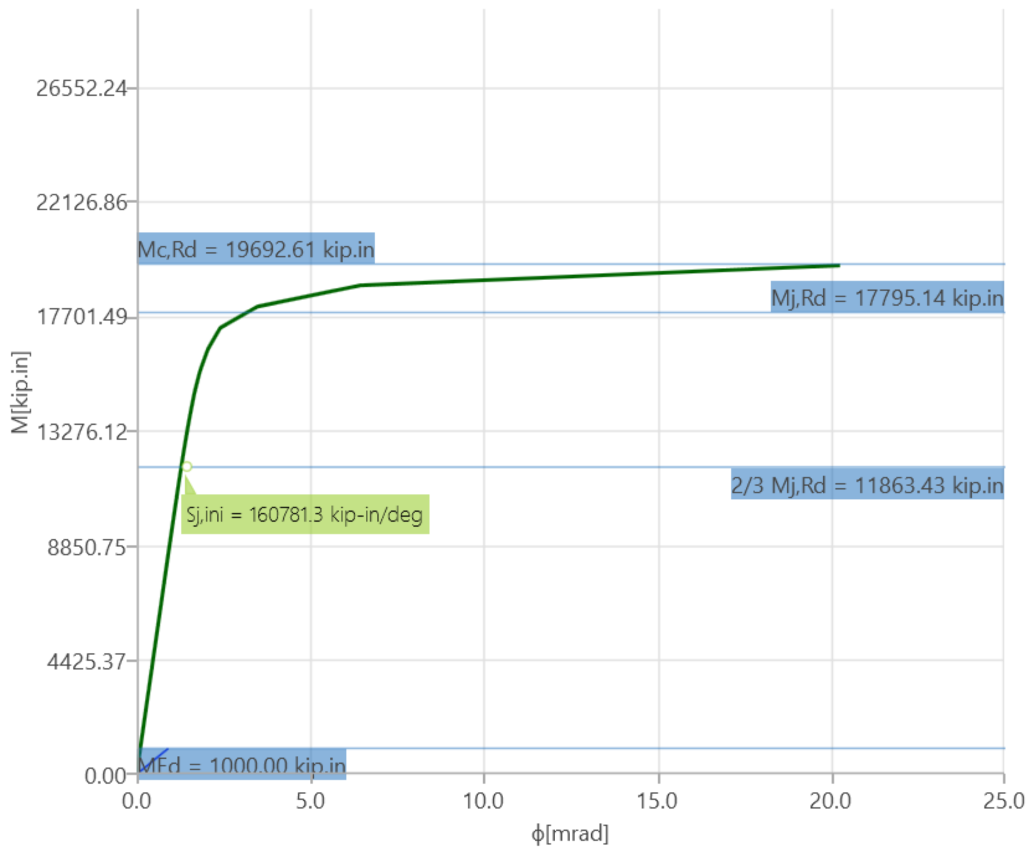


Figure 3.9: Moment-rotation relationship for Test No. BFP (IDEA StatiCa)

3.5. ABAQUS Analysis

In this section the output results from IDEA StatiCa were compared to ABAQUS software package (version 2020). In this study, Test BFP, as described in Table 3.2, was chosen as a base model. Numerical simulations with almost identical conditions (i.e., in terms of material properties, boundary condition, and loading) were carried out using both IDEA StatiCa and ABAQUS. The model was initially designed in IDEA StatiCa and then the assembly (including beam, column, and plates) was imported to ABAQUS using the IDEA StatiCa's viewer platform. Afterward, a simplified model for the bolt was designed and added to the ABAQUS model (see Figure 3.10).

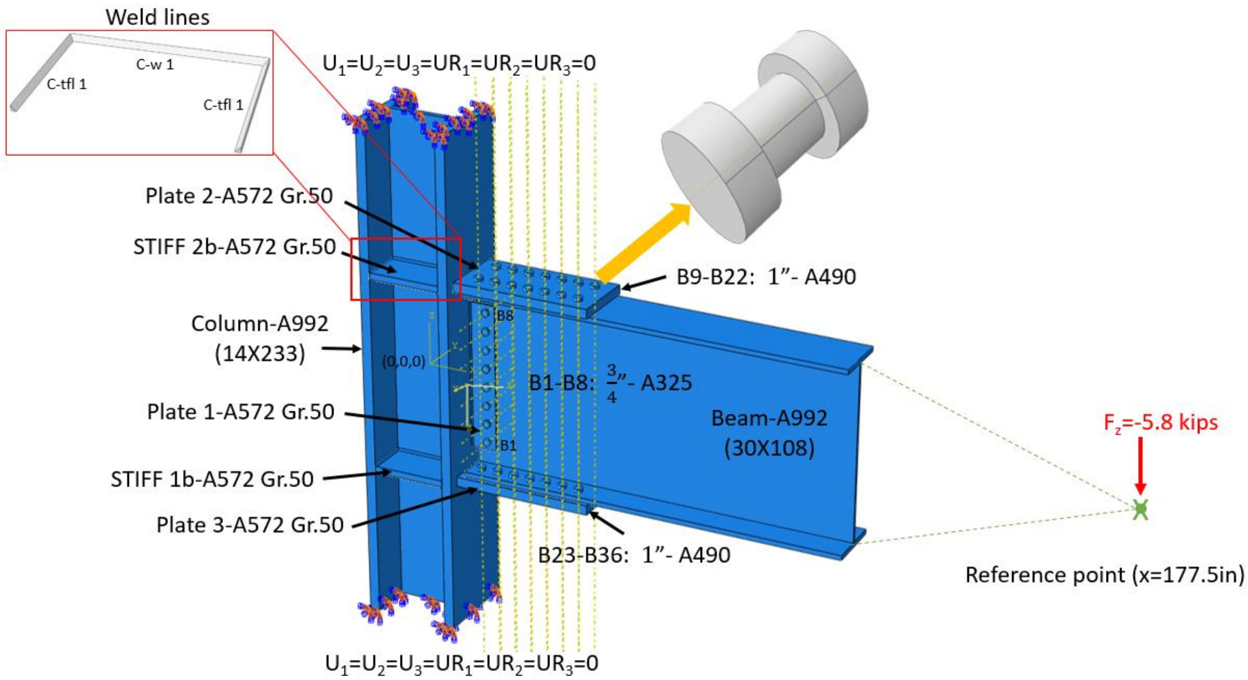


Figure 3.10: Model setup in ABAQUS

In ABAQUS, the element type was C3D8R (3D stress, 8-node linear brick, reduced integration), and a total of 681,016 elements were generated in the model (see Table 3.10 and Figure 3.11 for more details).

Table 3.10. Number of elements in the ABAQUS model

Item	Number of Elements
Column	100,569
Beam	38,091
Plate 1	5,164
Plates 2, 3	7,892
Stiffener	1,824
Bolt ¾ in.	7,032
Bolt 1.0 in.	16,152
Weld group	700

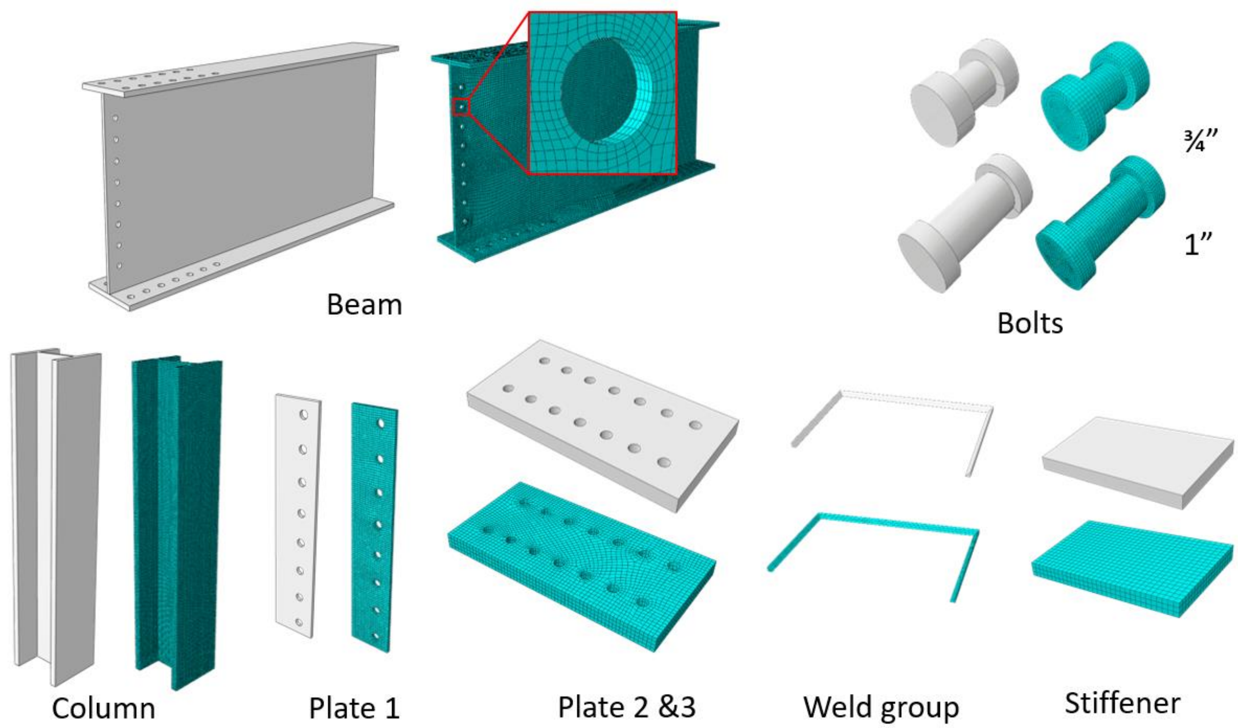


Figure 3.11: ABAQUS model mesh densities

In the ABAQUS model, the vertical shear force of 5.8 kips was applied on a reference point (or node) that was defined 177.5 in. away from the center of the column (i.e., x = 177.5 in.). Then, the coupling constraint (i.e., structural distributing) was defined to connect this reference point to the end section of the beam. The top and bottom of the column were fixed as a boundary condition (see Figure 3.10). The contact between all parts was defined as surface-to-surface with finite sliding formulation. Friction was defined with a penalty method, and a Coulomb friction coefficient of $\mu = 0.3$ was used between all the parts in contact. The top and bottom plates (i.e., Plate 2 and Plate 3) were also welded to the column.

The material behavior was modeled using a bi-linear plasticity in ABAQUS. Other parameters including density, elastic modulus, and poisons ratio were copied from the IDEA StatiCa materials library. The numerical simulations were carried out on eight processors (Intel Xenon (R) CPU E5-2698 v4 @ 2.20GHz) and the simulation took approximately 685 minutes. Figure 3.12 depicts the comparison between the predicted von-Mises stress in IDEA StatiCa and ABAQUS. Figure 3.13 also shows the side view in which the deformation scale factor of 20 was applied to models in both sotfware.

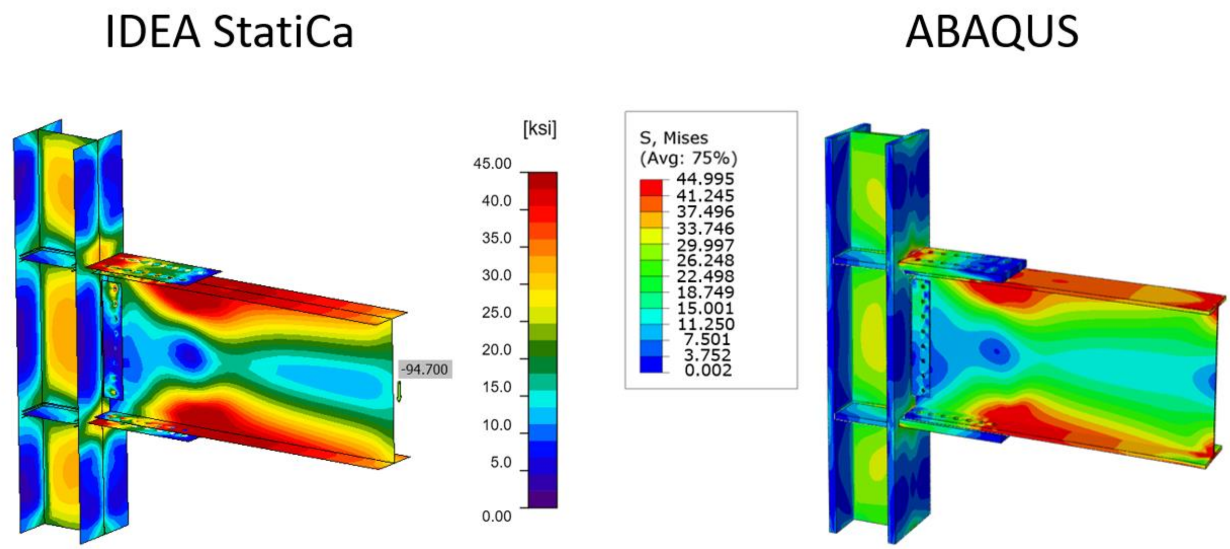


Figure 3.12: Comparison of the predicted von-Mises stress between IDEA StatiCa and ABAQUS

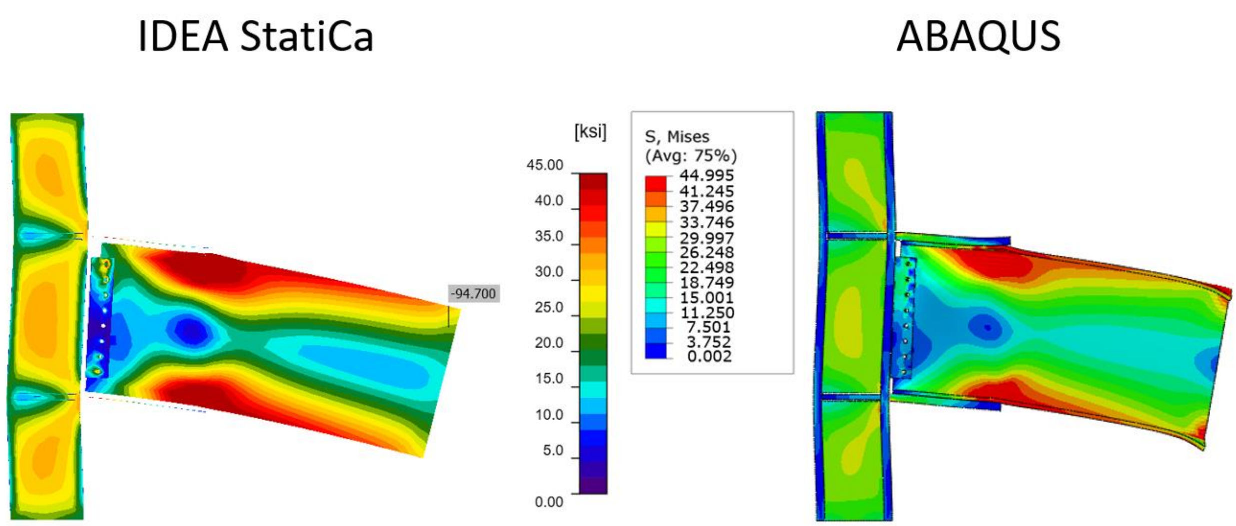


Figure 3.13: Side view comparison between IDEA StatiCa and ABAQUS with deformation scale factor of 20

3.6 Summary, Comparison and Results

3.6.1 Comparison of IDEA StatiCa Analysis Data, AISC Design Strengths, and Test Data

The design strength capacities (ϕR_n) of the ten rigid steel connections were calculated following the requirements of AISC Specification for Structural Steel Buildings (AISC 360, 2016) and AISC Steel Construction Manual (AISC Manual, 2017). The smallest calculated strengths were determined, and the moment capacities of the connections were obtained corresponding these controlling strengths.

The same specimens were modeled in IDEA StatiCa and analyzed under a shear force applied 117.5 in. away from the column centerline. The shear force was increased incrementally until the connections reached their capacities. The moment capacities of the connections were obtained by multiplying by the distance between where the shear force was applied, and the ultimate shear force reached in the incremental loading. The results are compared in Table 3.10.

Table 3.10: Comparison of the moment capacities of the specimens calculated from AISC design equations and IDEA StatiCa

Specimen Number	**AISC Design Strength Moment Capacities (kips-in.)**	**IDEA StatiCa Analysis, Moment Capacities (kips-in.)**
BFP	14,481.98	17,164.25
Model 1	14,481.98	17,048.88
Model 2	14,481.98	17,040.00
Model 3	14,481.98	17,057.75
Model 4	20,395.83	17,785.50
Model 5	19,954.78	17,821.00
Model 6	14,250.86	15,921.75
Model 7	11,628.93	11,360.00
Model 8	14,481.98	16,330.00
Model 9	8,852.351	10,827.50

The moment-rotation relationship of the Test No. BFP was calculated from IDEA StatiCa analysis using the mean values of material properties of the tested specimens measured in the experimental study and compared with the moment-rotation relationship obtained during static loading provided in the test report (Figure 3.14).

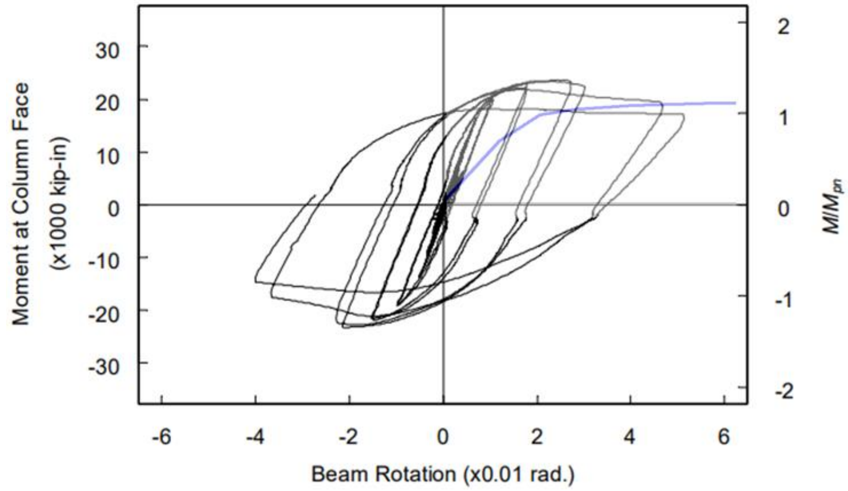


Figure 3.14: Comparison of the moment-rotation relationships of Test No. BFP measured during the experiment and calculated from IDEA StatiCa (blue line)

3.6.3 Comparison of IDEA StatiCa and ABAQUS Results

The comparison between the IDEA StatiCa and ABAQUS results were summarized in Tables 3.11-3.13. As shown in Figure 3.12 and Tables 3.11-3.13, there was a good agreement between the results of the two software packages. The stress distributions on the beam and column were very close. However, slightly higher stresses were predicted on the column, plate 1, and stiffeners in the ABAQUS model which is most likely due to the nature of the tie constraint. The predicted load on the bolts and weld groups were also very close between the two software. Stress distribution for each part can be seen in Appendix L.

Table 3.11. Specified yield strengths and calculated stress, strain, and plates check status

IDEA StatiCa					ABAQUS				
Item	F_y (ksi)	σ_{Ed} (ksi)	ε_{pl} (%)	**Check Status**	**Item**	F_y (ksi)	σ_{Ed} (ksi)	ε_{pl} (%)	**Check Status**
C-bfl 1	50.0	37.4	0	Ok	**Column**	50	44.995	0	Ok
C-tfl 1	50.0	24.5	0	Ok					
C-w 1	50.0	32.9	0	Ok					
B-bfl 1	50.0	45.3	1.0	Ok	**Beam**	50	44.995	3.9	Ok
B-tfl 1	50.0	46.4	5.0	Ok					
B- w 1	50.0	45.1	0.5	Ok					
STIFF 1a	50.0	19.5	0.0	Ok	**Back “bottom stiffener”**	50	26.618	0	Ok
STIFF 1b	50.0	19.5	0.0	Ok	**Front “bottom stiffener”**	50	26.531	0	Ok
FP 1	50.0	39.6	0.0	Ok	**Plate 1**	50	44.995	0.2	Ok
FP 2	50.0	45.1	0.2	Ok	**Plate 2**	50	44.995	0.4	Ok
FP 3	50.0	45.1	0.2	Ok	**Plate 3**	50	44.995	0.5	Ok
STIFF 2a	50.0	19.2	0.0	Ok	**Back “top stiffener”**	50	31.836	0	Ok
STIFF 2b	50.0	19.1	0.0	Ok	**Front “top stiffener”**	50	31.736	0	Ok

Table 3.12. Calculated tension force, shear force, and bolt bearing resistance

	IDEA StatiCa					ABAQUS				
Item	F_t $(kips)$	V $(kips)$	$\emptyset R_{n,bearing}$ $(kips)$	Ut_t (%)	Ut_s (%)	F_t $(kips)$	V $(kips)$	$\emptyset R_{n,bearing}$ $(kips)$	Ut_t (%)	Ut_s (%)
B1	1.234	10.277	32.906	4.1	45.6	1.156	9.956	34.551	3.6	40.2
B2	0.839	7.145	32.906	2.8	31.7	0.742	6.821	34.551	2.3	27.6
B3	0.412	3.732	32.906	1.4	16.6	0.445	3.863	34.551	1.4	15.6
B4	0.088	0.230	32.906	0.3	1.0	0.132	0.228	34.551	0.4	1.0
B5	0.657	3.156	24.018	2.2	14.0	0.698	3.191	34.551	2.2	12.9
B6	1.292	6.400	24.046	4.3	28.4	1.305	6.556	34.551	3.9	26.5
B7	1.874	6.400	24.072	4.3	28.4	1.945	6.674	34.551	6.0	27.0
B8	2.773	11.864	24.087	9.3	52.6	2.669	11.772	34.551	8.2	47.5
B9 (IDEA B29)	11.318	35.808	63.899	17.0	72.4	11.058	35.995	93.3471	16.7	72.9
B10 (IDEA B36)	11.140	35.625	63.899	16.7	72.0	11.015	35.995	93.3471	16.6	72.9
B11 (IDEA B28)	3.800	36.199	83.345	5.7	73.2	3.745	36.124	93.3471	5.7	43.2
B12 (IDEA B35)	3.469	36.057	83.345	5.2	72.9	3.562	36.325	93.3471	5.4	73.6
B13 (IDEA B27)	3.071	35.761	83.345	4.6	72.3	3.234	36.034	93.3471	4.9	73.0
B14 (IDEA B34)	2.757	35.688	83.345	4.1	72.1	2.945	35.956	93.3471	4.5	72.8
B15 (IDEA B23)	2.345	35.605	83.345	3.5	72.0	2.567	35.945	93.3471	3.9	72.8
B16 (IDEA B30)	2.203	35.572	83.345	3.3	71.9	2.456	35.857	93.3471	3.7	72.6
B17 (IDEA B24)	2.802	35.943	83.345	4.2	72.7	2.935	36.234	93.3471	4.5	73.4
B18 (IDEA B31)	2.722	35.934	83.345	4.1	72.6	2.845	36.145	93.3471	4.3	73.2
B19 (IDEA B25)	5.883	36.820	83.345	8.8	74.4	5.994	36.945	93.3471	9.1	74.8
B20 (IDEA B32)	5.862	36.820	83.345	8.8	74.4	5.905	36.94	93.3471	8.9	74.8
B21 (IDEA B26)	14.770	38.300	83.345	22.2	77.4	14.95	38.54	93.3471	22.5	78.1
B22 (IDEA B33)	14.770	38.310	83.345	22.2	77.4	14.95	38.54	93.3471	22.5	78.1
B23 (IDEA B57)	0.645	35.969	83.345	1.0	72.7	0.539	36.836	93.3471	0.9	74.6
B24 (IDEA B64)	0.660	35.892	83.345	1.0	72.6	0.634	36.12	93.3471	1	73.2
B25 (IDEA B56)	1.419	35.547	83.345	2.1	71.9	1.525	35.445	93.3471	2.3	71.8

B26 (IDEA B63)	1.472	35.501	83.345	2.2	71.8	1.59	35.966	93.3471	2.4	72.9
B27 (IDEA B55)	1.493	35.320	83.345	2.2	71.4	1.609	35.82	93.3471	2.5	72.6
B28 (IDEA B62)	1.536	35.312	83.345	2.3	71.4	1.635	35.745	93.3471	2.5	72.4
B29 (IDEA B51)	1.390	35.729	83.345	2.1	72.2	1.456	36.09	93.3471	2.2	73.1
B30 (IDEA B58)	1.418	35.743	83.345	2.1	72.3	1.467	36.09	93.3471	2.3	73.1
B31 (IDEA B52)	1.730	36.850	83.345	2.6	74.5	1.837	36.935	93.3471	2.8	74.8
B32 (IDEA B59)	1.732	36.877	83.345	2.6	74.5	1.845	36.894	93.3471	2.8	74.7
B33 (IDEA B53)	1.320	38.689	83.345	2.0	78.2	1.421	38.562	93.3471	2.2	78.1
B34 (IDEA B60)	1.340	38.742	83.345	2.0	78.3	1.456	38.63	93.3471	2.2	78.2
B35 (IDEA B54)	10.395	40.663	83.345	15.6	82.2	10.123	41.345	93.3471	15.3	83.7
B36 (IDEA B61)	10.398	40.682	83.345	15.6	82.2	10.141	41.366	93.347	15.3	83.8

Table 3.13. Calculated force in weld critical element, weld resistance, and welds check status

		IDEA StatiCa				ABAQUS			
Item	**Edge**	F_n $(kips)$	$\emptyset R_n$ $(kips)$	Ut (%)	***Status***	F_n $(kips)$	$\emptyset R_n$ $(kips)$	Ut (%)	***Status***
C-bfl 1	STIFF1a	11.900	15.370	77.4	OK	12.856	16.238	79.2	OK
		12.123	15.414	78.6	OK	12.959	16.238	79.8	OK
C-w 1	STIFF1a	6.939	12.317	56.3	OK	7.234	14.235	50.8	OK
		6.613	12.423	53.2	OK	7.129	14.235	50.1	OK
C-tfl 1	STIFF1a	6.206	13.375	46.4	OK	6.945	15.238	45.6	OK
		5.790	13.171	44.0	OK	6.536	15.238	42.9	OK
C-bfl 1	STIFF1b	12.113	15.415	78.6	OK	13.532	16.235	83.4	OK
		11.907	15.371	77.5	OK	13.134	16.235	80.9	OK
C-w 1	STIFF1b	6.598	12.427	53.1	OK	7.452	13.532	55.1	OK
		6.937	12.314	56.3	OK	7.848	13.532	58.0	OK
C-tfl 1	STIFF1b	5.787	13.172	43.9	OK	5.994	14.235	42.1	OK
		6.207	13.381	46.4	OK	6.345	14.235	44.6	OK
C-bfl 1	FP1	8.010	11.942	67.1	OK	8.556	12.265	69.8	OK
		6.915	11.855	58.3	OK	7.456	12.265	60.8	OK
C-bfl 1	STIFF12a	12.073	15.409	78.4	OK	12.768	17.238	74.1	OK
		11.859	15.359	77.2	OK	12.568	17.238	73.0	Ok
C-w 1	STIFF2a	6.534	12.406	52.7	OK	6.876	14.235	48.3	OK
		6.717	12.349	54.4	OK	6.978	14.235	49.0	OK
C-tfl 1	STIFF2a	5.761	13.260	43.4	OK	6.125	16.238	37.7	OK
		6.053	13.346	45.4	OK	6.532	16.238	40.2	OK
C-bfl 1	STIFF12b	11.867	15.360	77.3	OK	12.452	17.265	72.1	OK
		12.054	15.410	78.2	OK	12.645	17.265	73.2	OK
C-w 1	STIFF2b	6.717	12.343	54.4	OK	6.883	14.235	48.4	OK
		6.508	12.416	52.4	OK	6.475	14.235	45.5	OK
C-tfl 1	STIFF2b	6.051	13.364	45.3	OK	6.73	16.238	41.5	OK
		5.760	13.255	43.5	OK	6.134	16.238	37.8	OK

CHAPTER 4 SUMMARY AND CONCLUSIONS

IDEA StatiCa is a component-based finite element analysis (FEA) software package for steel connection design. It can be used for structural evaluation or design of a variety of welded and bolted structural steel connections and base plates. The main objective of this report was to verify the FEA results obtained from the IDEA StatiCa software package for three types of steel connections commonly used in the United States (i.e., simple, semi-rigid, and rigid) according to the U.S. building codes. Measured experimental response was available for the connection specimens selected for verification purposes in this study. For each connection type and ten variations of that, first, the code checks and calculations were performed following the requirements of the AISC 360, Specification for Structural Steel Building (2016), and AISC Steel Construction Manual (2017) codes. Then, the results were compared with the IDEA StatiCa predictions. Additionally, the results from the IDEA StatiCa were compared with ABAQUS, which is another robust FEA code in the market. Measured responses of the test specimens were also used to compare and better understand the overall behavior and failure mode of connection models.

In general, there was good agreement between the IDEA StatiCa results, code-checks according to the U.S. codes, and the ABAQUS results. The calculated results are different than those obtained with IDEA StatiCa possibly because AISC is a design code and may be conservative while the software is intended to capture the real behavior which is expected to be more accurate.

While there are many FEA software packages in the market capable of predicting the overall structural response to a variety of loading conditions, there is a lack of specialized FEA tools with a focus on connection design. Compared to other FEA software packages in the market, IDEA StatiCa software has many advantages. In addition to the ease of use, the most important characteristic of IDEA StatiCa was found to be the computational time in which the results can be obtained in a fraction of time compared to the conventional FEA codes such as ABAQUS. This will help the engineers to evaluate and modify their preliminary connection design faster and in a more efficient way if any changes are required. In addition, in common FEA software packages, the loads and capacities of the connection members (i.e., bolts, welds, plates) should be extracted from the model during the post-processing stage which is a cumbersome and time-consuming task. However, in IDEA StatiCa, the results are directly calculated and reported. Also, in IDEA StatiCa, load can be directly applied at any locations/members of the connection, while in typical FEA codes this should be done through defining the reference point and then coupling it with the connection which is an extra step.

A minor discrepancy, however, was found where the contacts defined between the plates and column/beam faces, although the same type of analysis was performed, i.e., small deformation. This could be due to the differences between solid elements and shell elements or contact algorithm(s) that are used in two software. Also, the way that IDEA StatiCa code calculates and utilizes the optimum element size was not clear. Additionally, due to the recommended plastic strain limit of 5% by Eurocode (EN1993-1-5 app. C par. C8 note 1) which is defined as a default value in the IDEA StaiCa software, different failure modes were observed.

Because of IDEA StatiCa's fast and easy connection modeling and analysis capabilities, complicated nonlinear modeling and time-consuming dynamic analysis of large steel structures can be performed relatively quickly. Properties of the connections in beam-column frame structures can be defined based on the analysis and design checks completed in IDEA StatiCa. Then, the connection model can be revised and re-analyzed if necessary after the frame analysis is completed using a structural analysis software, e.g., SAP2000. The connections can be made weaker or stronger in IDEA StatiCa depending on the desired optimum performance of the structural frame model. An easy and more robust approach to develop connection moment-rotation response in IDEA StatiCa will be very helpful because in programs like SAP2000 moment-rotation response of connections need to be defined as part of modeling of frame structures.

IDEA StatiCa software is only as good as its Graphical User Interface. If the GUI is not well executed, users will have trouble with using the application or the software. IDEA StatiCa design it well. Along with a good GUI — the quality of the software is also observed. Following a set of conventions or standards ensures consistency and makes it easy for users to navigate in the software. A standard and consistent language ensures users will understand terms when they see them. Models are easily modified, allowing fast variable exploration and checking.

The software consistently updates including faster load times, and even bug fixes to improve the overall user experience.

References

[1] AISC (2016). "Specification for Structural Steel Buildings," American Institute of Steel Construction ANSI/AISC 360-16, Chicago, Illinois.

[2] AISC (2017). "Steel Construction Manual," 15th edition, American Institute of Steel Construction, Chicago, Illinois.

[3] McMullin, K. M., & Astaneh-Asl, A. (1988). Analytical and experimental studies of double-angle framing connections. Structural Engineering, Mechanics, and Materials, Department of Civil Engineering, University of California, Berkeley.

[4] ABAQUS 2020, Dassault Systemes Simulia Corporation, Providence, RI, USA.

[5] IDEA StatiCa s.r.o., Sumavska 519/35, Brno, 602 00 Czech Republic; https://www.ideastatica.com/support-center/general-theoretical-background

[6] Azizinamini, A., Bradburn, J. H., and Radziminski, J. B. (1985). Static and cyclic behavior of semi-rigid steel beam-column connections. University of South Carolina.

[7] Sato, A., Newell, J., and Uang, C. M. (2007). Cyclic testing of bolted flange plate steel moment connections for special moment frames. Final Repor to American Institute of Steel Construction.

Appendix A. LRFD Strength Calculations for Simple Bolted-to-Beam and Welded-to-Column Connection Test Specimen (Test No. 4)

A.1 Loading and Assumptions

During testing of connection specimen Test No. 4, a setup was used to prevent moment in the beam and allow unrestrained rotation of the beam at the connection as much as possible (see Figure 1.1 and Figure A.1 below). Therefore, for design calculations in this Appendix, it is assumed that the simple or pin connection specimens, including Test No. 4 in this appendix, the double angle connection carries the applied shear force, and no moment is carried by the simple connection.

It is assumed that the following dead and live loads are applied at the tip of the cantilever.

P_D = 30 kips (dead load)

P_L = 35 kips (live load)

A.1.1 LRFD Load Demand or Design Loads

According to the Load and Resistance Factored Design (LRFD) procedure (AISC 360-16 or AISC Manual 2017), LRFD design load, factored load or demand, P_u under gravity loads is:

$P_u = 1.2P_D + 1.6P_L$ (LRFD)

$P_u = 1.2 \cdot (30 \text{ kips}) + 1.6 \cdot (35 \text{ kips}) = 92 \text{ kips}$ (LRFD)

Then, the required strength, R_u using the LRFD load combinations will be set equal to the design shear force, V_u applied on the connection. Then, for the cantilever beam and connection specimen:

$R_u = V_u = P_u = 1.2P_D + 1.6P_L$ (LRFD)

A.1.2 LRFD Strength Requirements

Design is performed according to the LRFD design equation below.

$R_u \leq \phi R_n$

where:

- R_u = required strength using LRFD load combinations (calculated in Section A.1.1)
- R_n = nominal strength (e.g., V_n for shear design, or M_n for flexural design)
- ϕ = resistance factor (or strength reduction factor; $\phi \leq 1.0$)
- ϕR_n = design strength, reduced strength, or factored strength (e.g., ϕV_n for shear design, or ϕM_n for flexural design)

A.2 Properties of Test Specimen (Test No. 4)

For the following analysis, consider:

- t_w = web thickness of beam
- d = depth of beam
- T = clear distance between web fillets
- k = distance from outer face of flange to web tow of fillet
- F_y = specified minimum yield strength
- F_u = specified minimum tensile strength
- t_f = flange thickness of column
- L= length of the double angles

- Beam: W24x68
  - Specified minimum yield strength, F_y = 50ksi ASTM A992
  - Specified minimum tensile strength, F_u = 65 ksi ASTM A992
  - Web thickness of beam, t_w = 0.415 in.
  - Depth of beam, d = 23.7 in.
  - Clear distance between web fillets, T = 20.75 in.
  - Distance from outer face of flange to web tow of fillet, k = $1\frac{1}{16}$ in. (Table 1-1, AISC Manual, 2017)

- Column: W10x77
  - Specified minimum yield strength, F_y = 50ksi ASTM A992
  - Specified minimum tensile strength, F_u = 65 ksi ASTM A992
  - Flange thickness of column, t_f = 0.87 in. (Table 1-1, AISC Manual)

- Double angles: 4 x 3½ x 3/8
  ASTM A36 F_y = 36 ksi F_u = 58 ksi
  - Length of the double angles, L= 20.5 in.
- Number of bolts: seven
- Diameter of bolts: ¾ in.
- Type of bolts: A325-X (threads are excluded)
- Weld size: ¼ in. welding with E70XX electrode

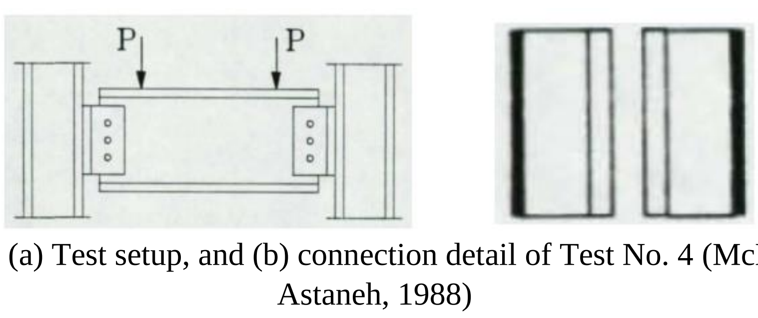


Figure A.1: (a) Test setup, and (b) connection detail of Test No. 4 (McMullin and Astaneh, 1988)

A.3 Geometric Checks

For the following analysis, consider:

- w = weld size
- w_{max}= maximum weld size
- w_{min}= minimum weld size
- $L_{w=}$ weld length
- t_{ang} = angle thickness
- L_{min} = minimum angle length
- L_{max} = maximum angle length
- L_{emin} = minimum edge distance
- L_{ev} = vertical edge distance of angle
- L_{eh} = horizontal edge distance of angle
- L_{svmin} = minimum vertical center to center bolt distance
- L_{sv} = vertical center to center bolt distance
- L_{smax} = maximum center to center bolt distance
- L_{wmin} = minimum weld length
- d = bolt diameter
- T= clear distance between web fillets of the beam

A.3.1 Angle Thickness

$w < t_{ang}$ - 1/16 in. (Section J2.2b, AISC 360-16)

- $t_{ang} \geq w$ + 1/16 in.
- $t_{ang} \geq$ 1/4 in. + 1/16 in.
- t_{ang} = 3/8 in. $\geq$ 5/16 in. **OK**

where
w = 1/4-in. – weld size
t_{ang} = 3/8 in. – angle thickness

A.3.2 Angle Length

$L_{min} = T/2$ (Page 10-9, AISC Manual)

- L_{min} = (20.75 in.)/2 = 10.38 in.
- L = 20.5 in. $\geq L_{min}$ = (20.75 in.)/2 = 10.375 in. **OK**

$L_{max} = T$ (Page 10-9, AISC Manual)

- L_{max} = 20.75 in.
- L = 20.5 in. $\leq L_{max}$ = 20.75 in. **OK**

where

T = 20.75 in. – clear distance between web fillets of the beam
L_{min} = 10.38 in. – minimum angle length
L_{max} = 20.75 in. – maximum angle length

A.3.3 Angle Edge Distance Check

L_{emin} = 1 in. for bolt diameter of ¾ in. (Table J3.4, AISC 360-16)

- L_{ev} = 1.25 in. > L_{emin} = 1 in. **OK**
- L_{eh} = 1.25 in. > L_{emin} = 1 in. **OK**

where

- L_{emin} = 1 in. – minimum edge distance
- L_{ev} = 1.25 in. – vertical edge distance of angle
- L_{eh} = 1.25 in. – horizontal edge distance of angle

A.3.4 Angle Bolt Spacing Check

$L_{svmin} = (8/3)\cdot d$ (Section J3.3, AISC 360-16)

- L_{svmin} = (8/3)·(¾ in.)
- L_{svmin} = 2 in.
- L_{sv} = 3 in. > L_{svmin} = 2 in. **OK**

L_{smax} = minimum of (24·t_{ang}) and 12 in. (Section J3.5, AISC 360-16)

- L_{smax} = 24·(0.375 in.)
- L_{smax} = 9 in.
- L_{sv} = 3 in. < L_{smax} = 9 in. **OK**

where:
L_{svmin} = 2 in. - minimum vertical center to center bolt distance
L_{sv} = 3 in. - vertical center to center bolt distance
L_{smax} = 9 in. - maximum center to center bolt distance
d = ¾ in. - bolt diameter

A.3.5 Weld Size Check

$w_{max} = t_{ang}$ - 1/16 in. (Section J2.2b, AISC 360-16)

- w_{max} = 0.375 in. - 1/16 in.
- w_{max} = 0.31 in.
- w = 0.25 in. < w_{max} = 0.31 in. **OK**

w_{min} = 0.19 in. for t_{ang} = 3/8 in. (Table J2.4, AISC 360-16)

- w = 0.25 in. > w_{min} = 0.19 in. **OK**

where:

- w = ¼ in. – weld size
- w_{max} = 0.31 in. – maximum weld size
- w_{min} = 0.19 in. – minimum weld size
- t_{ang} = 3/8 in. – angle thickness

A.3.6 Weld Length Check

$L_{wmin} = 4 \cdot w$ (Section J2.2b, AISC 360-16)

- $L_{wmin} = 4 \cdot 0.25$ in.
- L_{wmin} = 1 in.
- L_w = 20.5 in. > L_{wmin} = 1 in.

where:

- L_{wmin} = 1 in. – minimum weld length
- L_w = 20.5 in. – weld length

A.4 LRFD Design Checks

In this section, design of welds, angles, bolts, and beam are checked. The LRFD design strength ϕR_n is calculated following the requirements of AISC Construction Manual (2017) and AISC 360-16. The calculated design strength ϕR_n is then compared with the design demand R_u calculated from structural analysis using the factored external loads.

For bolts, shear, tensile bearing and tearout failure limit states are checked in Sections A.4.1 through A.4.4.

For angles, bearing, tearout, rupture, block shear, and yielding limit states are checked (Section A.4.5 through A.4.9).

For beam, yielding is checked (Section A.4.10).

For welding, rupture and available strength are checked (Section A.4.11 and A.4.13).

For the following analysis, consider:

- ϕR_n = Shear strength of one bolt
- ϕR_n =Total shear strength of seven bolts double angle
- F_{nv} = nominal shear strength of fasteners (Table J3.2, AISC 360-16)
- A_b = nominal bolt area
- ϕ = strength factor (LRFD)

- F_{nt} = nominal tensile strength of fasteners (Table J3.2, AISC 360-16)
- A_b = nominal bolt area
- d = nominal bolt diameter
- t = thickness of the beam
- F_u = specified minimum tensile strength of beam
- l_c = clear distance
- d_h = nominal hole dimension
- L_{sv} = vertical center to center bolt distance
- L_{ev} = vertical edge distance of angle
- L_{csv} = clear vertical distance between bolts
- L_{cev} = clear vertical edge distance
- F_u = specified minimum tensile strength of angle
- A_{nv} = net area subject to shear, in.^{2}
- L_h = hole dimension for tension and shear net area
- L_e = effective length
- n = number of the bolts
- U_{bs} = Stress index for uniform tension stress
- A_{nt} = net area subject to tension
- A_{gv} = gross area subject to shear
- A_{nv} = net area subject to shear
- L_{ev} = vertical edge distance of angle
- L_{eh} = horizontal edge distance of angle
- D = number of sixteenths-of-an-inch in the weld size
- L = length of the angles
- e = width of the leg of the angle
- F_{EXX} = electrode classification number

A.4.1 Bolt Shear Check

$\phi R_n = \phi \cdot F_n \cdot A_b$ (Eq. J3-1, AISC 360-16)

- Shear strength of one bolt:
- $\phi R_n = 0.75 \cdot 68 \cdot 0.442$ **= 22.54 kips/bolt**
- Total shear strength of seven bolts double angle:
- ϕR_n = 2 x 7 x 22.54 kips = **315.6 kips > 92 kips** **OK**

where:

- F_{nv} = 68 ksi – nominal shear strength of fasteners (Table J3.2, AISC 360-16)
- A_b = 0.442 in.^{2} – nominal bolt area
- ϕ = 0.75 – strength factor (LRFD)

A.4.2 Bolt Tensile Check

$\phi R_n = \phi \cdot F_n \cdot A_b$ (Eq. J3-1, AISC 360-16)

- Tension strength of one bolt:
- $\phi R_n = 0.75 \cdot 90 \cdot 0.442 =$ **29.84 kips/bolt**
- Total tension strength of seven bolts double angle:
- $\phi R_n = 2 \text{ x } 7 \text{ x } 29.84$ kips = **417.8 kips**

where:

- $F_{nt} = 90$ ksi – nominal tensile strength of fasteners (Table J3.2, AISC 360-16)
- $A_b = 0.442$ in.^{2} – nominal bolt area
- $\phi = 0.75$ – strength factor (LRFD)

A.4.3 Bolt Bearing on Beam

$r_n = 2.4 \cdot d \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6a)

- $r_n = 2.4 \cdot (3/4 \text{ in.}) \cdot (0.415 \text{ in.}) \cdot (65 \text{ ksi})$
- $r_n = 48.56$ kips/bolt
- $\phi R_n = 0.75 \cdot (7 \text{ bolts}) \cdot 48.56$ kips
- $\phi R_n =$ **254.9 kips > 92 kips** **OK**

where:

- $d = 3/4$ in. – nominal bolt diameter
- $t = 0.415$ in. – thickness of the beam
- $F_u = 65$ ksi – specified minimum tensile strength of beam
- $\phi = 0.75$ – strength factor (LRFD)

A.4.4 Bolt Tearout on Beam

The available tearout strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 1.2 \cdot l_c \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6c)

- $L_{csv} = 3 \text{ in.} - 13/16 \text{ in.} = 2.19$ in.
- $L_{cev} = [23.7 \text{ in.} - 6 \cdot (3 \text{ in.}) - (13/16) \text{ in.}]/2 = 2.44$ in.
- $R_{n\text{-}span} = 1.2 \cdot (2.19 \text{ in.}) \cdot (0.415 \text{ in.}) \cdot (65 \text{ ksi})$
- $R_{n\text{-}span} = 70.89$ kips/bolt
- $R_{n\text{-}end} = 1.2 \cdot (2.44 \text{ in.}) \cdot (0.415 \text{ in.}) \cdot (65 \text{ ksi})$
- $R_{n\text{-}end} = 78.98$ kips/bolt
- $\phi R_n = 0.75 \cdot [6 \cdot (70.89 \text{ kips}) + (78.98 \text{ kips})] =$ **378.2 kips > 92 kips OK**

where:

- l_c : clear distance
- d_h = 13/16 in. – nominal hole dimension (3/4-in.-diameter bolt)
- L_{sv} = 3 in. – vertical center to center bolt distance
- L_{ev} = 1.25 in. – vertical edge distance of angle
- L_{csv} = 2.19 in. – clear vertical distance between bolts
- L_{cev} = 2.44 in. – clear vertical edge distance
- ϕ = 0.75 – strength factor (LRFD)

A.4.5 Bolt Bearing on Angles

The nominal bearing strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 2.4 \cdot d \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6a)

- R_n =2.4·(3/4 in.)·(3/8 in.)·(58 ksi)
- R_n = 39.15 kips/bolt
- ϕR_n = 0.75·(2 angles)·(7 bolts)·39.15 kips
- ϕR_n = **411.1 kips > 92 kips** **OK**

where:

- d = 3/4 in. – nominal bolt diameter
- t = 3/8 in. – thickness of angle
- F_u = 58 ksi – specified minimum tensile strength of angle
- ϕ = 0.75 – strength factor (LRFD)

A.4.6 Bolt Tearout on Angles

The available tearout strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 1.2 \cdot l_c \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6c)

- $L_{csv} = L_{sv} - d_h$
- L_{csv} = 3 in. - 13/16 in. = 2.19 in.
- $R_{n\text{-}span}$ = 1.2·(2.19 in.)·(3/8 in.)·(58 ksi)
- $R_{n\text{-}span}$ = 57.16 kips/bolt
- $L_{cev} = L_{ev} - d_h/2$
- L_{cev} = 1.25 in. - (13/16)/2 in. = 0.84 in.
- $R_{n\text{-}end}$ = 1.2·(0.84 in.)·(3/8 in.)·(58 ksi)
- $R_{n\text{-}end}$ = 21.92 kips/bolt
- ϕR_n = 0.75·(2 angles)·[6·(57.16 kips) + (21.92 kips)] = **547.3 kips > 92 kips** **OK**

where:

- l_c : clear distance
- d_h = 13/16 in. - nominal hole dimension (3/4-in.-diameter bolt)
- L_{sv} = 3 in. - vertical center to center bolt distance
- L_{ev} = 1.25 in. - vertical edge distance of angle
- L_{csv} = 2.19 in. - clear vertical distance between bolts
- L_{cev} = 0.84 in. - clear vertical edge distance
- ϕ = 0.75 - strength factor (LRFD)

A.4.7 Shear Rupture on Angles (Beam Side)

$R_n = 0.60 \cdot F_u \cdot A_{nv}$ (AISC 360-16, Eq. J4-4)

- $L_h = d_h + 1/16$ (Section B4-3b, AISC Manual)
- L_h = 0.813 + 1/16 = 0.875 in.
- $L_e = L - n \cdot L_h$
- L_e = 20.5 - 7·0.875 = 14.375 in.
- $A_{nv} = L_e \cdot t_p$
- A_{nv} = (14.375 in.)·(0.375 in.) = 5.39 in.^{2}
- ϕR_n = (0.75)·(2 angles)·0.60·(58 kips)·(5.39 in.^{2})
- ϕR_n = **281.4 kips > 92 kips** **OK**

where:

- A_{nv} = 5.39 in.^{2} - net area subject to shear, in.^{2}
- L_h = 0.875 in. - hole dimension for tension and shear net area
- L_e = 14.375 in. - effective length
- ϕ = 0.75 - strength factor (LRFD)
- n = 7 - number of the bolts

A.4.8 Block Shear on Angles (Beam Side)

$R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$ (AISC 360-16, Eq. J4-5)

- $A_{gv} = (L - L_{ev}) \cdot (t_p)$
- A_{gv} = (2 angles)·(20.5 in. - 1.25 in.)·(0.375 in.) = 14.44 in.^{2}
- $A_{nv} = A_{gv}$ - (2 angles)·(n-0.5)·(d_h + 1/16 in.)·(t_p)
- A_{nv} = 14.44 in.^{2} - (2 angles)·(7-0.5)·(0.813 + 1/16 in.)·(0.375 in.)
- A_{nv} = 10.17 in.^{2}
- A_{nt} = (2 angles)·[L_{eh} - 0.5·(d_h + 1/16 in.)]·(t_p)
- A_{nt} = (2 angles)·[1.25 in. - 0.5·(0.813 in. + 1/16 in.)]·(0.375 in.)
- A_{nt} = 0.61 in.^{2}
- U_{bs} = 1 (as the tension stress is uniform)

- $R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$
- $R_n = 0.60 \cdot (58 \text{ kips/in.}^2) \cdot (10.17 \text{ in.}^2) + 1 \cdot (58 \text{ kips/in.}^2) \cdot (0.61 \text{ in.}^2) \leq 0.60 \cdot (36 \text{ kips/in.}^2) \cdot (14.44 \text{ in.}^2) + 1 \cdot (58 \text{ kips/in.}^2) \cdot (0.61 \text{ in.}^2)$
- R_n = 389.3 kips ≤ **347.3 kips**
- ϕR_n = 0.75·(347.3 kips)
- **ϕR_n = 260.5 kips > 92 kips** **OK**

where:

- $U_{bs} = 1$ – Stress index for uniform tension stress
- $A_{nt} = 0.61$ in.^{2} – net area subject to tension
- $A_{gv} = 14.44$ in.^{2} – gross area subject to shear
- $A_{nv} = 10.17$ in.^{2} – net area subject to shear
- $L_{ev} = 1.25$ in. – vertical edge distance of angle
- $L_{eh} = 1.25$ in. – horizontal edge distance of angle
- $\phi = 0.75$ – strength factor (LRFD)

A.4.9 Shear Yielding on Angles

$R_n = 0.60 \cdot F_y \cdot A_{gv}$ $\phi = 1.00$ (LRFD) (AISC 360-16, Eq. J4-3)

- A_{gv} = (20.5 in.)·(0.375 in.) = 7.69 in.^{2}
- ϕR_n = 1.00·(2 angles) 0.60·(36 kips)·(7.69 in.^{2})
- **ϕR_n = 332.21 kips > 92 kips** **OK**

where:

- $A_{gv} = 7.69$ in.^{2} - gross area subject to shear
- $\phi = 1.00$ - strength factor (LRFD)

A.4.10 Shear Yielding on Beam

$R_n = 0.60 \cdot F_y \cdot A_{gv}$ (AISC 360-16, Eq. J4-3)

- A_{gv} = (23.7 in.)·(0.415 in.) = 9.84 in.^{2}
- ϕR_n = 1.00·0.60·(50 kips)·(9.84 in.^{2})
- **ϕR_n = 295.2 kips > 92 kips** **OK**

where:

- $A_{gv} = 9.84$ in.^{2} – gross area subject to shear, in.^{2}
- $\phi = 1.00$ – strength factor (LRFD)

A.4.11 Weld Rupture on Angles (Support side)

$t_{min} = [0.6 \cdot (F_{EXX}) \cdot (2^{1/2}/2) \cdot (D/16)] / [0.6 \cdot (F_u)]$ (Page 9-5, AISC Manual)

- t_{min} = [0.6·(70 ksi)·($2^{1/2}$/2)·(4/16)] / [0.6·(58)]
- t_{min} = **0.21 in. < t_f = 0.87 in.** **OK**

where:

- D = 4 – number of sixteenths-of-an-inch in the weld size
- F_u = 58 ksi – specified minimum tensile strength of the connecting element
- F_{EXX} = 70 ksi – electrode classification number

A.4.12 Weld Capacity

$\phi R_n = (0.75)\cdot 2\,[(F_{EXX})\cdot 0.6\cdot(D/16)\cdot L\cdot(2^{1/2}/2) / (1 + 12.96\cdot e^2/L^2)^{1/2}]$ (Page 10-11, AISC Manual)

- $\phi R_n = 2\,[(1.392\cdot D\cdot L) / (1 + 12.96\cdot e^2/L^2)^{1/2}]$
- $\phi R_n = 2\,[(1.392\cdot 4\cdot 20.5) / (1 + 12.96\cdot 4^2/20.5^2)^{1/2}]$
- ϕR_n = **186.8 kips > 92 kips** **OK**

where:

- D = 4 – number of sixteenths-of-an-inch in the weld size
- L = 20.5 in. – length of the angles
- e = 4 – width of the leg of the angle
- F_{EXX} = 70 ksi – filler metal electrode classification strength

A.4.13 Weld Capacity (no eccentricity)

$R_n = F_{nw}\cdot A_{we}$ (AISC 360-16, Eq. J4-2)

- $F_{nw} = 0.60\cdot(F_{EXX})\cdot(1.0+0.50\sin(\theta)^{1.5}) = 0.60\cdot 70$ ksi = 42 ksi (It is assumed that $\theta = 0$)
- $A_{we} = (D/16)\cdot L\cdot(2^{1/2}/2) = (4/16)\cdot(20.5\text{ in.})\cdot(2^{1/2}/2) = 3.62\text{ in.}^2$
- $\phi R_n = (0.75)\cdot(42\text{ ksi})\cdot(3.62\text{ in.}^2) = 114.0$ kips (for one angle)
- ϕR_n = (2 angles)·(114.0 kips) = **228.0 kips > 92 kips** **OK** (for double angle)

where:

- D = 4 – number of sixteenths-of-an-inch in the weld size
- L = 20.5 in. – length of the angles
- e = 4 – width of the leg of the angle
- F_{EXX} = 70 ksi – filler metal electrode classification strength
- F_{nw} = 42 ksi – nominal stress of the weld metal
- A_{we} = 3.62 in.^{2} – effective area of the weld
- θ = 0 – angle between the line of action of the required force and the weld longitudinal axis, degree

Appendix B. ASD Strength Calculations for Connection Specimen Test No. 4

B.1 Connection Properties, Loading and Assumptions

The properties of connection specimen Test No. 4 and geometric checks according to AISC code are provided in Appendix A. Appendix B includes only the strength calculations and checks following the requirements of AISC Allowable Strength Design (ASD).

It is assumed that the following dead and live loads are applied at the tip of the cantilever specimen Test No. 4 (see Figure 1.1 and Figure A.1 in Appendix A).

P_D = 30 kips (dead load)

P_L = 35 kips (live load)

B.1.1 ASD Load Demand or Design Loads

According the ASD procedure (AISC 360-16 or AISC Manual 2017), ASD design load, factored load or demand, P_a under gravity loads is:

$P_a = P_D + P_L$ (ASD)

P_a = 30 kips + 35 kips = 65 kips (ASD)

Then, the required strength, R_a using the ASD load combinations will be set equal to the design shear force, V_a applied on the connection. Then, for the cantilever beam and connection specimen:

$R_a = V_a = P_a = P_D + P_L$ (ASD)

B.1.2 ASD Strength Requirements

Design is performed according to the ASD design equation below.

$R_a \leq R_n/\Omega$

where:

- R_a = required strength using ASD load combinations (calculated in Section B.1.1)
- R_a = nominal strength (e.g., V_a for shear design, or M_a for flexural design)
- Ω = safety factor ($\Omega \geq 1.0$)
- R_n/Ω = design strength (e.g., V_n/Ω for shear design, or M_n/Ω for flexural design)

B.2 ASD Design Checks

B.2.1 Bolt Shear Check

$R_n/\Omega = F_n \cdot A_b/\Omega$ (Eq. J3-1, AISC 360-16)

- Shear strength of one bolt:
- $R_n/\Omega = 68 \cdot 0.442 / 2 =$ **15.03 kips/bolt**
- Total shear strength of seven bolts double angle:
- $R_n/\Omega = 2 \cdot 7 \cdot 15.03$ kips = **210.42 kips > 65 kips** **OK**

where:

- F_{nv} = 68 ksi – nominal shear strength of fasteners (Table J3.2, AISC 360-16)
- A_b = 0.442 in.^{2} – nominal bolt area
- Ω = 2 – safety factor (ASD)

B.2.2 Bolt Tensile Check

$R_n/\Omega = F_n \cdot A_b/\Omega$ (Eq. J3-1, AISC 360-16)

- Tension strength of one bolt:
- $R_n/\Omega = 90 \cdot 0.442 / 2 =$ **19.89 kips/bolt**
- Total tension strength of seven bolts double angle:
- $R_n/\Omega = 2 \cdot 7 \cdot 19.89$ kips = **278.46 kips**

where:

- F_{nt} = 90 ksi – nominal tensile strength of fasteners (Table J3.2, AISC 360-16)
- A_b = 0.442 in.^{2} – nominal bolt area
- Ω = 2 – safety factor (ASD)

B.2.3 Bolt Bearing on Beam

$R_n/\Omega = 2.4 \cdot d \cdot t \cdot F_u/\Omega$ (AISC 360-16, Eq. J3-6a)

- $R_n/\Omega = 2.4 \cdot (3/4 \text{ in.}) \cdot (0.415 \text{ in.}) \cdot (65 \text{ ksi}) / 2$
- R_n/Ω = 24.28 kips/bolt
- R_n/Ω = (7 bolts)·24.28 kips
- R_n/Ω = **169.96 kips > 65 kips** **OK**

where:

- d = 3/4 in. – nominal bolt diameter
- t = 0.415 in. – thickness of the beam
- F_u = 65 ksi – specified minimum tensile strength of beam
- Ω = 2 – safety factor (ASD)

B.2.4 Bolt Tearout on Beam

The available tearout strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n/\Omega = 1.2 \cdot l_c \cdot t \cdot F_u/\Omega$ (AISC 360-16, Eq. J3-6c)

- L_{csv} = 3 in. - 13/16 in. = 2.19 in.
- L_{cev} = [23.7 in. - 6·(3 in.) - (13/16) in.]/2 = 2.44 in.
- $R_{n\text{-}span}$ = 1.2·(2.19 in.)·(0.415 in.)·(65 ksi)
- $R_{n\text{-}span}$ = 70.89 kips/bolt
- $R_{n\text{-}end}$ = 1.2·(2.44 in.)·(0.415 in.)·(65 ksi)
- $R_{n\text{-}end}$ = 78.98 kips/bolt
- R_n/Ω = [6·(70.89 kips) + (78.98 kips)] / 2 = **252.16 kips > 65 kips** **OK**

where:

- l_c : clear distance
- d_h = 13/16 in. – nominal hole dimension (3/4-in.-diameter bolt)
- L_{sv} = 3 in. – vertical center to center bolt distance
- L_{ev} = 1.25 in. – vertical edge distance of angle
- L_{csv} = 2.19 in. – clear vertical distance between bolts
- L_{cev} = 2.44 in. – clear vertical edge distance
- Ω = 2 – safety factor (ASD)

B.2.5 Bolt Bearing on Angles

The nominal bearing strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n/\Omega = 2.4 \cdot d \cdot t \cdot F_u/\Omega$ (AISC 360-16, Eq. J3-6a)

- R_n/Ω =2.4·(3/4 in.)·(3/8 in.)·(58 ksi) / 2
- R_n/Ω = 19.58 kips/bolt
- R_n/Ω = (2 angles)·(7 bolts)·19.58 kips
- R_n/Ω = **274.12 kips > 65 kips** **OK**

where:

- d = 3/4 in. – nominal bolt diameter
- t = 3/8 in. – thickness of angle
- F_u = 58 ksi – specified minimum tensile strength of angle
- Ω = 2 – safety factor (ASD)

B.2.6 Bolt Tearout on Angles

The available tearout strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n/\Omega = 1.2 \cdot l_c \cdot t \cdot F_u/\Omega$ (AISC 360-16, Eq. J3-6c)

- $L_{csv} = L_{sv} - d_h$
- L_{csv} = 3 in. - 13/16 in. = 2.19 in.
- $R_{n\text{-}span}$ = 1.2·(2.19 in.)·(3/8 in.)·(58 ksi)
- $R_{n\text{-}span}$ = 57.16 kips/bolt
- $L_{cev} = L_{ev} - d_h/2$
- L_{cev} = 1.25 in. - (13/16)/2 in. = 0.84 in.
- $R_{n\text{-}end}$ = 1.2·(0.84 in.)·(3/8 in.)·(58 ksi)
- $R_{n\text{-}end}$ = 21.92 kips/bolt
- R_n/Ω = (2 angles)·[6·(57.16 kips) + (21.92 kips)] / 2 = **364.88 kip > 65 kips** **OK**

where:

- l_c : clear distance
- d_h = 13/16 in. – nominal hole dimension (3/4-in.-diameter bolt)
- L_{sv} = 3 in. – vertical center to center bolt distance
- L_{ev} = 1.25 in. – vertical edge distance of angle
- L_{csv} = 2.19 in. – clear vertical distance between bolts
- L_{cev} = 0.84 in. – clear vertical edge distance
- Ω = 2 – safety factor (ASD)

B.2.7 Shear Rupture on Angles (Beam Side)

$R_n/\Omega = 0.60 \cdot F_u \cdot A_{nv}/\Omega$ (AISC 360-16, Eq. J4-4)

- $L_h = d_h + 1/16$ (Section B4-3b, AISC Manual)
- L_h = 0.813 + 1/16 = 0.875 in.
- $L_e = L - n \cdot L_h$
- L_e = 20.5 - 7·0.875 = 14.375 in.
- $A_{nv} = L_e \cdot t_p$
- A_{nv} = (14.375 in.)·(0.375 in.) = 5.39 in.^{2}
- R_n/Ω = (2 angles)·0.60·(58 kips)·(5.39 in.^{2}) / 2
- R_n/Ω = **187.57 kips > 65 kips** **OK**

where:

- A_{nv} = 5.39 in.^{2} – net area subject to shear, in.^{2}
- L_h = 0.875 in. – hole dimension for tension and shear net area
- L_e = 14.375 in. – effective length

- $n = 7$ – number of the bolts
- $\Omega = 2$ – safety factor (ASD)

B.2.8 Block Shear on Angles (Beam Side)

$R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$ (AISC 360-16, Eq. J4-5)

- $A_{gv} = (L\text{-}L_{ev}) \cdot (t_p)$
- A_{gv} = (2 angles)·(20.5 in. - 1.25 in.)·(0.375 in.) = 14.44 in.^{2}
- $A_{nv} = A_{gv}$ - (2 angles)·(n-0.5)·(d_h + 1/16 in.)·(t_p)
- A_{nv} = 14.44 in.^{2} - (2 angles)·(7-0.5)·(0.813 + 1/16 in.)·(0.375 in.)
- A_{nv} = 10.17 in.^{2}
- A_{nt} = (2 angles)·[L_{eh} - 0.5·(d_h + 1/16 in.)]·(t_p)
- A_{nt} = (2 angles)·[1.25 in. - 0.5·(0.813 in. + 1/16 in.)]·(0.375 in.)
- A_{nt} = 0.61 in.^{2}
- $U_{bs} = 1$ (as the tension stress is uniform)
- $R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$
- R_n = 0.60·(58 kips/in.^{2})·(10.17 in.^{2}) + 1·(58 kips/in.^{2})·(0.61 in.^{2}) ≤ 0.60·(36 kips/in.^{2})·(14.44 in.^{2}) + 1·(58 kips/in.^{2})·(0.61 in.^{2})
- R_n = 389.3 kips ≤ **347.3 kips**
- R_n/Ω = (347.3 kips) / 2
- R_n/Ω **= 173.65 kips > 65 kips** **OK**

where:

- $U_{bs} = 1$ – Stress index for uniform tension stress
- A_{nt} = 0.61 in.^{2} – net area subject to tension
- A_{gv} = 14.44 in.^{2} – gross area subject to shear
- A_{nv} = 10.17 in.^{2} – net area subject to shear
- L_{ev} = 1.25 in. – vertical edge distance of angle
- L_{eh} = 1.25 in. – horizontal edge distance of angle
- $\Omega = 2$ – safety factor (ASD)

B.2.9 Shear Yielding on Angles

$R_n/\Omega = 0.60 \cdot F_y \cdot A_{gv}/\Omega$ (AISC 360-16, Eq. J4-3)

- A_{gv} = (20.5 in.)·(0.375 in.) = 7.69 in.^{2}
- R_n/Ω = (2 angles) 0.60·(36 kips)·(7.69 in.^{2}) / 1.5
- R_n/Ω = **221.47 kips > 65 kips** **OK**

where:

- A_{gv} = 7.69 in.^{2} – gross area subject to shear
- $\Omega = 1.5$ – safety factor (ASD)

B.2.10 Shear Yielding on Beam

$R_n/\Omega = 0.60 \cdot F_y \cdot A_{gv}/\Omega$ (AISC 360-16, Eq. J4-3)

- A_{gv} = (23.7 in.)·(0.415 in.) = 9.84 in.^{2}
- R_n/Ω = 0.60·(50 kips)·(9.84 in.^{2}) / 1.5
- R_n/Ω = **196.8 kips > 65 kips** **OK**

where:

- A_{gv} = 9.84 in.^{2} – gross area subject to shear, in.^{2}
- Ω = 1.5 – safety factor (ASD)

B.2.11 Welds Rupture on Angles (Support side)

$t_{min} = [0.6 \cdot (F_{EXX}) \cdot (2^{1/2}/2) \cdot (D/16)] / [0.6 \cdot (F_u)]$ (Page 9-5, AISC Manual)

- t_{min} = [0.6·(70 ksi)·($2^{1/2}$/2)·(4/16)] / [0.6·(58)]
- t_{min} = **0.21 in. < t_f = 0.87 in.** **OK**

where:

- D = 4 – number of sixteenths-of-an-inch in the weld size
- F_u = 58 ksi – specified minimum tensile strength of the connecting element
- F_{EXX} = 70 ksi – electrode classification number

B.2.12 Weld Capacity

$R_n/\Omega = 2\,[(F_{EXX}) \cdot 0.6 \cdot (D/16) \cdot L \cdot (2^{1/2}/2) / (1 + 12.96 \cdot e^2/L^2)^{1/2}]/\Omega$ (Page 10-11, AISC Manual)

- $R_n/\Omega = 2\,[70 \cdot 0.6 \cdot (D/16) \cdot L \cdot (2^{1/2}/2) / (1 + 12.96 \cdot e^2/L^2)^{1/2}] / 2$
- $R_n/\Omega = 2\,[(0.928 \cdot D \cdot L) / (1 + 12.96 \cdot e^2/L^2)^{1/2}]$
- $R_n/\Omega = 2\,[(0.928 \cdot 4 \cdot 20.5) / (1 + 12.96 \cdot 4^2/20.5^2)^{1/2}]$
- R_n/Ω = **124.5 kips > 65 kips** **OK**

where:

- D = 4 – number of sixteenths-of-an-inch in the weld size
- L = 20.5 in. – length of the angles
- e = 4 – width of the leg of the angle
- F_{EXX} = 70 ksi – filler metal electrode classification strength

B.2.13 Weld Capacity (no eccentricity)

$R_n = F_{nw} \cdot A_{we}$ (AISC 360-16, Eq. J4-2)

- $F_{nw} = 0.60 \cdot (F_{EXX}) \cdot (1.0+0.50 \sin(\theta)^{1.5}) = 0.60 \cdot 70$ ksi = 42 ksi (It is assumed that $\theta = 0$)

- $A_{we} = (D/16) \cdot L \cdot (2^{1/2}/2) = (4/16) \cdot (20.5 \text{ in.}) \cdot (2^{1/2}/2) = 3.62 \text{ in.}^2$
- $R_n/\Omega = (42 \text{ ksi}) \cdot (3.62 \text{ in.}^2) / 2 = 76.0$ kips (for one angle)
- $R_n/\Omega = (2 \text{ angles}) \cdot (76.0 \text{ kips}) =$ **152.0 kips > 65 kips** **OK** (for double angle)

where:

- $D = 4$ – number of sixteenths-of-an-inch in the weld size
- $L = 20.5$ in. – length of the angles
- $e = 4$ – width of the leg of the angle
- $F_{EXX} = 70$ ksi – filler metal electrode classification strength
- $F_{nw} = 42$ ksi – nominal stress of the weld metal
- $A_{we} = 3.62 \text{ in.}^2$ – effective area of the weld
- $\theta = 0$ – angle between the line of action of the required force and the weld longitudinal axis, degree

Appendix C. IDEA StatiCa Model

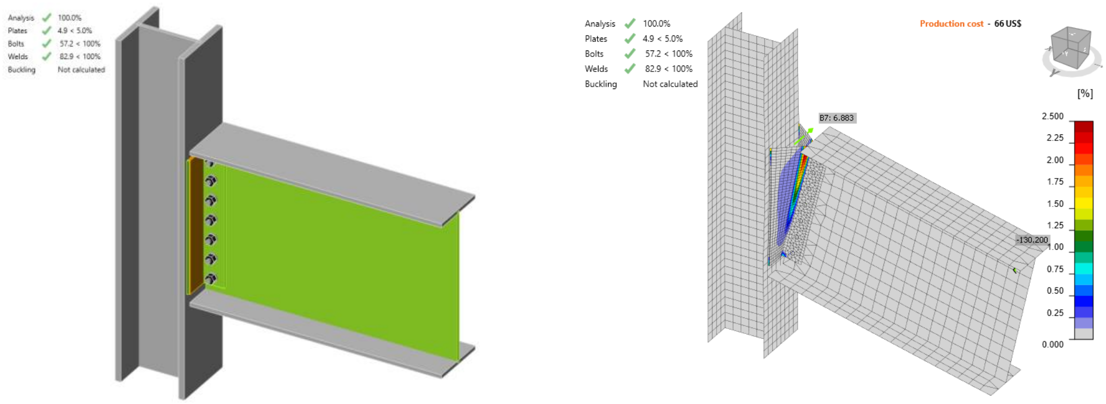


Figure C.1: IDEA StatiCa force applied on bolts in Test No. 4

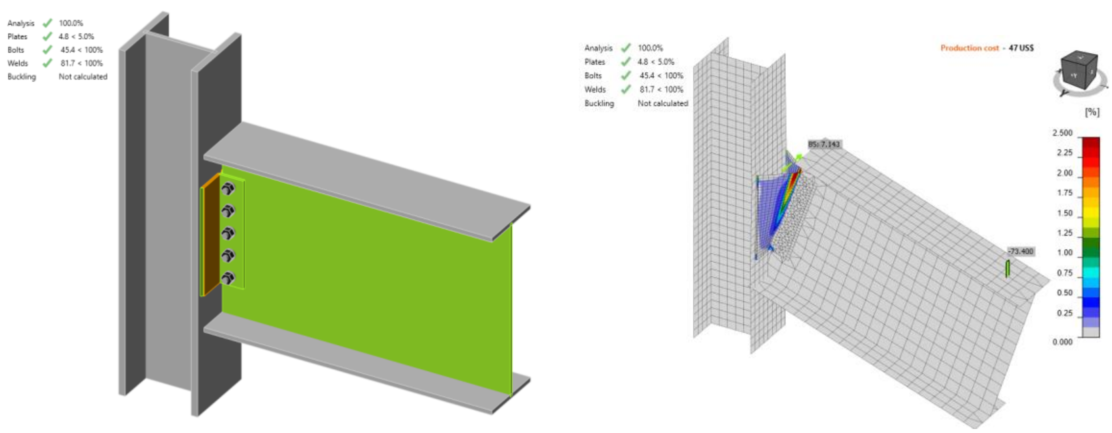


Figure C.2: IDEA StatiCa force applied on bolts in Test No. 5

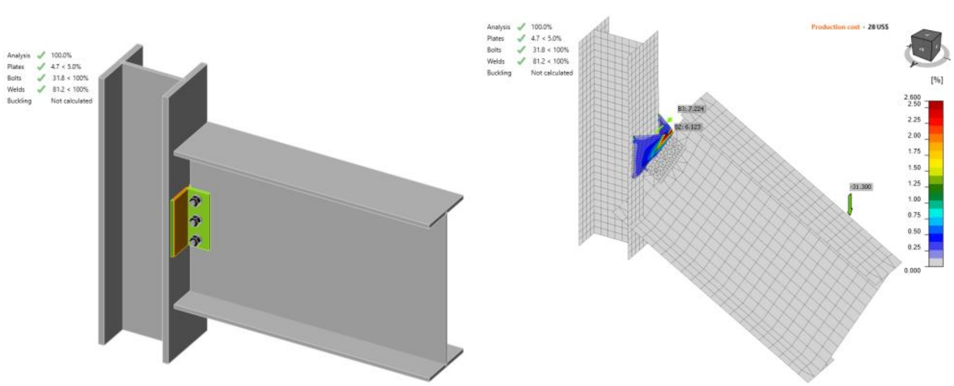


Figure C.3: IDEA StatiCa force applied on bolts in Test No. 6

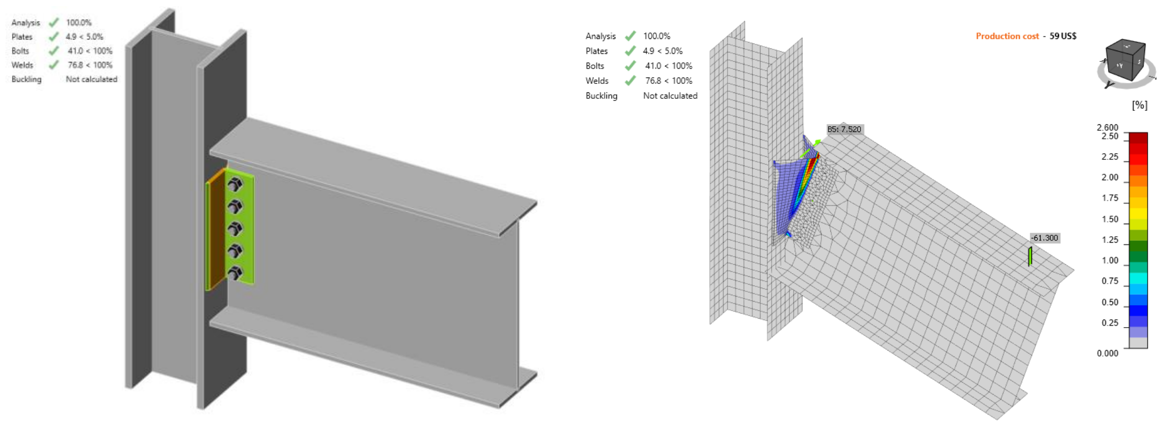


Figure C.4: IDEA StatiCa force applied on bolts in Test No. 9

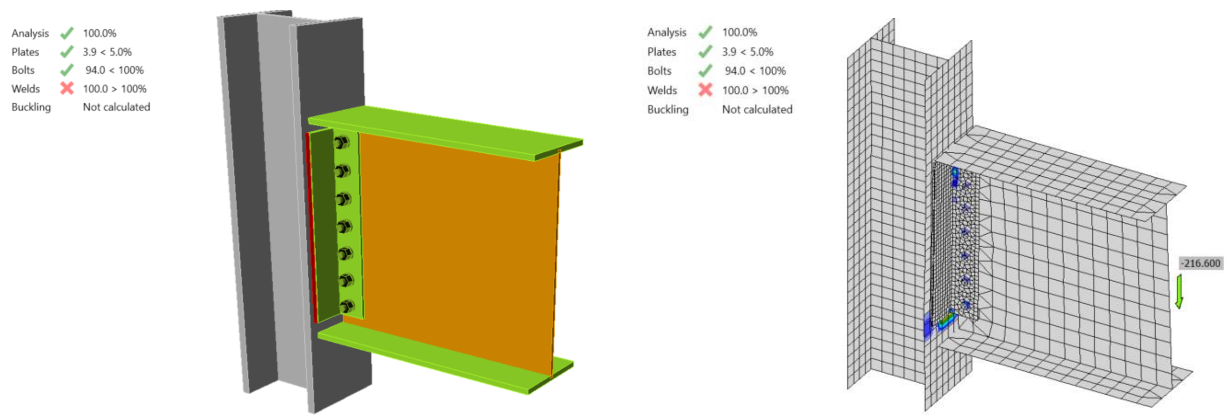


Figure C.5: IDEA StatiCa force applied on welding in Test No. 4

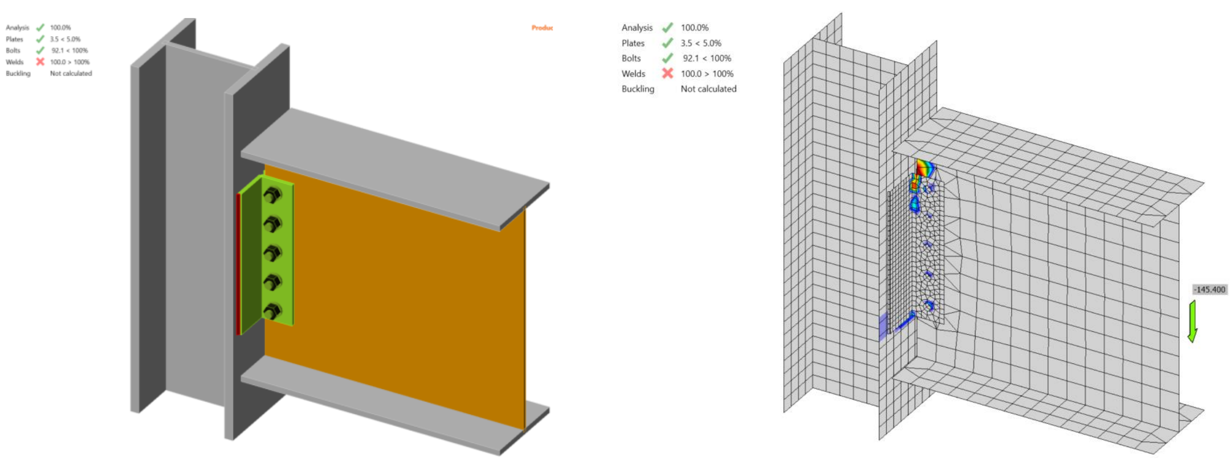


Figure C.6: IDEA StatiCa force applied on welding in Test No. 5

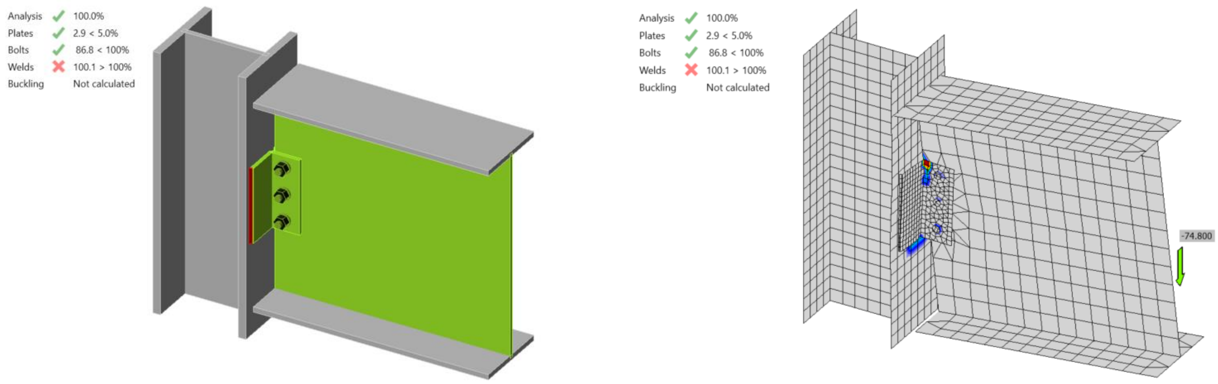


Figure C.7: IDEA StatiCa force applied on welding in Test No. 6

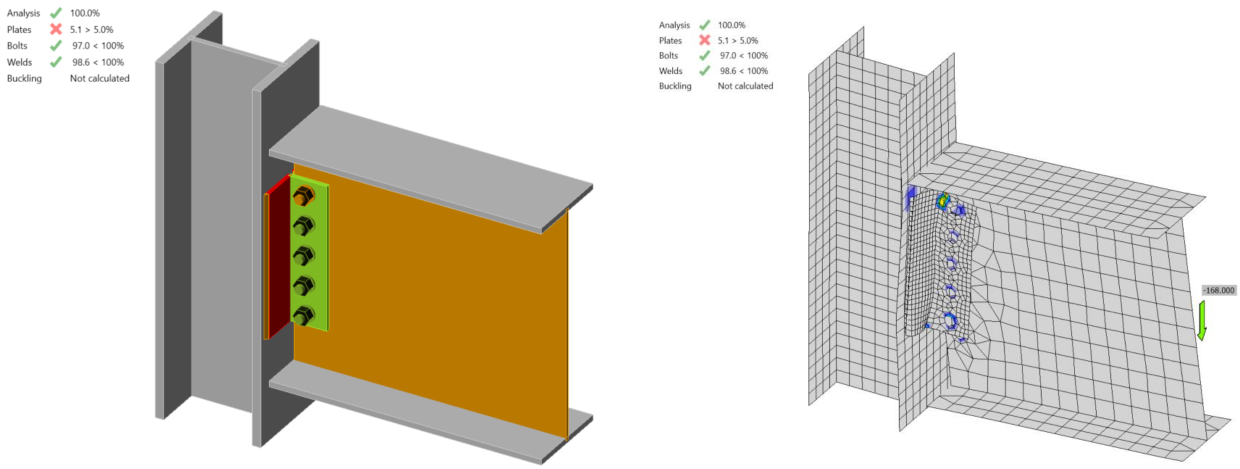


Figure C.8: IDEA StatiCa force applied on welding in Test No. 9

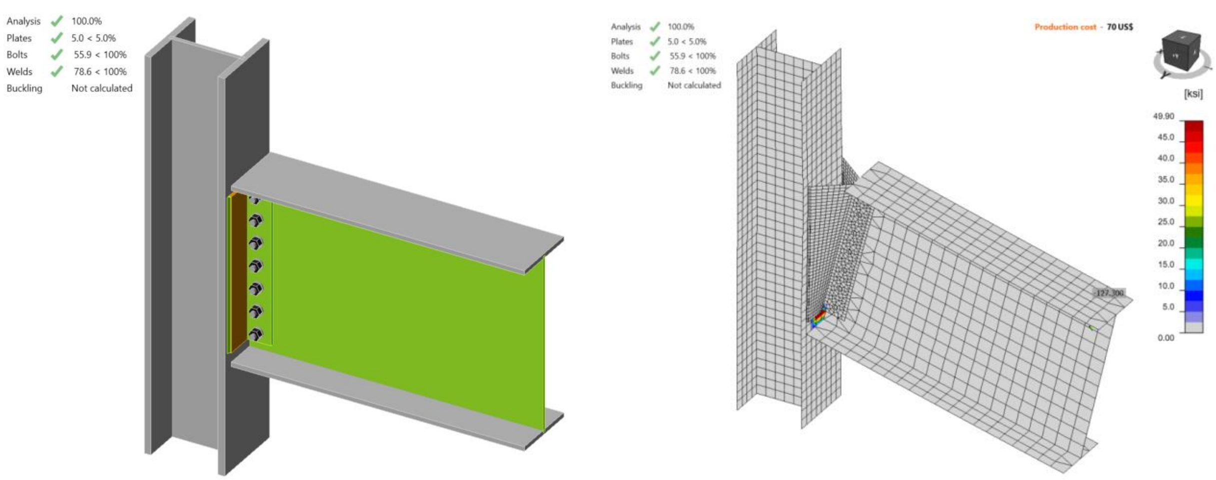


Figure C.9: IDEA StatiCa force applied on bolts in Model 1

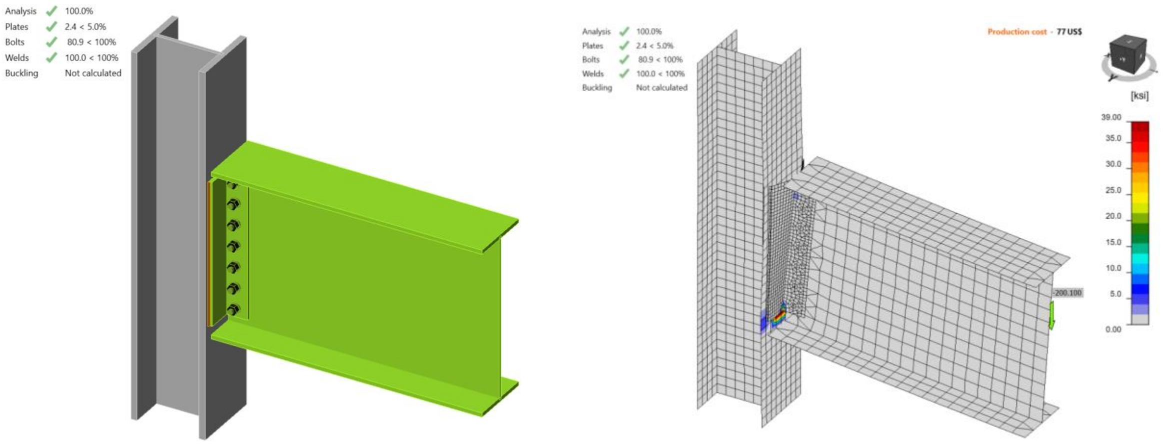


Figure C.10: IDEA StatiCa force applied on bolts in Model 2

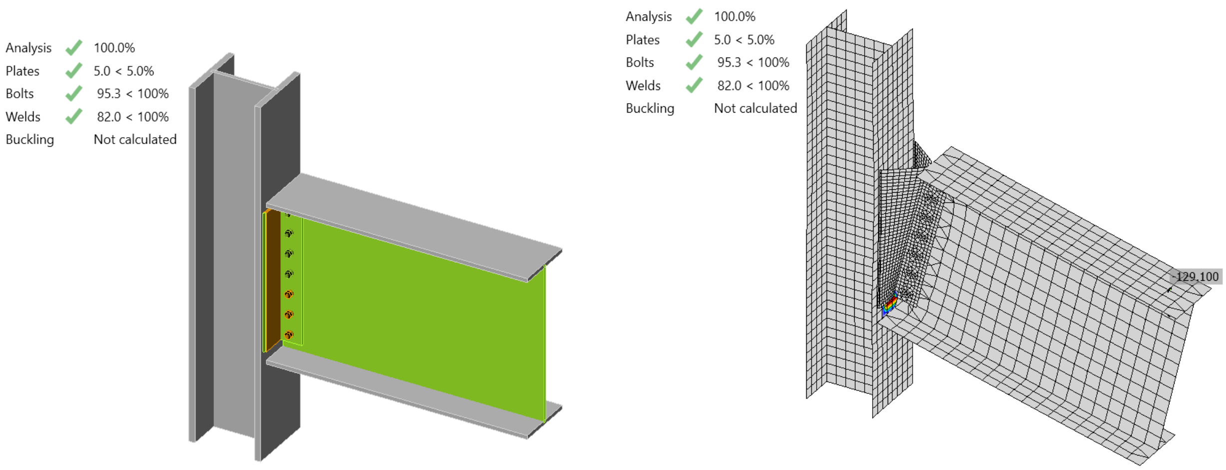


Figure C.11: IDEA StatiCa force applied on bolts in Model 3

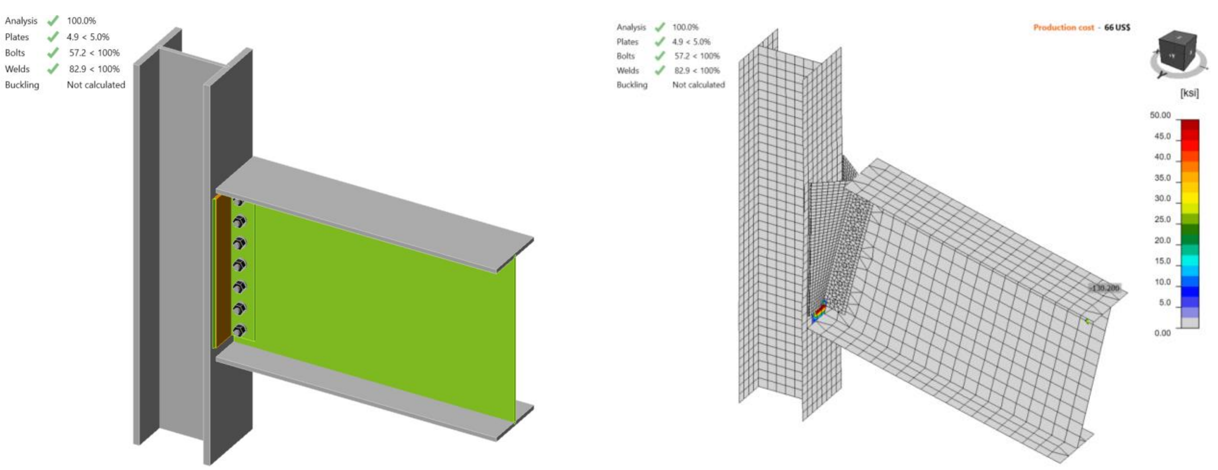


Figure C.12: IDEA StatiCa force applied on bolts in Model 4

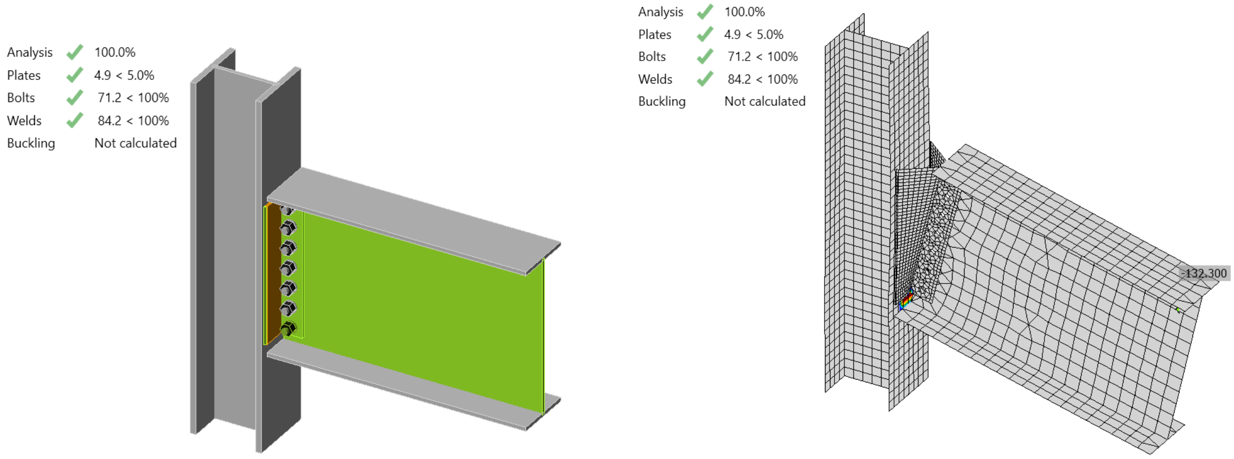


Figure C.13: IDEA StatiCa force applied on bolts in Model 5

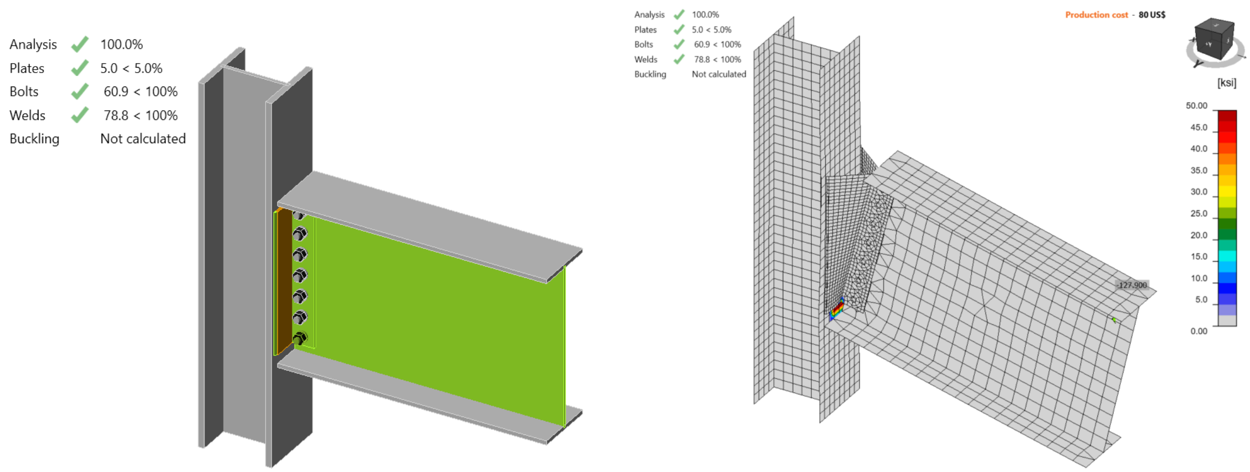


Figure C.14: IDEA StatiCa force applied on bolts in Model 6

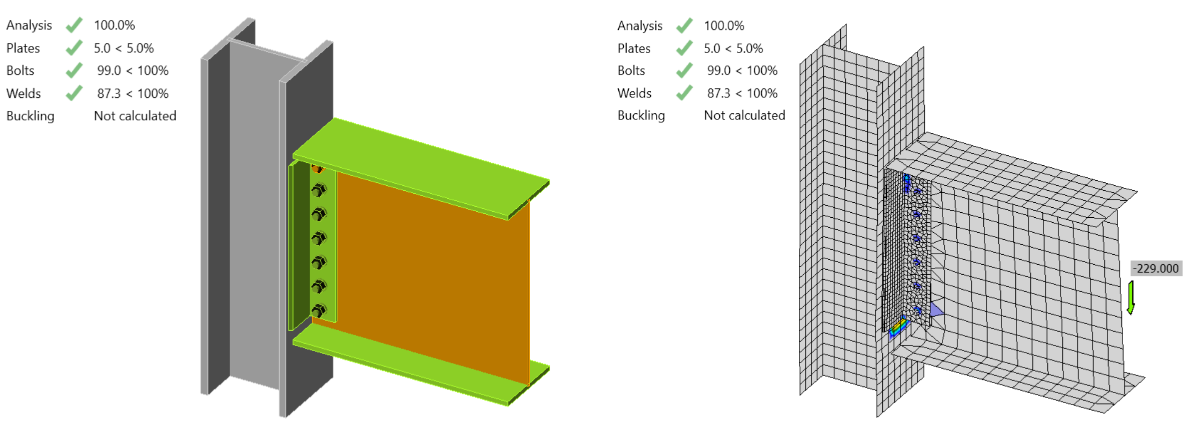


Figure C.15: IDEA StatiCa force applied on welding in Model 1

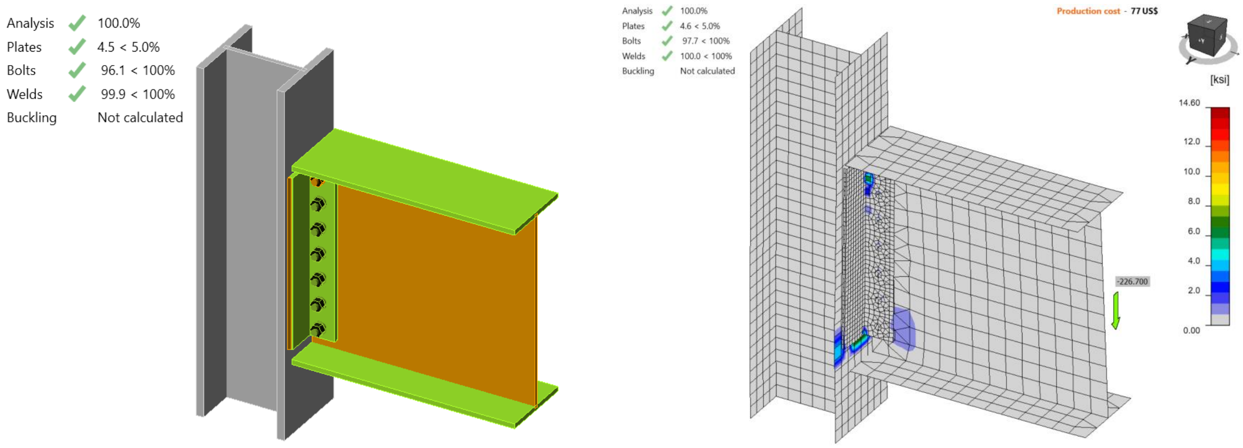


Figure C.16: IDEA StatiCa force applied on welding in Model 2

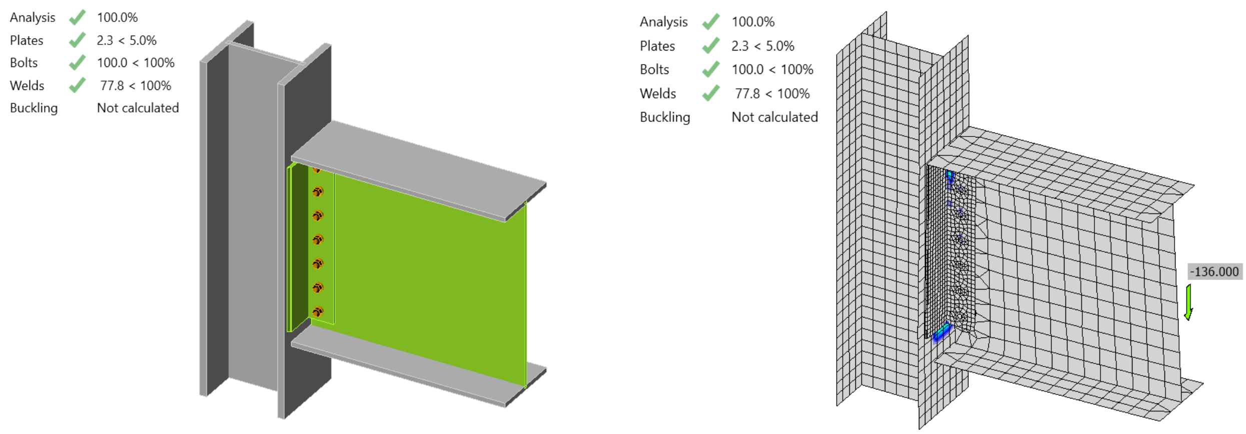


Figure C.17: IDEA StatiCa force applied on welding in Model 3

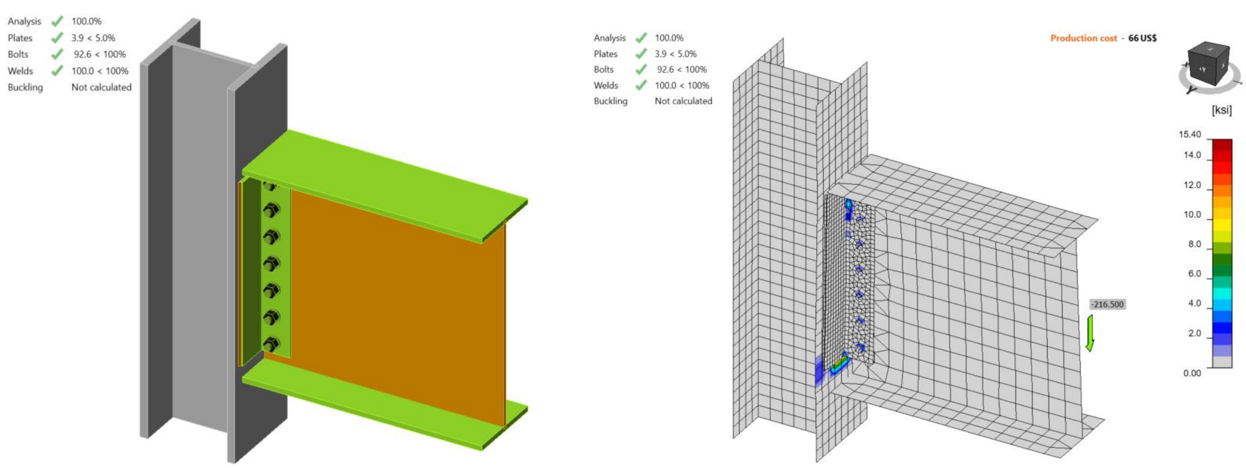


Figure C.18: IDEA StatiCa force applied on welding in Model 4

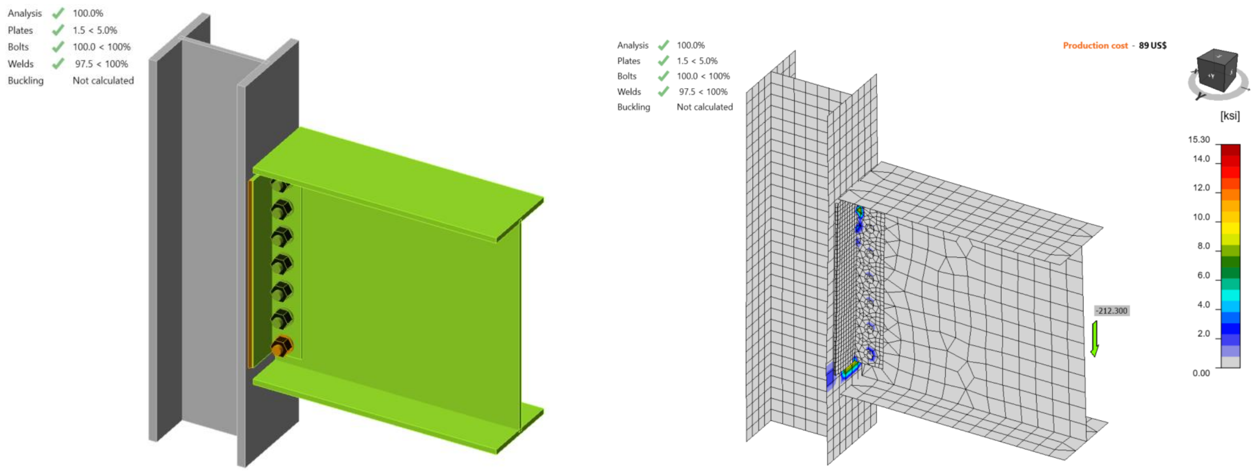


Figure C.19: IDEA StatiCa force applied on welding in Model 5

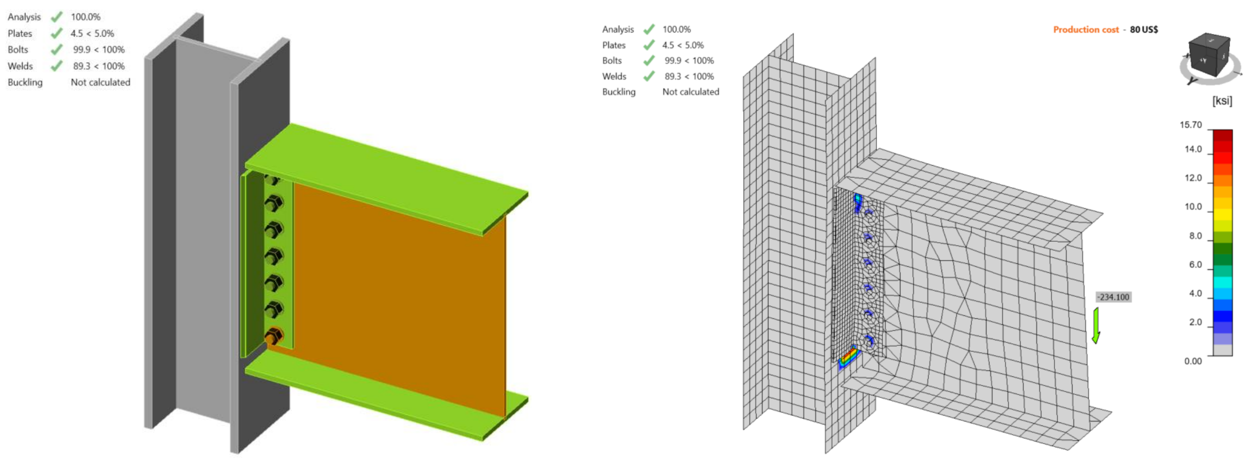


Figure C.20: IDEA StatiCa force applied on welding in Model 6

Appendix D. ABAQUS Model

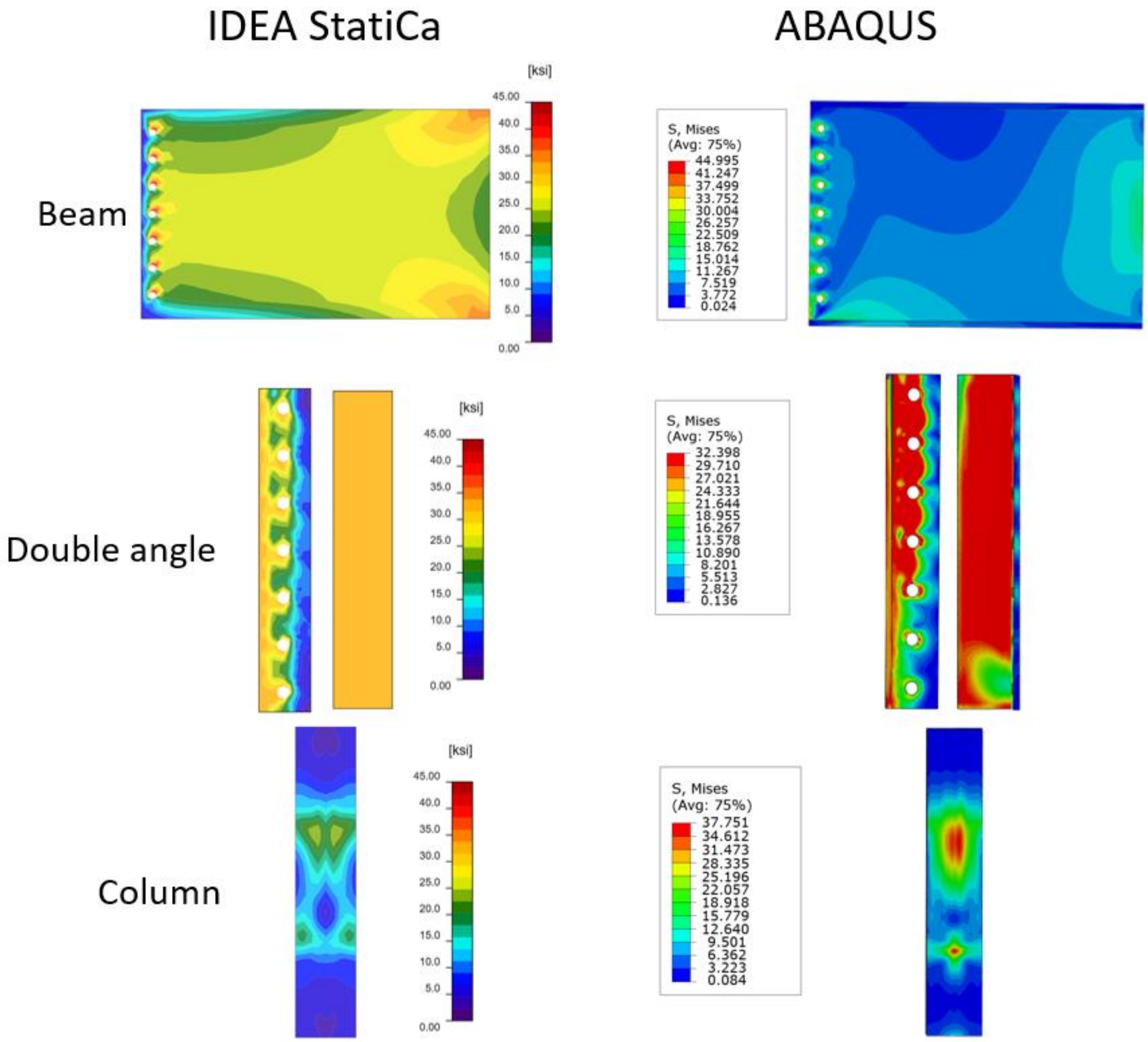


Figure D.1: Comparsion of the predicted stress between IDEA StatiCa and ABAQUS for case 1

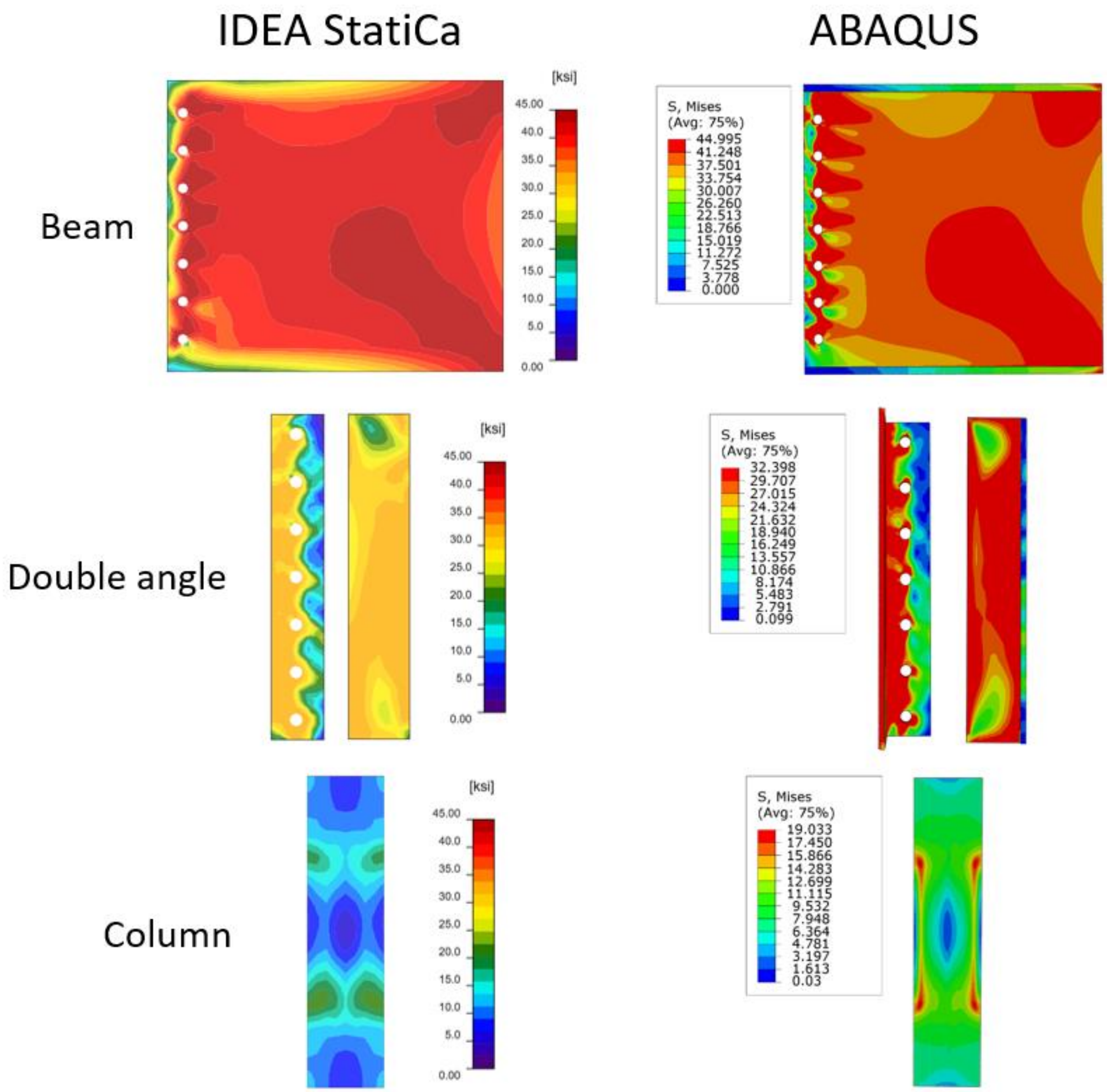


Figure D.2: Comparsion of the predicted stress between IDEA StatiCa and ABAQUS for case 2

Note 1: In ABAQUS, node set was defined in different positions (e.g., shear plains) in order to extract data (i.e., nodal forces, shear forces, etc.) from the model and calculate the bolt loads.

Note 2: In ABAQUS, to calculate the weld capacity, first the element with maximum stress was identified (critical element). Then, the resultant force and its angle with the weld longitudinal axis was obtained and approximated, respectively.

Appendix E. LRFD Strength Calculations for Semi-rigid Connection Test Specimen (Test No. 14S1)

E.1 Loading and Assumptions

It is assumed that the double web-angle carries the applied shear force, and the top and seat angles resist the applied moment.

P_D = 10 kips (dead load)

P_L = 30 kips (live load)

Strong-axis moment:

The shear loads are applied 2 in. away from the boltline.

M_D = (10 kips)·(2 in.) = 20 kips-in.

M_L = (30 kips)·(2 in.) = 60 kips-in.

E.1.1 LRFD Load Demand or Design Loads

According to the Load and Resistance Factored Design (LRFD) procedure (AISC 360-16 or AISC Manual 2017), LRFD design load, factored load or demand, P_u under gravity loads is:

$P_u = 1.2P_D + 1.6P_L$ (LRFD)

P_u = 1.2·(10 kips) + 1.6·(30 kips) = 60 kips (LRFD)

$M_u = 1.2M_D + 1.6M_L$ (LRFD)

M_u = 1.2·(20 kips) + 1.6·(60 kips) = 120 kips-in. = 120 kips-ft (LRFD)

Then, the required strength, R_u using the LRFD load combinations will be set equal to the design shear force, V_u applied on the connection. Then, for the cantilever beam and connection specimen:

$R_u = V_u = P_u = 1.2P_D + 1.6P_L$ (LRFD)

E.2 Properties of Test Specimen (Test No. 14S1)

For the following analysis, consider:

- t_w = web thickness of beam
- d = depth of beam
- T = clear distance between web fillets
- F_y = specified minimum yield strength
- F_u = specified minimum tensile strength

- t_f = flange thickness of column
- L = length of the angles

- Beam: W14x38
  - Specified minimum yield strength, F_y = 36 ksi ASTM A36
  - Specified minimum tensile strength, F_u = 58 ksi ASTM A36
  - Web thickness of beam, t_w = 0.31 in.
  - Web thickness of beam, t_f = 0.515 in.
  - Depth of beam, d = 14.1 in.

- Column: W12x96
  - Specified minimum yield strength, F_y = 36 ksi ASTM A36
  - Specified minimum tensile strength, F_u = 58 ksi ASTM A36
  - Flange thickness of column, t_f = 0.9 in. (Table 1-1, AISC Manual)

- Double angles: 4 x 3½ x 1/4
  - ASTM A36 F_y = 36 ksi F_u = 58 ksi
  - Length of the double angles, L= 8.5 in.
  - Number of bolts: 3
  - Diameter of bolts: ¾ in.
  - Type of bolts: A325-X (threads are excluded)

- Top and seat angles: 6 x 4 x 3/8
  - ASTM A36 F_y = 36 ksi F_u = 58 ksi
  - Length of the double angles, L= 8 in.
  - Gage in leg on the column flange = 2.5 in.
  - Bolt spacing in leg on column flange = 5.5 in.
  - Diameter of bolts: ¾ in.
  - Type of bolts: A325-X (threads are excluded)

E.3. Double Web-Angle Geometric Checks

For the following analysis, consider:

- t_{ang} = angle thickness
- L_{min} = minimum angle length
- L_{max} = maximum angle length
- L_{emin} = minimum edge distance
- L_{ev} = vertical edge distance of angle
- L_{eh} = horizontal edge distance of angle
- L_{svmin} = minimum vertical center to center bolt distance
- L_{sv} = vertical center to center bolt distance
- L_{smax} = maximum center to center bolt distance
- L_{wmin} = minimum weld length

- d = bolt diameter
- T = clear distance between web fillets of the beam

E.3.1 Angle Thickness

$t_{ang} \leq 5/8$ in. (Section J2.2b, AISC 360-16)

- t_{ang} = 1/4 in. ≤ 5/8 in. **OK**

where:

- t_{ang} = 1/4 in. – angle thickness

E.3.2 Angle Length

$L_{min} = T/2$ (Page 10-9, AISC Manual)

- L_{min} = (12.27 in.)/2 = 6.14 in.
- L = 9 in. ≥ L_{min} = (12.27 in.)/2 = 6.14 in. **OK**

$L_{max} = T$ (Page 10-9, AISC Manual)

- L_{max} = 12.27 in.
- L = 9 in. ≤ L_{max} = 12.27 in. **OK**

where:

- T = 12.27 in. – clear distance between web fillets of the beam
- L_{min} = 6.14 in. – minimum angle length
- L_{max} = 12.27 in. – maximum angle length

E.3.3 Angle Edge Distance Check (Bolted to Beam)

L_{emin} = 1 in. for bolt diameter of ¾ in. (Table J3.4, AISC 360-16)

- L_{ev} = 1.50 in. > L_{emin} = 1 in. **OK**
- L_{eh} = 1.50 in. > L_{emin} = 1 in. **OK**

where:

- L_{emin} = 1 in. – minimum edge distance
- L_{ev} = 1.50 in. – vertical edge distance of angle
- L_{eh} = 1.50 in. – horizontal edge distance of angle

E.3.4 Angle Bolt Spacing Check (Bolted to Beam)

$L_{svmin} = (8/3)\cdot d$ (Section J3.3, AISC 360-16)

- L_{svmin} = (8/3)·(¾ in.)
- L_{svmin} = 2 in.
- L_{sv} = 3 in. > L_{svmin} = 2 in. **OK**

L_{smax} = minimum of (24·t_{ang}) and 12 in. (Section J3.5, AISC 360-16)

- L_{smax} = 24·(0.25 in.)
- L_{smax} = 6 in.
- L_{sv} = 3 in. < L_{smax} = 6 in. **OK**

where:

- L_{svmin} = 2 in. – minimum vertical center to center bolt distance
- L_{sv} = 3 in. – vertical center to center bolt distance
- L_{smax} = 6 in. – maximum center to center bolt distance
- d = ¾ in. – bolt diameter

E.3.5 Angle Edge Distance Check (Bolted to Colum)

L_{emin} = 1 in. for bolt diameter of ¾ in. (Table J3.4, AISC 360-16)

- L_{ev} = 1.25 in. > L_{emin} = 1 in. **OK**
- L_{eh} = 1.25 in. > L_{emin} = 1 in. **OK**

where:

- L_{emin} = 1 in. – minimum edge distance
- L_{ev} = 1.50 in. – vertical edge distance of angle
- L_{eh} = 1.50 in. – horizontal edge distance of angle

E.3.6 Angle Bolt Spacing Check (Bolted to Column)

L_{svmin} = (8/3)·d (Section J3.3, AISC 360-16)

- L_{svmin} = (8/3)·(¾ in.)
- L_{svmin} = 2 in.
- L_{sv} = 3 in. > L_{svmin} = 2 in. **OK**

L_{smax} = minimum of (24·t_{ang}) and 12 in. (Section J3.5, AISC 360-16)

- L_{smax} = 24·(0.25 in.)
- L_{smax} = 6 in.
- L_{sv} = 3 in. < L_{smax} = 6 in. **OK**

where:

- L_{svmin} = 2 in. – minimum vertical center to center bolt distance

- L_{sv} = 3 in. – vertical center to center bolt distance
- L_{smax} = 6 in. – maximum center to center bolt distance
- d = ¾ in. – bolt diameter

E.4. Double Web-Angle LRFD Design Checks

In this section, design of angles, bolts, beam and column are checked for the double web-angle. The LRFD design strength ϕR_n is calculated following the requirements of AISC Construction Manual (2017) and AISC 360-16. The calculated design strength ϕR_n is then compared with the design demand R_u calculated from structural analysis using the factored external loads.

For the side of the web-angle bolted to beam web, shear, bearing, tearout, shear yielding, rupture, and block shear failure limit states are checked in Sections E.4.1.1 through E.4.1.6.

For the side of the web-angle bolted to column flange, shear, bearing, tearout, shear yielding, rupture, and block shear failure limit states are checked in Sections E.4.2.1 through E.4.2.6.

For beam, bearing, tearout and shear yielding failure limit states are checked in Sections E.4.3.1 through E.4.3.3.

For column, bearing is checked (Section E.4.4.1).

For the following analysis, consider:

- F_{nv} = nominal shear strength of fasteners
- A_b = nominal bolt area
- ϕ = strength factor (LRFD)
- F_{nt} = nominal tensile strength of fasteners
- d = nominal bolt diameter
- t = thickness of the beam
- F_u = specified minimum tensile strength of beam
- l_c = clear distance
- d_h = nominal hole dimension
- L_{sv} = vertical center to center bolt distance
- L_{ev} = vertical edge distance of angle
- L_{csv} = clear vertical distance between bolts
- L_{cev} = clear vertical edge distance
- A_{nv} = net area subject to shear, in.^{2}
- L_h = hole dimension for tension and shear net area
- L_e = effective length
- n = number of the bolts
- U_{bs} = stress index for uniform tension stress
- A_{nt} = net area subject to tension

- A_{gv} = gross area subject to shear
- A_{nv} = net area subject to shear
- L_{ev} = vertical edge distance of angle
- L_{eh} = horizontal edge distance of angle
- L = length of the angles

E.4.1 Angle (Beam Side)

E.4.1.1 Bolt shear

$R_n = F_n \cdot A_b$ (Eq. J3-1, AISC 360-16)

- Shear strength of one bolt:
- $\phi R_n = 0.75 \cdot 68 \cdot 0.442 =$ **22.54 kips/bolt**
- Total shear strength of three bolts double angle:
- ϕR_n = 2 x 3 x 22.54 kips = **135.24 kips > 60 kips OK**

where:

- F_{nv} = 68 ksi – nominal shear strength of fasteners (Table J3.2, AISC 360-16)
- A_b = 0.442 in.^{2} – nominal bolt area
- ϕ = 0.75 – strength factor (LRFD)

E.4.1.2 Bolt bearing

The nominal bearing strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 2.4 \cdot d \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6a)

- $R_n = 2.4 \cdot (3/4 \text{ in.}) \cdot (0.25 \text{ in.}) \cdot (58 \text{ ksi})$
- R_n = 26.1 kips/bolt
- ϕR_n = 0.75·(2 angles)·(3 bolts)·26.1 kips
- ϕR_n = **117.45 kips/connection > 60 kips OK**

where:

- d = 3/4 in. – nominal bolt diameter
- t = 0.25 in. – thickness of angle
- F_u = 58 ksi – specified minimum tensile stress of angle
- ϕ = 0.75 – strength factor (LRFD)

E.4.1.3 Bolt tearout

The available tearout strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 1.2 \cdot l_c \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6c)

- $L_{csv} = L_{sv} - d_h$
- L_{csv} = 3 in. - 13/16 in. = 2.19 in.
- $R_{n\text{-}span}$ = 1.2·(2.19 in.)·(0.25 in.)·(58 ksi)
- $R_{n\text{-}span}$ = 38.1 kips/bolt
- $L_{cev} = L_{ev} - d_h /2$
- L_{cev} = 1.5 in. - (13/16)/2 in. = 1.09 in.
- $R_{n\text{-}end}$ = 1.2·(1.09 in.)·(0.25 in.)·(58 ksi)
- $R_{n\text{-}end}$ = 18.97 kips/bolt
- ϕR_n = 0.75·(2 angles)·[2·(38.1 kips) + (18.97 kips)]
- ϕR_n = **142.76 kips/conn. > 60 kips OK**

where:

- l_c : clear distance
- d_h = 13/16 in. – nominal hole dimension (3/4-in.-diameter bolt)
- L_{sv} = 3 in. – vertical center to center bolt distance
- L_{ev} = 1.5 in. – vertical edge distance of angle
- L_{csv} = 2.19 in. – clear vertical distance between bolts
- L_{cev} = 1.09 in. – clear vertical edge distance
- ϕ = 0.75 – strength factor (LRFD)

E.4.1.4 Shear yielding

$R_n = 0.60 \cdot F_y \cdot A_{gv}$ ϕ = 1.00 (LRFD) (AISC 360-16, Eq. J4-3)

- A_{gv} = (8.5 in.)·(0.25 in.) = 2.125 in.^{2}
- ϕR_n = 1.00·(2 angles) 0.60·(36 kips)·(2.125 in.^{2})
- ϕR_n = **91.8 kips > 60 kips OK**

where:

- A_{gv} = 2.125 in.^{2} – gross area subject to shear
- ϕ = 1.00 – strength factor (LRFD)
- L = 8.5 in. – length of angle
- t = 0.25 in. – thickness of angle
- F_y = 36 ksi – specified minimum yield strength

E.4.1.5 Shear rupture

$R_n = 0.60 \cdot F_u \cdot A_{nv}$ (AISC 360-16, Eq. J4-4)

- $L_h = d_h + 1/16$ (Section B4-3b, AISC Manual)
- L_h = 0.813 + 1/16 = 0.875 in.
- $L_e = L - n \cdot L_h$

- L_e = 8.5 - 3·0.875 = 5.875 in.
- $A_{nv} = L_e \cdot t_p$
- A_{nv} = (5.875 in.)·(0.25 in.) = 1.47 in.^{2}
- ϕR_n = (0.75)·(2 angles)·0.60·(58 kips)·(1.47 in.^{2})
- ϕR_n = **76.73 kips > 60 kips OK**

where:

- A_{nv} = 1.47 in.^{2} – net area subject to shear, in.^{2}
- L_h = 0.875 in. – hole dimension for tension and shear net area
- L_e = 5.875 in. – effective length
- ϕ = 0.75 – strength factor (LRFD)
- n = 3 – number of the bolts
- t = 0.25 in. – thickness of angle

E.4.1.6 Block shear

$R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$ (AISC 360-16, Eq. J4-5)

- $A_{gv} = (L - L_{ev}) \cdot (t_p)$
- A_{gv} = (2 angles)·(8.5 in. - 1.25 in.)·(0.25 in.) = 3.63 in.^{2}
- $A_{nv} = A_{gv}$ - (2 angles)·(n-0.5)·(d_h + 1/16 in.)·(t_p)
- A_{nv} = 3.63 in.^{2} - (2 angles)·(3-0.5)·(0.813 + 1/16 in.)·(0.25 in.)
- A_{nv} = 2.54 in.^{2}
- A_{nt} = (2 angles)·[L_{eh} - 0.5·(d_h + 1/16 in.)]·(t_p)
- A_{nt} = (2 angles)·[1.25 in. - 0.5·(0.813 in. + 1/16 in.)]·(0.25 in.)
- A_{nt} = 0.41 in.^{2}
- U_{bs} = 1 (as the tension stress is uniform)
- $R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$
- R_n = 0.60·(58 kips/in.^{2})·(2.54 in.^{2}) + 1·(58 kips/in.^{2})·(0.41 in.^{2}) ≤ 0.60·(36 kips/in.^{2})·(3.63 in.^{2}) + 1·(58 kips/in.^{2})·(0.41 in.^{2})
- R_n = 112.17 kips ≤ **102.19 kips**
- ϕR_n = 0.75·(111.74 kips)
- ϕR_n **= 71.53 kips > 60 kips OK**

where:

- U_{bs} = 1 – Stress index for uniform tension stress
- A_{nt} = 0.41 in.^{2} – net area subject to tension
- A_{gv} = 3.63 in.^{2} – gross area subject to shear
- A_{nv} = 2.54 in.^{2} – net area subject to shear
- L_{ev} = 1.25 in. – vertical edge distance of angle
- L_{eh} = 1.25 in. – horizontal edge distance of angle

- ϕ = 0.75 – strength factor (LRFD)

E.4.2 Angle (Column Side)

E.4.2.1 Bolt shear

$R_n = F_n \cdot A_b$ (Eq. J3-1, AISC 360-16)

- Shear strength of one bolt:
- ϕR_n = 0.75·68·0.442 = **22.54 kips/bolt**
- Total shear strength of three bolts double angle:
- ϕR_n = 2 x 3 x 22.54 kips = **135.24 kips > 60 kips OK**

where:

- F_{nv} = 68 ksi – nominal shear strength of fasteners (Table J3.2, AISC 360-16)
- A_b = 0.442 in.^{2} – nominal bolt area
- ϕ = 0.75 – strength factor (LRFD)

E.4.2.2 Bolt bearing

The nominal bearing strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 2.4 \cdot d \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6a)

- R_n =2.4·(3/4 in.)·(0.25 in.)·(58 ksi)
- R_n = 26.1 kips/bolt
- ϕR_n = 0.75·(2 angles)·(3 bolts)·26.1 kips
- ϕR_n = **117.45 kips/connection > 60 kips OK**

where:

- d = 3/4 in. – nominal bolt diameter
- t = 0.25 in. – thickness of angle
- F_u = 58 ksi – specified minimum tensile stress of angle
- ϕ = 0.75 – strength factor (LRFD)

E.4.2.3 Bolt tearout

The available tearout strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 1.2 \cdot l_c \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6c)

- $L_{csv} = L_{sv} - d_h$
- L_{csv} = 3 in. - 13/16 in. = 2.19 in.

- $R_{n\text{-}span}$ = 1.2·(2.19 in.)·(0.25 in.)·(58 ksi)
- $R_{n\text{-}span}$ = 38.1 kips/bolt
- $L_{cev} = L_{ev} - d_h/2$
- L_{cev} = 1.5 in. - (13/16)/2 in. = 1.09 in.
- $R_{n\text{-}end}$ = 1.2·(1.09 in.)·(0.25 in.)·(58 ksi)
- $R_{n\text{-}end}$ = 18.97 kips/bolt
- ϕR_n = 0.75·(2 angles)·[2·(38.1 kips) + (18.97 kips)]
- ϕR_n = **142.76 kips/conn. > 60 kips OK**

where:

- l_c : clear distance
- d_h = 13/16 in. – nominal hole dimension (3/4-in.-diameter bolt)
- L_{sv} = 3 in. – vertical center to center bolt distance
- L_{ev} = 1.5 in. – vertical edge distance of angle
- L_{csv} = 2.19 in. – clear vertical distance between bolts
- L_{cev} = 1.09 in. – clear vertical edge distance
- ϕ = 0.75 – strength factor (LRFD)

E.4.2.4 Shear yielding

$R_n = 0.60 \cdot F_y \cdot A_{gv}$ ϕ = 1.00 (LRFD) (AISC 360-16, Eq. J4-3)

- A_{gv} = (8.5 in.)·(0.25 in.) = 2.125 in.^{2}
- ϕR_n = 1.00·(2 angles) 0.60·(36 kips)·(2.125 in.^{2})
- ϕR_n = **91.8 kips > 60 kips OK**

where:

- A_{gv} = 2.125 in.^{2} – gross area subject to shear
- ϕ = 1.00 – strength factor (LRFD)
- L = 8.5 in. – length of angle
- t = 0.25 in. – thickness of angle
- F_y = 36 ksi – specified minimum yield strength

E.4.2.5 Shear rupture

$R_n = 0.60 \cdot F_u \cdot A_{nv}$ (AISC 360-16, Eq. J4-4)

- $L_h = d_h + 1/16$ (Section B4-3b, AISC Manual)
- L_h = 0.813 + 1/16 = 0.875 in.
- $L_e = L - n \cdot L_h$
- L_e = 8.5 - 3·0.875 = 5.875 in.
- $A_{nv} = L_e \cdot t_p$

- A_{nv} = (5.875 in.)·(0.25 in.) = 1.47 in.^{2}
- ϕR_n = (0.75)·(2 angles)·0.60·(58 kips)·(1.47 in.^{2})
- ϕR_n = **76.73 kips > 60 kips OK**

where:

- A_{nv} = 1.47 in.^{2} – net area subject to shear, in.^{2}
- L_h = 0.875 in. – hole dimension for tension and shear net area
- L_e = 5.875 in. – effective length
- ϕ = 0.75 – strength factor (LRFD)
- n = 3 – number of the bolts
- t = 0.25 in. – thickness of angle

E.4.2.6 Block shear

$R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$ (AISC 360-16, Eq. J4-5)

- $A_{gv} = (L - L_{ev}) \cdot (t_p)$
- A_{gv} = (2 angles)·(8.5 in. - 1.25 in.)·(0.25 in.) = 3.63 in.^{2}
- $A_{nv} = A_{gv}$ - (2 angles)·(n-0.5)·(d_h + 1/16 in.)·(t_p)
- A_{nv} = 3.63 in.^{2} - (2 angles)·(3-0.5)·(0.813 + 1/16 in.)·(0.25 in.)
- A_{nv} = 2.54 in.^{2}
- A_{nt} = (2 angles)·[L_{eh} - 0.5·(d_h + 1/16 in.)]·(t_p)
- A_{nt} = (2 angles)·[1.25 in. - 0.5·(0.813 in. + 1/16 in.)]·(0.25 in.)
- A_{nt} = 0.41 in.^{2}
- U_{bs} = 1 (as the tension stress is uniform)
- $R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$
- R_n = 0.60·(58 kips/in.^{2})·(2.54 in.^{2}) + 1·(58 kips/in.^{2})·(0.41 in.^{2}) ≤ 0.60·(36 kips/in.^{2})·(3.63 in.^{2}) + 1·(58 kips/in.^{2})·(0.41 in.^{2})
- R_n = 112.17 kips ≤ **102.19 kips**
- ϕR_n = 0.75·(111.74 kips)
- ϕR_n **= 71.53 kips > 60 kips OK**

where:

- U_{bs} = 1 – stress index for uniform tension stress
- A_{nt} = 0.41 in.^{2} – net area subject to tension
- A_{gv} = 3.63 in.^{2} – gross area subject to shear
- A_{nv} = 2.54 in.^{2} – net area subject to shear
- L_{ev} = 1.25 in. – vertical edge distance of angle
- L_{eh} = 1.25 in. – horizontal edge distance of angle
- ϕ = 0.75 – strength factor (LRFD)

E.4.3 Beam

E.4.3.1 Bolt bearing

$r_n = 2.4 \cdot d \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6a)

- r_n =2.4·(3/4 in.)·(0.31 in.)·(58 ksi)
- r_n = 32.36 kips/bolt
- ϕR_n = 0.75·(3 bolts)·32.36 kips
- ϕR_n = **72.81 kips/connection > 60 kips OK**

where:

- d = 3/4 in. – nominal bolt diameter
- t = 0.31 in. – thickness of the beam
- F_u = 58 ksi – specified minimum tensile stress of beam
- ϕ = 0.75 – strength factor (LRFD)

E.4.3.2 Bolt tearout

The available tearout strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 1.2 \cdot l_c \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6c)

- L_{csv} = 3 in. - 13/16 in. = 2.19 in.
- L_{cev} = [14.1 in. - 2·(3 in.) - (13/16) in.]/2 = 3.64 in.
- $R_{n\text{-}span}$ = 1.2·(2.19 in.)·(0.31 in.)·(58 ksi)
- $R_{n\text{-}span}$ = 47.25 kips/bolt
- $R_{n\text{-}end}$ = 1.2·(3.64 in.)·(0.31 in.)·(58 ksi)
- $R_{n\text{-}end}$ = 78.53 kips/bolt
- ϕR_n = 0.75·[2·(47.25 kips) + (78.53 kips)] = **129.77 kips/connection > 60 kips OK**

where:

- l_c : clear distance
- d_h = 13/16 in. – nominal hole dimension (3/4-in.-diameter bolt)
- L_{sv} = 3 in. – vertical center to center bolt distance
- L_{csv} = 2.19 in. – clear vertical distance between bolts
- L_{cev} = 3.64 in. – clear vertical edge distance
- ϕ = 0.75 – strength factor (LRFD)

E.4.3.3 Shear yielding

$R_n = 0.60 \cdot F_y \cdot A_{gv}$ (AISC 360-16, Eq. J4-3)

- A_{gv} = (14.1 in.)·(0.31 in.) = 4.37 in.^{2}

- ϕR_n = 1.00·0.60·(36 kips)·(4.37 in.^{2})
- ϕR_n = **94.41 kips > 60 kips OK**

where:

- A_{gv} = 4.37 in.^{2} – gross area subject to shear
- ϕ = 1.00 – strength factor (LRFD)
- F_y = 36 ksi – specified minimum yield strength

E.4.4 Column

E.4.4.1 Bolt bearing

$r_n = 2.4 \cdot d \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6a)

- r_n =2.4·(3/4 in.)·(0.9 in.)·(58 ksi)
- r_n = 93.96 kips/bolt
- ϕR_n = 0.75·(3 bolts)·93.96 kips
- ϕR_n = **211.41 kips/connection > 60 kips OK**

where:

- d = 3/4 in. – nominal bolt diameter
- t = 0.9 in. – thickness of column web
- F_u = 58 ksi – specified minimum tensile stress of beam
- ϕ = 0.75 – strength factor (LRFD)

E.5 Top- and Seat-Angle Geometric Checks

For the following analysis, consider:

- L_{emin} = minimum edge distance
- L_{el} = longitudinal edge distance of angle
- L_{et} = transverse edge distance of angle
- L_{st} = center to center bolt distance
- L_{smax} = maximum center to center bolt distance
- d = bolt diameter

E.5.1 Angle Edge Distance Check (Bolted to Top Beam Flange)

L_{emin} = 1 in. for bolt diameter of ¾ in. (Table J3.4, AISC 360-16)

- L_{el} = 1.25 in. > L_{emin} = 1 in. **OK**
- L_{et} = 1.25 in. > L_{emin} = 1 in. **OK**

where:

- L_{emin} = 1 in. – minimum edge distance

- L_{el} = 1.25 in. – longitudinal edge distance of angle
- L_{et} = 1.25 in. – transverse edge distance of angle

E.5.2 Angle Bolt Spacing Check (Bolted to Top Beam Flange)

$L_{smin} = (8/3)\cdot d$ (Section J3.3, AISC 360-16)

- $L_{svmin} = (8/3)\cdot(¾\text{ in.})$
- L_{svmin} = 2 in.
- L_{st} = 3.5 in. > L_{svmin} = 2 in. **OK**
- L_{sl} = 2.5 in. > L_{svmin} = 2 in. **OK**

L_{smax} = minimum of $(24\cdot t_{ang})$ and 12 in. (Section J3.5, AISC 360-16)

- $L_{smax} = 24\cdot(0.375\text{ in.})$
- L_{smax} = 9 in.
- L_{st} = 3.5 in. < L_{smax} = 9 in. **OK**
- L_{sl} = 2.5 in. < L_{smax} = 9 in. **OK**

where:

- L_{smin} = 2 in. – minimum center to center bolt distance
- L_{st} = 3.5 in. – transfer center to center bolt distance
- L_{smax} = 9 in. – maximum center to center bolt distance
- d = ¾ in. – bolt diameter

E.5.3 Angle Bolt Spacing Check (Bolted to Column Flange)

$L_{smin} = (8/3)\cdot d$ (Section J3.3, AISC 360-16)

- $L_{svmin} = (8/3)\cdot(¾\text{ in.})$
- L_{svmin} = 2 in.
- L_{st} = 5.5 in. > L_{svmin} = 2 in. **OK**

L_{smax} = minimum of $(24\cdot t_{ang})$ and 12 in. (Section J3.5, AISC 360-16)

- $L_{smax} = 24\cdot(0.375\text{ in.})$
- L_{smax} = 9 in.
- L_{sl} = 5.5 in. < L_{smax} = 9 in. **OK**

where:

- L_{smin} = 2 in. – minimum center to center bolt distance
- L_{st} = 3.5 in. – transfer center to center bolt distance
- L_{smax} = 9 in. – maximum center to center bolt distance
- d = ¾ in. – bolt diameter

E.6 Top and Seat Angle LRFD Design Checks

In this section, design of angles, bolts, beam and column are checked for the top and seat angles. The LRFD design strength ϕR_n is calculated following the requirements of AISC Construction Manual (2017) and AISC 360-16. The calculated design strength ϕR_n is then compared with the design demand R_u calculated from structural analysis using the factored external loads.

Flange bolt shear force and flange tension force corresponding the design moment are calculated in Sections E.6.1.1 and E.6.1.2.

For the side of the web-angle bolted to beam web, tension yielding, tension rupture, compression, bolt shear, bearing, tearout, and block shear failure limit states are checked in Sections E.6.1.3 through E.6.1.9.

For the side of the web-angle bolted to column flange, shear yielding, shear rupture, tension (due prying action) failure limit states are checked in Sections E.6.2.1 through E.6.2.3.

For beam, bearing, tearout, block shear and flexural failure limit states are checked in Sections E.6.3.1 through E.6.3.4.

For column, web panel zone, flange local bending, web local yielding and web local crippling are checked in Sections E.6.4.1 through E.6.4.4.

E.6.1 Angle (Beam Side)

E.6.1.1 Flange Bolt Shear Force

$P_{uf} = M_u/d_m$

- P_{uf} = 120 kips-in. / 14.1 in. = **8.51 kips**

where:

- d_m = 14.1 in. – depth of beam
- P_{uf} = 12.07 kips – flange bolt shear

E.6.1.2 Flange Tension Force

The moment arm between flange forces is equal to the summation of gage in leg on column flange, beam depth and a half of the thickness of seated-angle.

$P_{uf} = M_u/(d + g + t/2)$

- P_{uf} = 120 kips-in. / (14.1 in. + 2.5 in. + 0.375 in./2) = **7.15 kips**

where:

- g = 2.5 in – gage in leg on column flange
- d_m = 14.1 in – depth of beam
- t = 0.375 in. – plate thickness
- P_{uf} = 11.49 kips – flange tension force

E.6.1.3 Tension yielding

$R_n = F_y \cdot A_g$ (AISC 360-16, Eq. J4-1)

where:

- A_g = (8 in.)·(0.375 in.) = 3 in.^{2}
- R_n = (36 ksi)·3 = 108 kips
- ϕR_n = 0.9·(108 ksi) = **97.2 kips > 7.15 kips OK**

where:

- A_g =3 in.^{2} – gross area
- F_y = 36 ksi – specified minimum yield strength
- ϕ = 0.9 – strength factor (LRFD)

E.6.1.4 Tension rupture

$R_n = F_u \cdot A_e$ (AISC 360-16, Eq. J4-2)

- A_g = (8 in.)·(0.375 in.) = 3 in.^{2}
- $L_h = d_h + 1/16$ (Section B4-3b, AISC Manual)
- L_h = 0.813 + 1/16 = 0.875 in.
- $A_n = (L - n \cdot L_h) \cdot t_p$
- A_n = (8 in. - 2·0.875 in.)·(0.375 in.) = 2.34 in.^{2}
- $A_e = A_n \leq 0.85A_g$ (AISC 360-16, Sec. J4-1)
- A_e = 2.34 in.^{2}
- R_n = (58 ksi)·(2.34 in.^{2}) = 135.7 kips
- ϕR_n = 0.75·(135.7 ksi) = **101.78 kips > 7.15 kips OK**

where:

- A_g =3 in.^{2} – gross area
- A_e = 2.34 in.^{2} – effective net area
- L_h = 0.875 in. – hole dimension
- F_u = 58 ksi – specified minimum tensile stress of beam
- ϕ = 0.75 – strength factor (LRFD)

E.6.1.5 Compression

$P_n = F_y \cdot A_g$ when $L_c/r \leq 25$ (AISC 360-16, Sec. J4-4)

- $r = t_p/(12)^{1/2}$ = 0.375 in./(12)$^{1/2}$
- r = 0.108 in.
- $L_c = K \cdot L/r$ = 0.65·4.75 in./0.108 in.
- L_c = 28.6 > 25 **OK**
- $F_e = \pi^2 \cdot E/(L_c/r)^2$ (AISC 360-16, Eq. E3-4)

- F_e = π^2·2.90E+7 psi / (28.6/0.108 in.)^2 = 3.52E+05 psi
- $F_{cr} = 0.658^{(Fy/Fe)} \cdot F_y$ when $L_c/r \le 4.71 \cdot (E/F_y)^{1/2}$ (AISC 360-16, Eq. E3-2)
- 28.6/0.108 in. ≤ 4.71·(2.90E+7 psi / 36000 psi)$^{1/2}$ **OK**
- F_{cr} = 0.658(36000 /3.52E+05)·36,000 psi
- F_{cr} = 34,490.89 psi
- $A_g = L_p \cdot t_p$ = 8 in.·0.375 in.
- A_g = 3 in.^{2}
- $\phi P_n = \phi \cdot F_{cr} \cdot A_g$ = 0.9·(34,490.89 psi)·(3 in.^{2})
- ϕP_n = **93.125 kips > 7.15 kips** **OK**

where:

- K = 0.65 – effective length factor
- L = 4.75 in. – laterally unbraced length of the member
- L_c = 28.6 – effective length
- F_e = 3.52E+05 psi – elastic buckling stress
- F_{cr} = 34,490.89 psi – critical stress
- A_g = 3 in.^{2} – gross area
- ϕ = 0.9 – strength factor (LRFD)

E.6.1.6 Bolt shear

$R_n = F_n \cdot A_b$ (Eq. J3-1, AISC 360-16)

- Shear strength of one bolt:
- ϕR_n = 0.75·68·0.442 = 22.54 kips/bolt
- Total shear strength of four bolts:
- ϕR_n = 4 x 22.54 kips = **90.16 kips > 8.51 kips** **OK**

where:

- F_{nv} = 68 ksi – nominal shear strength of fasteners (Table J3.2, AISC 360-16)
- A_b = 0.442 in.^{2} – nominal bolt area
- ϕ = 0.75 – strength factor (LRFD)

E.6.1.7 Bolt bearing

The nominal bearing strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 2.4 \cdot d \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6a)

- R_n =2.4·(3/4 in.)·(0.375 in.)·(58 ksi)
- R_n = 39.15 kips/bolt
- ϕR_n = 0.75·(4 bolts)·39.15 kips

- ϕR_n = **117.45 kips/connection > 8.51 kips OK**

where:

- d = 3/4 in. – nominal bolt diameter
- t = 0.375 in. – thickness of angle
- F_u = 58 ksi – specified minimum tensile stress of angle
- ϕ = 0.75 – strength factor (LRFD)

E.6.1.8 Bolt Tearout

The available tearout strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 1.2 \cdot l_c \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6c)

- $L_{csv} = L_{sv} - d_h$
- L_{csv} = 2.5 in. - 13/16 in. = 1.69 in.
- $R_{n\text{-}span}$ = 1.2·(1.69 in.)·(0.375 in.)·(58 ksi)
- $R_{n\text{-}span}$ = 44.1 kips/bolt
- $L_{cev} = L_{ev} - d_h/2$
- L_{cev} = 1.25 in. - (13/16)/2 in. = 0.84 in.
- $R_{n\text{-}end}$ = 1.2·(0.84 in.)·(0.375 in.)·(58 ksi)
- $R_{n\text{-}end}$ = 21.92 kips/bolt
- ϕR_n = 0.75·2·[(44.1 kips) + (21.92 kips)] = **99.03 kips/connection > 8.51 kips OK**

where:

- l_c : clear distance
- d_h = 13/16 in. – nominal hole dimension (3/4-in.-diameter bolt)
- L_{sv} = 2.5 in. – vertical center to center bolt distance
- L_{ev} = 1.25 in. – vertical edge distance of angle
- L_{csv} = 1.69 in. – clear vertical distance between bolts
- L_{cev} = 0.84 in. – clear vertical edge distance
- ϕ = 0.75 – strength factor (LRFD)

E.6.1.9 Block shear

$R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$ (AISC 360-16, Eq. J4-5)

- $A_{gv} = (L - L_{ev}) \cdot (t_p)$
- A_{gv} = 2·(2.5 in. + 1.25 in.)·(0.375 in.) = 2.81 in.^{2}
- $A_{nv} = A_{gv}$ - 2·(n-0.5)·(d_h + 1/16 in.)·(t_p)
- A_{nv} = 2.81 in.^{2} - 2·(2-0.5)·(0.813 + 1/16 in.)·(0.375 in.)
- A_{nv} = 1.83 in.^{2}

- $A_{nt} = 2 \cdot [L_{eh} - 0.5 \cdot (d_h + 1/16 \text{ in.})] \cdot (t_p)$
- $A_{nt} = 2 \cdot [1.25 \text{ in.} - 0.5 \cdot (0.813 \text{ in.} + 1/16 \text{ in.})] \cdot (0.375 \text{ in.})$
- $A_{nt} = 0.61 \text{ in.}^2$
- $U_{bs} = 1$ (as the tension stress is uniform)
- $R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$
- $R_n = 0.60 \cdot (58 \text{ kips/in.}^2) \cdot (1.83 \text{ in.}^2) + 1 \cdot (58 \text{ kips/in.}^2) \cdot (0.61 \text{ in.}^2) \leq 0.60 \cdot (36 \text{ kips/in.}^2) \cdot (2.81 \text{ in.}^2) + 1 \cdot (58 \text{ kips/in.}^2) \cdot (0.61 \text{ in.}^2)$
- $R_n = 99.06$ kips $\leq$ **96.08 kips**
- $\phi R_n = 0.75 \cdot (96.08 \text{ kips})$
- ϕR_n **= 67.25 kips > 7.15 kips OK**

where:

- $U_{bs} = 1$ – stress index for uniform tension stress
- $A_{nt} = 0.61 \text{ in.}^2$ – net area subject to tension
- $A_{gv} = 2.81 \text{ in.}^2$ – gross area subject to shear
- $A_{nv} = 1.83 \text{ in.}^2$ – net area subject to shear
- $L_{ev} = 1.25$ in. – vertical edge distance of angle
- $L_{eh} = 1.25$ in. – horizontal edge distance of angle
- $\phi = 0.75$ – strength factor (LRFD)

E.6.2 Angle (Column Side)

E.6.2.1 Shear yielding

$R_n = 0.60 \cdot F_y \cdot A_{gv}$ $\phi = 1.00$ (LRFD) (AISC 360-16, Eq. J4-3)

- $A_{gv} = (8 \text{ in.}) \cdot (0.375 \text{ in.}) = 3 \text{ in.}^2$
- $\phi R_n = 1.00 \cdot 0.60 \cdot (36 \text{ kips}) \cdot (3 \text{ in.}^2)$
- ϕR_n = **64.8 kips > 7.15 kips OK**

where:

- $A_{gv} = 3 \text{ in.}^2$ – gross area subject to shear
- $L = 8$ in. – length of angle
- $t = 0.375$ in. – thickness of angle
- $\phi = 1.00$ – strength factor (LRFD)

E.6.2.2 Shear rupture

$R_n = 0.60 \cdot F_u \cdot A_{nv}$ (AISC 360-16, Eq. J4-4)

- $L_h = d_h + 1/16$ (Section B4-3b, AISC Manual)
- $L_h = 0.813 + 1/16 = 0.875$ in.
- $L_e = L - n \cdot L_h$

- L_e = 8 - 2·0.875 = 6.25 in.
- $A_{nv} = L_e \cdot t_p$
- A_{nv} = (6.25 in.)·(0.375 in.) = 2.34 in.^{2}
- ϕR_n = (0.75)·0.60·(58 kips)·(2.34 in.^{2})
- ϕR_n = **61.07 kips > 7.15 kips OK**

where:

- A_{nv} = 2.34 in.^{2} – net area subject to shear, in.^{2}
- L_h = 0.875 in. – hole dimension for tension and shear net area
- L_e = 6.25 in. – effective length
- n = 2 – number of the bolts
- t = 0.25 in. – thickness of angle
- ϕ = 0.75 – strength factor (LRFD)

E.6.2.3 Tension capacity due prying action

$T_c = B_c \cdot Q$ (AISC Manual, Eq. 9-27)

$a' = (a + d/2) \leq (1.25 \cdot b + d/2)$

a' = (1.5 in. + 0.75 in./2) ≤ (1.25·2.31 in. + 0.75 in./2)

a' = **1.875 in.** ≤ 3.26 in.

a' = 1.875 in.

$b' = b - d/2$ = 2.31 in. – (0.75 in.)/2

b' = 1.94 in.

$\rho = b'/a'$ = (1.94 in.)/(1.875 in.)

ρ = 1.03

$p = L/n$ = (8 in.)/2 = 4 in.

$\delta = 1 - d'/p$ = 1 - (0.813 in.)/(4 in.)

δ = 0.80

$B_c = \phi R_n = \phi \cdot F_{nt} \cdot A_b$

B_c = 0.75·(90 ksi)·(0.442 in.^{2})

B_c = 29.84 kips (per bolt)

$t_c = ((4 \cdot B \cdot b')/(\phi \cdot p \cdot F_u))^{1/2}$

$t_c = [4 \cdot (29.84 \text{ kips}) \cdot (1.938 \text{ in.})/(0.9 \cdot (4 \text{ in.}) \cdot (58 \text{ ksi}))]^{1/2}$

$t_c = 1.05$ in.

$\alpha' = (1/(\delta \cdot (1 + \rho))) \cdot ((t_c/t_p)^2 - 1)$

$\alpha' = (1/(0.80 \cdot (1 + 1.03))) \cdot ((1.05 \text{ in.})/(0.375 \text{ in.})^2 - 1)$

$\alpha' = 3.99$

$Q = (t_p/t_c)^2 \cdot (1 + \delta)$

$Q = ((0.375 \text{ in.})/(1.05 \text{ in.}))^2 \cdot (1 + 0.8)$

$Q = 0.23$

$\phi T_c = (2 \text{ bolts}) \cdot B_c \cdot Q = 2 \cdot (29.84 \text{ kips}) \cdot (0.23)$

ϕT_c = **13.73 kips > 7.15 kips** **OK (Governs)**

where:

- a =1.5 in. – distance from the bolt centerline to the edge of the fitting
- a' = 1.875 in. – distance for prying action
- b' = 1.94 in. – distance for prying action
- ρ = 1.03 – prying distances ratio
- δ = 0.80 – ratio of the net length at bolt line to gross length at the face of the stem or leg of angle
- p = 4 in. – average pitch of the bolts
- t_c = 1.05 in. – flange thickness
- t_p = 0.375 in. – thickness of angle
- B_c = 29.84 kips – available tensile strength per bolt
- Q = 0.23 – prying action coefficient
- ϕ = 0.75 – strength factor (LRFD)

E.6.3 Beam

E.6.3.1 Bolt bearing

$r_n = 2.4 \cdot d \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6a)

- r_n =2.4·(3/4 in.)·(0.515 in.)·(58 ksi)
- r_n = 53.77 kips/bolt
- ϕR_n = 0.75·(3 bolts)·53.77 kips

- ϕR_n = **120.97 kips/connection > 8.51 kips OK**

where:

- d = 3/4 in. – nominal bolt diameter
- t = 0.515 in. – thickness of the beam flange
- F_u = 58 ksi – specified minimum tensile stress of beam
- ϕ = 0.75 – strength factor (LRFD)

E.6.3.2 Bolt tearout

The available tearout strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 1.2 \cdot l_c \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6c)

- L_{csv} = 2.5 in. - 13/16 in. = 1.69 in.
- L_{cev} = [8.5 in. - 2·(2.5 in.) - (13/16) in.]/2 = 1.34 in.
- $R_{n\text{-}span}$ = 1.2·(1.69 in.)·(0.515 in.)·(58 ksi)
- $R_{n\text{-}span}$ = 60.58 kips/bolt
- $R_{n\text{-}end}$ = 1.2·(1.34 in.)·(0.515 in.)·(58 ksi)
- $R_{n\text{-}end}$ = 47.10 kips/bolt
- ϕR_n = 2·0.75·[·(60.58 kips) + (47.10 kips)] = **161.52 kips/connection > 8.51 kips OK**

where:

- l_c : clear distance
- d_h = 13/16 in. – nominal hole dimension (3/4-in.-diameter bolt)
- L_{sv} = 2.5 in. – vertical center to center bolt distance
- L_{csv} = 1.69 in. – clear vertical distance between bolts
- L_{cev} = 1.34 in. – clear vertical edge distance
- t = 0.515 in. – thickness of the beam flange
- ϕ = 0.75 – strength factor (LRFD)

E.6.3.3 Block shear

$R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$ (AISC 360-16, Eq. J4-5)

- $A_{gv} = 2 \cdot (L_s - L_{ev}) \cdot (t_p)$
- A_{gv} = 2·(2.5 in. + 1.75 in.)·(0.515 in.) = 4.38 in.^{2}
- A_{nv} = 2·(t_p)·[L_{ev} + (n-1)·(L_s - (d_h + 1/16 in.)) - (d_h + 1/16 in.)/2)]
- A_{nv} = 2·(0.515 in.)·(1.75 in. + (2-1)·(2.5 - 0.813 in. - 1/16 in.)-(0.813 in. + 1/16 in.)/2)
- A_{nv} = 3.02 in.^{2}
- A_{nt} = [b- (n-1)·(d_h + 1/16 in.)]·(t_p)
- A_{nt} = [3.25 in. - 1·(0.813 in. + 1/16 in.)]·(0.515 in.)

- A_{nt} = 1.23 in.^{2}
- U_{bs} = 1 (as the tension stress is uniform)
- $R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$
- R_n = 0.60·(58 kips/in.^{2})·(3.02 in.^{2}) + 1·(58 kips/in.^{2})·(4.38 in.^{2}) ≤ 0.60·(36 kips/in.^{2})·(4.38 in.^{2}) + 1·(58 kips/in.^{2})·(1.23 in.^{2})
- R_n = 359.14 kips ≤ **165.95 kips**
- ϕR_n = 0.75·(165.95 kips)
- ϕR_n **= 124.46 kips > 8.51 kips OK**

where:

- U_{bs} = 1 – stress index for uniform tension stress
- A_{nt} = 1.23 in.^{2} – net area subject to tension
- A_{gv} = 4.38 in.^{2} – gross area subject to shear
- A_{nv} = 3.02 in.^{2} – net area subject to shear
- L_{ev} = 1.75 in. – vertical edge distance of angle
- L_s = 2.5 in. – width of angle
- t = 0.515 in. – thickness of the beam flange
- ϕ = 0.75 – strength factor (LRFD)

E.6.3.4 Flexural strength

$M_n = S_x \cdot F_u \cdot A_{fn}/A_{fg}$ when $F_u \cdot A_{fn} < Y_t \cdot F_y \cdot A_{fg}$ (AISC 360-16, Eq. F13-1)

Otherwise, the limit state of tensile rupture does not apply

- $A_{fg} = (b_f) \cdot (t_f)$
- A_{fg} = (6.77 in.)·(0.515 in.) = 3.49 in.^{2}
- $A_{fn} = A_{fg}$ - n·(d_h + 1/16 in.)·(t_p)
- A_{fn} = 3.49 in.^{2} - 2·(0.813 in. + 1/16 in.)·(0.515 in.) = 2.59 in.^{2}
- F_y/F_u = (36 ksi)/(58 ksi) = 0.62 < 0.8 **therefore Y_t = 1.0**
- $F_u \cdot A_{fn}$ = (58 ksi)·(2.59 in.^{2}) = 150.22 kips
- $Y_t \cdot F_y \cdot A_{fg}$ = 1.0·(36 ksi)·(3.49 in.^{2}) = 125.64 kips
- $F_u \cdot A_{fn}$ = 150.22 kips < $Y_t \cdot F_y \cdot A_{fg}$ = 125.64 kips **NOT OK**
- $\phi M_n = \phi \cdot F_y \cdot S_x$ = 0.90·(36 ksi)·(61.5 in.^{3}) / 12
- ϕM_n = **166.05 kips-ft > 10 kips-ft OK**

where:

- A_{fg} = 3.49 in.^{2} – gross area of tension flange (Section B4.3a)
- A_{fn} = 2.59 in.^{2} – net area of tension flange (Section B4.3b)
- F_u = 58 ksi – specified minimum tensile strength
- F_y = 36 ksi – specified minimum yield strength
- Y_t = 1.0 – beam flexural factor (= 1.0 for $F_y/F_u \leq 0.8$, otherwise = 1.1)

- b_f = 6.77 in. – beam flange width
- t_f = 0.515 in. – beam flange thickness
- S_x = 61.5 in.^{3} – minimum elastic section modulus of beam
- ϕ = 0.9 – strength factor (LRFD)

E.6.4 Column

E.6.4.1 Web panel zone shear

$R_n = 0.60 \cdot F_y \cdot d_c \cdot t_w$ for $P_r < 0.40 \cdot P_c$ (AISC 360-16, Eq. J10-9)

- $P_y = F_y \cdot A_g$ = (36 ksi)·(28.2 in.^{2}) = 1015.2 kips
- $P_r = 0 < 0.40 \cdot P_y$ = 0.40·(1,015.2 kips) = 406.08 kips
- $R_n = 0.60 \cdot F_y \cdot d_c \cdot t_w$
- R_n = 0.60·(36 ksi)·(12.7 in.)·(0.55 in.) = 150.88 kips
- ϕR_n = 0.9·(150.88 kips) = **135.79 kips > 7.15 kips OK**

where:

- A_g = 28.2 in.^{2} – column cross-sectional area
- P_r = 0 kips – required axial strength
- P_y = 1015.2 kips – axial yield strength of the column
- d_c = 12.7 in. – depth of column
- t_w = 0.55 in. – thickness of column web
- F_y = 36 ksi – specified minimum yield strength
- ϕ = 0.9 – strength factor (LRFD)

E.6.4.2 Flange local bending

$R_n = 6.25 \cdot (F_{yf}) \cdot (t_f)^2$ (AISC 360-16, Eq. J10-1)

- R_n = 6.25·(36 ksi)·(0.9 in.^{2})^2
- R_n = 182.25 kips
- ϕR_n = (0.9)·182.25 kips
- ϕR_n =**164.03 kips > 7.15 kips OK**

where:

- F_{yf} = 36 ksi – specified minimum yield stress of the flange
- t_f = 0.9 in.^{2} – thickness of the column flange
- ϕ = 0.9 – strength factor (LRFD)

E.6.4.3 Web local yielding

$R_n = (F_{yw}) \cdot (t_w) \cdot (5 \cdot k + l_b)$ (AISC 360-16, Eq. J10-2)

- R_n = (36 ksi)·(0.55 in.)·(5·(1.5 in.) + 0.75 in.)
- R_n =163.35 kips
- ϕR_n = (1.0)·163.35 kips = **163.35 kips > 7.15 kips OK**

where:

- F_{yw} = 36 ksi – specified minimum yield strength of the web material
- t_w = 0.55 in.^{2} – thickness of column web
- k = 1.5 in. – distance from outer face of the flange to the web toe of the fillet
- l_b = 0.75 in. – length of bearing (not less than k for end beam reactions)
- ϕ = 1.0 – strength factor (LRFD)

E.6.4.4 Web local crippling

$R_n = 0.80\cdot(t_w)^2\cdot[1+3\cdot(l_b/d)(t_w/t_f)^{1.5}]\cdot[E\cdot(F_{yw})\cdot(t_f)/(t_w)]^{1/2}\cdot Q_f$ (AISC 360-16, Eq. J10-4)

- R_n = 0.80·(0.55 in.)^2·[1+3·(0.9 in.^{2}/12.7 in.)(0.55 in.^{2}/0.9 in.^{2})$^{1.5}$]·[2.9E+7·(36 ksi)·(0.9 in.^{2})/(0.55)]$^{1/2}$·(1.0) = 343.08 kips
- ϕR_n = 0.75·(343.08 kips) = **257.31 kips > 7.15 kips OK**

where:

- F_{yw} = 36 ksi – specified minimum yield strength of the web material
- t_w = 0.55 in.^{2} – thickness of column web
- t_f = 0.9 in.^{2} – thickness of column flange
- l_b = 0.75 in. – length of bearing (not less than k for end beam reactions)
- d = 12.7 in. – full nominal depth of the member
- ϕ = 0.75 – strength factor (LRFD)
- E = 2.9E+7 – Elastic Modulus
- Q_f = 1.0 – 1.0 for wide-flange sections and for HSS (connecting surface) in tension = as given in Table K3.2 (AISC 360-16) for all other HSS conditions

Appendix F. ASD Strength Calculations for Semi-rigid Connection Test Specimen (Test No. 14S1)

F.1 Loading and Assumptions

It is assumed that the double web angle carries the applied shear force, and the top and seat angles resist the applied moment.

Vertical shear force:

P_D = 10 kips (dead load)

P_L = 30 kips (live load)

Strong-axis moment:

The shear loads are applied 2 in. away from the boltline.

M_D = (10 kips)·(2 in.) = 20 kips-in.

M_L = (30 kips)·(2 in.) = 60 kips-in.

F.1.1 ASD Load Demand or Design Loads

According to the Load and Resistance Factored Design (ASD) procedure (AISC 360-16 or AISC Manual 2017), ASD design load, factored load or demand, P_u under gravity loads is:

$P_a = P_D + P_L$ (ASD)

P_u = 10 kips + 30 kips = 40 kips (ASD)

$M_u = M_D + M_L$ (ASD)

M_u = 20 kips + 60 kips = 80 kips-in. = 6.67 kips-ft (ASD)

Then, the required strength, R_u using the ASD load combinations will be set equal to the design shear force, V_u applied on the connection. Then, for the cantilever beam and connection specimen:

$R_u = V_u = P_u = P_D + P_L$ (ASD)

F.2. Double Web-Angle ASD Design Checks

F.2.1 Angle (Beam Side)

F.2.1.1 Bolt shear

$R_n = F_n \cdot A_b$ (Eq. J3-1, AISC 360-16)

- Shear strength of one bolt:
- R_n/Ω = 68·0.442 / 2 = **15.03 kips/bolt**
- Total shear strength of three bolts double angle:
- R_n/Ω = 2 x 3 x 22.54 kips = **90.18 kips > 40 kips OK**

where:

- F_{nv} = 68 ksi – nominal shear strength of fasteners (Table J3.2, AISC 360-16)
- A_b = 0.442 in.^{2} – nominal bolt area
- Ω = 2 – safety factor (ASD)

F.2.1.2 Bolt bearing

The nominal bearing strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 2.4 \cdot d \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6a)

- R_n =2.4·(3/4 in.)·(0.25 in.)·(58 ksi)
- R_n = 26.1 kips/bolt
- R_n/Ω = (2 angles)·(3 bolts)·(26.1 kips) / 2
- R_n/Ω = **78.30 kips/connection > 40 kips OK**

where:

- d = 3/4 in. – nominal bolt diameter
- t = 0.25 in. – thickness of angle
- F_u = 58 ksi – specified minimum tensile stress of angle
- Ω = 2 – safety factor (ASD)

F.2.1.3 Bolt tearout

The available tearout strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 1.2 \cdot l_c \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6c)

- $L_{csv} = L_{sv} - d_h$
- L_{csv} = 3 in. - 13/16 in. = 2.19 in.
- $R_{n\text{-}span}$ = 1.2·(2.19 in.)·(0.25 in.)·(58 ksi)
- $R_{n\text{-}span}$ = 38.1 kips/bolt
- $L_{cev} = L_{ev} - d_h/2$
- L_{cev} = 1.5 in. - (13/16)/2 in. = 1.09 in.
- $R_{n\text{-}end}$ = 1.2·(1.09 in.)·(0.25 in.)·(58 ksi)
- $R_{n\text{-}end}$ = 18.97 kips/bolt

- R_n/Ω = (2 angles)·[2·(38.1 kips) + (18.97 kips)] / 2
- R_n/Ω = **95.15 kips/conn. > 40 kips OK**

where:

- l_c : clear distance
- d_h = 13/16 in. – nominal hole dimension (3/4-in.-diameter bolt)
- L_{sv} = 3 in. – vertical center to center bolt distance
- L_{ev} = 1.5 in. – vertical edge distance of angle
- L_{csv} = 2.19 in. – clear vertical distance between bolts
- L_{cev} = 1.09 in. – clear vertical edge distance
- Ω = 2 – safety factor (ASD)

F.2.1.4 Shear yielding

$R_n = 0.60 \cdot F_y \cdot A_{gv}$ (AISC 360-16, Eq. J4-3)

- A_{gv} = (8.5 in.)·(0.25 in.) = 2.125 in.^{2}
- R_n/Ω = (2 angles) 0.60·(36 kips)·(2.125 in.^{2}) / 1.5
- R_n/Ω = **61.20 kips > 40 kips OK**

where:

- A_{gv} = 2.125 in.^{2} – gross area subject to shear
- L = 8.5 in. – length of angle
- t = 0.25 in. – thickness of angle
- F_y = 36 ksi – specified minimum yield strength
- Ω = 1.5 – safety factor (ASD)

F.2.1.5 Shear rupture

$R_n = 0.60 \cdot F_u \cdot A_{nv}$ (AISC 360-16, Eq. J4-4)

- $L_h = d_h + 1/16$ (Section B4-3b, AISC Manual)
- L_h = 0.813 + 1/16 = 0.875 in.
- $L_e = L - n \cdot L_h$
- L_e = 8.5 - 3·0.875 = 5.875 in.
- $A_{nv} = L_e \cdot t_p$
- A_{nv} = (5.875 in.)·(0.25 in.) = 1.47 in.^{2}
- R_n/Ω = (2 angles)·0.60·(58 kips)·(1.47 in.^{2}) / 2
- R_n/Ω = **51.16 kips > 40 kips OK**

where:

- A_{nv} = 1.47 in.^{2} – net area subject to shear, in.^{2}

- L_h = 0.875 in. – hole dimension for tension and shear net area
- L_e = 5.875 in. – effective length
- n = 3 – number of the bolts
- t = 0.25 in. – thickness of angle
- Ω = 2 – safety factor (ASD)

F.2.1.6 Block shear

$R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$ (AISC 360-16, Eq. J4-5)

- $A_{gv} = (L\text{-}L_{ev}) \cdot (t_p)$
- A_{gv} = (2 angles)·(8.5 in. - 1.25 in.)·(0.25 in.) = 3.63 in.^{2}
- $A_{nv} = A_{gv}$ - (2 angles)·(n-0.5)·(d_h + 1/16 in.)·(t_p)
- A_{nv} = 3.63 in.^{2} - (2 angles)·(3-0.5)·(0.813 + 1/16 in.)·(0.25 in.)
- A_{nv} = 2.54 in.^{2}
- A_{nt} = (2 angles)·[L_{eh} - 0.5·(d_h + 1/16 in.)]·(t_p)
- A_{nt} = (2 angles)·[1.25 in. - 0.5·(0.813 in. + 1/16 in.)]·(0.25 in.)
- A_{nt} = 0.41 in.^{2}
- U_{bs} = 1 (as the tension stress is uniform)
- $R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$
- R_n = 0.60·(58 kips/in.^{2})·(2.54 in.^{2}) + 1·(58 kips/in.^{2})·(0.41 in.^{2}) ≤ 0.60·(36 kips/in.^{2})·(3.63 in.^{2}) + 1·(58 kips/in.^{2})·(0.41 in.^{2})
- R_n = 112.17 kips ≤ **102.19 kips**
- R_n/Ω = (111.74 kips)/2
- R_n/Ω **= 55.87 kips > 40 kips OK**

where:

- U_{bs} = 1 – Stress index for uniform tension stress
- A_{nt} = 0.41 in.^{2} – net area subject to tension
- A_{gv} = 3.63 in.^{2} – gross area subject to shear
- A_{nv} = 2.54 in.^{2} – net area subject to shear
- L_{ev} = 1.25 in. – vertical edge distance of angle
- L_{eh} = 1.25 in. – horizontal edge distance of angle
- Ω = 2 – safety factor (ASD)

F.2.2 Angle (Column Side)

F.2.2.1 Bolt shear

$R_n = F_n \cdot A_b$ (Eq. J3-1, AISC 360-16)

- Shear strength of one bolt:
- R_n/Ω = 68·0.442 / 2 = **15.03 kips/bolt**

- Total shear strength of three bolts double angle:
- R_n/Ω = 2 x 3 x 15.03 kips = **90.18 kips > 40 kips OK**

where:

- F_{nv} = 68 ksi – nominal shear strength of fasteners (Table J3.2, AISC 360-16)
- A_b = 0.442 in.^{2} – nominal bolt area
- Ω = 2 – safety factor (ASD)

F.2.2.2 Bolt bearing

The nominal bearing strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 2.4 \cdot d \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6a)

- R_n =2.4·(3/4 in.)·(0.25 in.)·(58 ksi)
- R_n = 26.1 kips/bolt
- R_n/Ω = (2 angles)·(3 bolts)·(26.1 kips) / 2
- R_n/Ω = **78.30 kips/connection > 40 kips OK**

where:

- d = 3/4 in. – nominal bolt diameter
- t = 0.25 in. – thickness of angle
- F_u = 58 ksi – specified minimum tensile stress of angle
- Ω = 2 – safety factor (ASD)

F.2.3.3 Bolt tearout

The available tearout strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 1.2 \cdot l_c \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6c)

- $L_{csv} = L_{sv} - d_h$
- L_{csv} = 3 in. - 13/16 in. = 2.19 in.
- $R_{n\text{-}span}$ = 1.2·(2.19 in.)·(0.25 in.)·(58 ksi)
- $R_{n\text{-}span}$ = 38.1 kips/bolt
- $L_{cev} = L_{ev} - d_h/2$
- L_{cev} = 1.5 in. - (13/16)/2 in. = 1.09 in.
- $R_{n\text{-}end}$ = 1.2·(1.09 in.)·(0.25 in.)·(58 ksi)
- $R_{n\text{-}end}$ = 18.97 kips/bolt
- R_n/Ω = (2 angles)·[2·(38.1 kips) + (18.97 kips)] / 2 = **95.15 kips/conn. > 40 kips OK**

where:

- l_c : clear distance
- d_h = 13/16 in. – nominal hole dimension (3/4-in.-diameter bolt)
- L_{sv} = 3 in. – vertical center to center bolt distance
- L_{ev} = 1.5 in. – vertical edge distance of angle
- L_{csv} = 2.19 in. – clear vertical distance between bolts
- L_{cev} = 1.09 in. – clear vertical edge distance
- $\Omega = 2$ – safety factor (ASD)

F.2.4.4 Shear yielding

$R_n = 0.60 \cdot F_y \cdot A_{gv}$ (AISC 360-16, Eq. J4-3)

- A_{gv} = (8.5 in.)·(0.25 in.) = 2.125 in.^{2}
- R_n/Ω = (2 angles) 0.60·(36 kips)·(2.125 in.^{2}) / 1.5
- R_n/Ω = **61.2 kips > 40 kips OK**

where:

- A_{gv} = 2.125 in.^{2} – gross area subject to shear
- L = 8.5 in. – length of angle
- t = 0.25 in. – thickness of angle
- F_y = 36 ksi – specified minimum yield strength
- $\Omega = 1.5$ – safety factor (ASD)

F.2.5.5 Shear rupture

$R_n = 0.60 \cdot F_u \cdot A_{nv}$ (AISC 360-16, Eq. J4-4)

- $L_h = d_h + 1/16$ (Section B4-3b, AISC Manual)
- L_h = 0.813 + 1/16 = 0.875 in.
- $L_e = L - n \cdot L_h$
- L_e = 8.5 - 3·0.875 = 5.875 in.
- $A_{nv} = L_e \cdot t_p$
- A_{nv} = (5.875 in.)·(0.25 in.) = 1.47 in.^{2}
- R_n/Ω = (2 angles)·0.60·(58 kips)·(1.47 in.^{2}) / 2
- R_n/Ω = **51.16 kips > 40 kips OK**

where:

- A_{nv} = 1.47 in.^{2} – net area subject to shear, in.^{2}
- L_h = 0.875 in. – hole dimension for tension and shear net area
- L_e = 5.875 in. – effective length
- $n = 3$ – number of the bolts
- t = 0.25 in. – thickness of angle

- $\Omega = 2$ – safety factor (ASD)

F.2.6.6 Block shear

$R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$ (AISC 360-16, Eq. J4-5)

- $A_{gv} = (L - L_{ev}) \cdot (t_p)$
- A_{gv} = (2 angles)·(8.5 in. - 1.25 in.)·(0.25 in.) = 3.63 in.^{2}
- $A_{nv} = A_{gv}$ - (2 angles)·(n-0.5)·(d_h + 1/16 in.)·(t_p)
- A_{nv} = 3.63 in.^{2} - (2 angles)·(3-0.5)·(0.813 + 1/16 in.)·(0.25 in.)
- A_{nv} = 2.54 in.^{2}
- A_{nt} = (2 angles)·[L_{eh} - 0.5·(d_h + 1/16 in.)]·(t_p)
- A_{nt} = (2 angles)·[1.25 in. - 0.5·(0.813 in. + 1/16 in.)]·(0.25 in.)
- A_{nt} = 0.41 in.^{2}
- U_{bs} = 1 (as the tension stress is uniform)
- $R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$
- R_n = 0.60·(58 kips/in.^{2})·(2.54 in.^{2}) + 1·(58 kips/in.^{2})·(0.41 in.^{2}) ≤ 0.60·(36 kips/in.^{2})·(3.63 in.^{2}) + 1·(58 kips/in.^{2})·(0.41 in.^{2})
- R_n = 112.17 kips ≤ **102.19 kips**
- R_n/Ω = (111.74 kips) / 2
- R_n/Ω **= 55.87 kips > 40 kips OK**

where:

- U_{bs} = 1 – stress index for uniform tension stress
- A_{nt} = 0.41 in.^{2} – net area subject to tension
- A_{gv} = 3.63 in.^{2} – gross area subject to shear
- A_{nv} = 2.54 in.^{2} – net area subject to shear
- L_{ev} = 1.25 in. – vertical edge distance of angle
- L_{eh} = 1.25 in. – horizontal edge distance of angle
- $\Omega = 2$ – safety factor (ASD)

F.2.3 Beam

F.2.3.1 Bolt bearing

$r_n = 2.4 \cdot d \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6a)

- r_n =2.4·(3/4 in.)·(0.31 in.)·(58 ksi)
- r_n = 32.36 kips/bolt
- R_n/Ω = (3 bolts)·(32.36 kips) / 2
- R_n/Ω = **48.54 kips/connection > 40 kips OK**

where:

- d = 3/4 in. – nominal bolt diameter
- t = 0.31 in. – thickness of the beam
- F_u = 58 ksi – specified minimum tensile stress of beam
- Ω = 2 – safety factor (ASD)

F.2.3.2 Bolt tearout

The available tearout strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 1.2 \cdot l_c \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6c)

- L_{csv} = 3 in. - 13/16 in. = 2.19 in.
- L_{cev} = [14.1 in. - 2·(3 in.) - (13/16) in.]/2 = 3.64 in.
- $R_{n\text{-}span}$ = 1.2·(2.19 in.)·(0.31 in.)·(58 ksi)
- $R_{n\text{-}span}$ = 47.25 kips/bolt
- $R_{n\text{-}end}$ = 1.2·(3.64 in.)·(0.31 in.)·(58 ksi)
- $R_{n\text{-}end}$ = 78.53 kips/bolt
- R_n/Ω = [2·(47.25 kips) + (78.53 kips)] / 2 = **86.52 kips/connection > 40 kips OK**

where:

- l_c : clear distance
- d_h = 13/16 in. – nominal hole dimension (3/4-in.-diameter bolt)
- L_{sv} = 3 in. – vertical center to center bolt distance
- L_{csv} = 2.19 in. – clear vertical distance between bolts
- L_{cev} = 3.64 in. – clear vertical edge distance
- Ω = 2 – safety factor (ASD)

F.2.3.3 Shear yielding

$R_n = 0.60 \cdot F_y \cdot A_{gv}$ (AISC 360-16, Eq. J4-3)

- A_{gv} = (14.1 in.)·(0.31 in.) = 4.37 in.^{2}
- R_n/Ω = 0.60·(36 kips)·(4.37 in.^{2}) / 1.5
- R_n/Ω = **62.93 kips > 40 kips OK**

where:

- A_{gv} = 4.37 in.^{2} – gross area subject to shear
- F_y = 36 ksi – specified minimum yield strength
- Ω = 1.5 – safety factor (ASD)

F.2.4 Column

F.2.4.1 Bolt bearing

$r_n = 2.4 \cdot d \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6a)

- r_n =2.4·(3/4 in.)·(0.9 in.)·(58 ksi)
- r_n = 93.96 kips/bolt
- R_n/Ω = (3 bolts)·(93.96 kips) / 2
- R_n/Ω = **140.94 kips/connection > 40 kips OK**

where:

- d = 3/4 in. – nominal bolt diameter
- t = 0.9 in. – thickness of column web
- F_u = 58 ksi – specified minimum tensile stress of beam
- Ω = 2 – safety factor (ASD)

F.3 Top- and Seat-Angle ASD Design Checks

F.3.1 Angle (Beam Side)

F.3.1.1 Flange Bolt Shear Force

$P_{uf} = M_u/d_m$

- P_{uf} = 80 kips-in. / 14.1 in. = **5.67 kips**

where:

- d_m = 14.1 in. – depth of beam
- P_{uf} = 5.67 kips – flange bolt shear

F.3.1.2 Flange Tension Force

The moment arm between flange forces is equal to the summation of gage in leg on column flange, beam depth and a half of the thickness of seated-angle.

$P_{uf} = M_u/(d + g + t/2)$

- P_{uf} = 80 kips-in. / (14.1 in. + 2.5 in. + 0.375 in./2) = **4.15 kips**

where:

- g = 2.5 in – gage in leg on column flange
- d_m = 14.1 in – depth of beam
- t = 0.375 in. – plate thickness
- P_{uf} = 11.49 kips – flange tension force

F.3.1.3 Tension yielding

$R_n = F_y \cdot A_g$ (AISC 360-16, Eq. J4-1)

where:

- A_g = (8 in.)·(0.375 in.) = 3 in.^{2}

- R_n = (36 ksi)·3 = 108 kips
- R_n/Ω = (108 ksi) / 1.67 = **64.67 kips > 4.15 kips**

where:

- A_g =3 in.^{2} – gross area
- F_y = 36 ksi – specified minimum yield strength
- Ω = 1.67 – safety factor (ASD)

F.3.1.4 Tension rupture

$R_n = F_u \cdot A_e$ (AISC 360-16, Eq. J4-2)

- A_g = (8 in.)·(0.375 in.) = 3 in.^{2}
- $L_h = d_h$ + 1/16 (Section B4-3b, AISC Manual)
- L_h = 0.813 + 1/16 = 0.875 in.
- $A_n = (L - n \cdot L_h) \cdot t_p$
- A_n = (8 in. - 2·0.875 in.)·(0.375 in.) = 2.34 in.^{2}
- $A_e = A_n \leq 0.85A_g$ (AISC 360-16, Sec. J4-1)
- A_e = 2.34 in.^{2}
- R_n = (58 ksi)·(2.34 in.^{2}) = 135.7 kips
- R_n/Ω = (135.7 ksi) / 2 = **67.85 kips > 4.15 kips**

where:

- A_g =3 in.^{2} – gross area
- A_e = 2.34 in.^{2} – effective net area
- L_h = 0.875 in. – hole dimension
- F_u = 58 ksi – specified minimum tensile stress of beam
- Ω = 2 – safety factor (ASD)

F.3.1.5 Compression

$P_n = F_y \cdot A_g$ when $L_c/r \leq 25$ (AISC 360-16, Sec. J4-4)

- $r = t_p/(12)^{1/2}$ = 0.375 in./$(12)^{1/2}$
- r = 0.108 in.
- $L_c = K \cdot L/r$ = 0.65·4.75 in./0.108 in.
- L_c = 28.6 > 25 **OK**
- $F_e = \pi^2 \cdot E/(L_c/r)^2$ (AISC 360-16, Eq. E3-4)
- $F_e = \pi^2$·2.90E+7 psi / (28.6/0.108 in.)^2 = 3.52E+05 psi
- $F_{cr} = 0.658^{(Fy/Fe)} \cdot F_y$ when $L_c/r \leq 4.71 \cdot (E/F_y)^{1/2}$ (AISC 360-16, Eq. E3-2)
- 28.6/0.108 in. ≤ 4.71·(2.90E+7 psi / 36,000 psi)$^{1/2}$ **OK**
- F_{cr} = 0.658(36000 /3.52E+05)·36,000 psi

- F_{cr} = 34,490.89 psi
- $A_g = L_p \cdot t_p$ = 8 in.·0.375 in.
- A_g = 3 in.^{2}
- $R_n/\Omega = F_{cr} \cdot A_g/\Omega$ = (34,490.89 psi)·(3 in.^{2}) / 1.67
- R_n/Ω = **61.96 kips > 4.15 kips**

where:

- K = 0.65 – effective length factor
- L = 4.75 in. – laterally unbraced length of the member
- L_c = 28.6 – effective length
- F_e = 3.52E+05 psi – elastic buckling stress
- F_{cr} = 34,490.89 psi – critical stress
- A_g = 3 in.^{2} – gross area
- Ω = 1.67 – safety factor (ASD)

F.3.1.6 Bolt shear

$R_n = F_n \cdot A_b$ (Eq. J3-1, AISC 360-16)

- Shear strength of one bolt:
- R_n/Ω = 68·0.442 / 2 = 15.03 kips/bolt
- Total shear strength of four bolts:
- R_n/Ω = 4 x 15.03 kips = **60.12 kips > 5.67 kips**

where:

- F_{nv} = 68 ksi – nominal shear strength of fasteners (Table J3.2, AISC 360-16)
- A_b = 0.442 in.^{2} – nominal bolt area
- Ω = 2 – safety factor (ASD)

F.3.1.7 Bolt bearing

The nominal bearing strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 2.4 \cdot d \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6a)

- R_n =2.4·(3/4 in.)·(0.375 in.)·(58 ksi)
- R_n = 39.15 kips/bolt
- R_n/Ω = (4 bolts)·(39.15 kips) / 2
- R_n/Ω = **78.30 kips/connection > 5.67 kips**

where:

- d = 3/4 in. – nominal bolt diameter

- t = 0.375 in. – thickness of angle
- F_u = 58 ksi – specified minimum tensile stress of angle
- Ω = 2 – safety factor (ASD)

F.3.1.8 Bolt Tearout

The available tearout strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 1.2 \cdot l_c \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6c)

- $L_{csv} = L_{sv}$ - d_h
- L_{csv} = 2.5 in. - 13/16 in. = 1.69 in.
- $R_{n\text{-}span}$ = 1.2·(1.69 in.)·(0.375 in.)·(58 ksi)
- $R_{n\text{-}span}$ = 44.1 kips/bolt
- $L_{cev} = L_{ev}$ - $d_h/2$
- L_{cev} = 1.25 in. - (13/16)/2 in. = 0.84 in.
- $R_{n\text{-}end}$ = 1.2·(0.84 in.)·(0.375 in.)·(58 ksi)
- $R_{n\text{-}end}$ = 21.92 kips/bolt
- R_n/Ω = 2·[(44.1 kips) + (21.92 kips)] / 2 = **66.02 kips/connection > 5.67 kips**

where:

- l_c : clear distance
- d_h = 13/16 in. – nominal hole dimension (3/4-in.-diameter bolt)
- L_{sv} = 2.5 in. – vertical center to center bolt distance
- L_{ev} = 1.25 in. – vertical edge distance of angle
- L_{csv} = 1.69 in. – clear vertical distance between bolts
- L_{cev} = 0.84 in. – clear vertical edge distance
- Ω = 2 – safety factor (ASD)

F.3.1.9 Block shear

$R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$ (AISC 360-16, Eq. J4-5)

- $A_{gv} = (L\text{-}L_{ev}) \cdot (t_p)$
- A_{gv} = 2·(2.5 in. + 1.25 in.)·(0.375 in.) = 2.81 in.^{2}
- $A_{nv} = A_{gv}$ - 2·(n-0.5)·(d_h + 1/16 in.)·(t_p)
- A_{nv} = 2.81 in.^{2} - 2·(2-0.5)·(0.813 + 1/16 in.)·(0.375 in.)
- A_{nv} = 1.83 in.^{2}
- A_{nt} = 2·[L_{eh} - 0.5·(d_h + 1/16 in.)]·(t_p)
- A_{nt} = 2·[1.25 in. - 0.5·(0.813 in. + 1/16 in.)]·(0.375 in.)
- A_{nt} = 0.61 in.^{2}
- U_{bs} = 1 (as the tension stress is uniform)

- $R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$
- $R_n = 0.60 \cdot (58 \text{ kips/in.}^2) \cdot (1.83 \text{ in.}^2) + 1 \cdot (58 \text{ kips/in.}^2) \cdot (0.61 \text{ in.}^2) \leq 0.60 \cdot (36 \text{ kips/in.}^2) \cdot (2.81 \text{ in.}^2) + 1 \cdot (58 \text{ kips/in.}^2) \cdot (0.61 \text{ in.}^2)$
- R_n = 99.06 kips ≤ **96.08 kips**
- R_n/Ω = (96.08 kips)/2
- R_n/Ω **= 48.04 kips > 4.15 kips**

where:

- $U_{bs} = 1$ – stress index for uniform tension stress
- $A_{nt} = 0.61$ in.^{2} – net area subject to tension
- $A_{gv} = 2.81$ in.^{2} – gross area subject to shear
- $A_{nv} = 1.83$ in.^{2} – net area subject to shear
- $L_{ev} = 1.25$ in. – vertical edge distance of angle
- $L_{eh} = 1.25$ in. – horizontal edge distance of angle
- $\Omega = 2$ – safety factor (ASD)

F.3.2 Angle (Column Side)

F.3.2.1 Shear yielding

$R_n = 0.60 \cdot F_y \cdot A_{gv}$ (AISC 360-16, Eq. J4-3)

- A_{gv} = (8 in.)·(0.375 in.) = 3 in.^{2}
- R_n/Ω = 0.60·(36 kips)·(3 in.^{2}) / 1.5
- R_n/Ω = **43.2 kips > 4.15 kips**

where:

- $A_{gv} = 3$ in.^{2} – gross area subject to shear
- $L = 8$ in. – length of angle
- $t = 0.375$ in. – thickness of angle
- $\Omega = 1.5$ – safety factor (ASD)

F.3.2.2 Shear rupture

$R_n = 0.60 \cdot F_u \cdot A_{nv}$ (AISC 360-16, Eq. J4-4)

- $L_h = d_h + 1/16$ (Section B4-3b, AISC Manual)
- L_h = 0.813 + 1/16 = 0.875 in.
- $L_e = L - n \cdot L_h$
- L_e = 8 - 2·0.875 = 6.25 in.
- $A_{nv} = L_e \cdot t_p$
- A_{nv} = (6.25 in.)·(0.375 in.) = 2.34 in.^{2}
- R_n/Ω = 0.60·(58 kips)·(2.34 in.^{2}) / 2

- R_n/Ω = **40.72 kips > 4.15 kips**

where:

- A_{nv} = 2.34 in.^{2} – net area subject to shear, in.^{2}
- L_h = 0.875 in. – hole dimension for tension and shear net area
- L_e = 6.25 in. – effective length
- n = 2 – number of the bolts
- t = 0.25 in. – thickness of angle
- Ω = 2 – safety factor (ASD)

F.3.2.3 Tension capacity due prying action

$T_c = B_c \cdot Q$ (AISC Manual, Eq. 9-27)

$a' = (a + d/2) \leq (1.25 \cdot b + d/2)$

a' = (1.5 in. + 0.75 in./2) ≤ (1.25·(2.31 in.) + 0.75 in./2)

a' = **1.875 in.** ≤ 3.26 in.

a' = 1.875 in.

$b' = b - d/2$ = 2.31 in. – (0.75 in.)/2

b' = 1.94 in.

$\rho = b'/a'$ = (1.94 in.)/(1.875 in.)

ρ = 1.03

$p = L/n$ = (8 in.)/2 = 4 in.

$\delta = 1 - d'/p$ = 1 - (0.813 in.)/(4 in.)

δ = 0.80

$B_c = R_n/\Omega = F_{nt} \cdot A_b/\Omega$

B_c = (90 ksi)·(0.442 in.^{2}) / 2

B_c = 19.89 kips (per bolt)

$t_c = ((4 \cdot B \cdot b')/(p \cdot F_u/\Omega))^{1/2}$

t_c = [4·(19.89 kips)·(1.938 in.)/((4 in.)·(58 ksi)/1.67)]$^{1/2}$

t_c = 1.05 in.

$\alpha' = (1/(\delta \cdot (1 + \rho))) \cdot ((t_c/t_p)^2 - 1)$

$\alpha' = (1/(0.80 \cdot (1 + 1.03))) \cdot ((1.05 \text{ in.})/(0.375 \text{ in.})^2 - 1)$

$\alpha' = 4.25$

$Q = (t_p/t_c)^2 \cdot (1 + \delta)$

$Q = ((0.375 \text{ in.})/(1.05 \text{ in.}))^2 \cdot (1 + 0.8)$

$Q = 0.23$

$\Omega T_c = (2 \text{ bolts}) \cdot B_c \cdot Q = 2 \cdot (19.89 \text{ kips}) \cdot (0.23)$

ΩT_c = **9.15 kips > 4.15 kips (Governs)**

where:

- a =1.5 in. – distance from the bolt centerline to the edge of the fitting
- a' = 1.875 in. – distance for prying action
- b' = 1.94 in. – distance for prying action
- ρ = 1.03 – prying distances ratio
- δ = 0.80 – ratio of the net length at bolt line to gross length at the face of the stem or leg of angle
- p = 4 in. – average pitch of the bolts
- t_c = 1.05 in. – flange thickness
- t_p = 0.375 in. – thickness of angle
- B_c = 29.84 kips – available tensile strength per bolt
- Q = 0.23 – prying action coefficient
- Ω = 2 – safety factor (ASD)

F.3.3 Beam

F.3.3.1 Bolt bearing

$r_n = 2.4 \cdot d \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6a)

- $r_n = 2.4 \cdot (3/4 \text{ in.}) \cdot (0.515 \text{ in.}) \cdot (58 \text{ ksi})$
- r_n = 53.77 kips/bolt
- R_n/Ω = (3 bolts)·(53.77 kips) / 2
- R_n/Ω = **80.66 kips/connection > 5.67 kips OK**

where:

- d = 3/4 in. – nominal bolt diameter
- t = 0.515 in. – thickness of the beam flange

- F_u = 58 ksi – specified minimum tensile stress of beam
- $\Omega = 2$ – safety factor (ASD)

F.3.3.2 Bolt tearout

The available tearout strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 1.2 \cdot l_c \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6c)

- L_{csv} = 2.5 in. - 13/16 in. = 1.69 in.
- L_{cev} = [8.5 in. - 2·(2.5 in.) - (13/16) in.]/2 = 1.34 in.
- $R_{n\text{-}span}$ = 1.2·(1.69 in.)·(0.515 in.)·(58 ksi)
- $R_{n\text{-}span}$ = 60.58 kips/bolt
- $R_{n\text{-}end}$ = 1.2·(1.34 in.)·(0.515 in.)·(58 ksi)
- $R_{n\text{-}end}$ = 47.10 kips/bolt
- R_n/Ω = 2·[·(60.58 kips) + (47.10 kips)] / 2 = **107.68 kips/connection > 5.67 kips OK**

where:

- l_c : clear distance
- d_h = 13/16 in. – nominal hole dimension (3/4-in.-diameter bolt)
- L_{sv} = 2.5 in. – vertical center to center bolt distance
- L_{csv} = 1.69 in. – clear vertical distance between bolts
- L_{cev} = 1.34 in. – clear vertical edge distance
- t = 0.515 in. – thickness of the beam flange
- $\Omega = 2$ – safety factor (ASD)

F.3.3.3 Block shear

$R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$ (AISC 360-16, Eq. J4-5)

- $A_{gv} = 2 \cdot (L_s - L_{ev}) \cdot (t_p)$
- A_{gv} = 2·(2.5 in. + 1.75 in.)·(0.515 in.) = 4.38 in.^{2}
- $A_{nv} = 2 \cdot (t_p) \cdot [\, L_{ev} + (n-1) \cdot (L_s - (d_h + 1/16 \text{ in.})) - (d_h + 1/16 \text{ in.})/2)]$
- A_{nv} = 2·(0.515 in.)·(1.75 in. + (2-1)·(2.5 - 0.813 in. - 1/16 in.)-(0.813 in. + 1/16 in.)/2)
- A_{nv} = 3.02 in.^{2}
- $A_{nt} = [b - (n-1) \cdot (\, d_h + 1/16 \text{ in.})] \cdot (t_p)$
- A_{nt} = [3.25 in. - 1·(0.813 in. + 1/16 in.)]·(0.515 in.)
- A_{nt} = 1.23 in.^{2}
- U_{bs} = 1 (as the tension stress is uniform)
- $R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$
- R_n = 0.60·(58 kips/in.^{2})·(3.02 in.^{2}) + 1·(58 kips/in.^{2})·(4.38 in.^{2}) ≤ 0.60·(36 kips/in.^{2})·(4.38 in.^{2}) + 1·(58 kips/in.^{2})·(1.23 in.^{2})

- R_n = 359.14 kips ≤ **165.95 kips**
- R_n/Ω = (165.95 kips) / 2
- R_n/Ω **= 82.98 kips > 5.67 kips OK**

where:

- U_{bs} = 1 – stress index for uniform tension stress
- A_{nt} = 1.23 in.^{2} – net area subject to tension
- A_{gv} = 4.38 in.^{2} – gross area subject to shear
- A_{nv} = 3.02 in.^{2} – net area subject to shear
- L_{ev} = 1.75 in. – vertical edge distance of angle
- L_s = 2.5 in. – width of angle
- t = 0.515 in. – thickness of the beam flange
- Ω = 2 – safety factor (ASD)

F.3.3.4 Flexural strength

$M_n = S_x \cdot F_u \cdot A_{fn}/A_{fg}$ when $F_u \cdot A_{fn} < Y_t \cdot F_y \cdot A_{fg}$ (AISC 360-16, Eq. F13-1)

Otherwise, the limit state of tensile rupture does not apply

- $A_{fg} = (b_f) \cdot (t_f)$
- A_{fg} = (6.77 in.)·(0.515 in.) = 3.49 in.^{2}
- $A_{fn} = A_{fg}$ - n·(d_h + 1/16 in.)·(t_p)
- A_{fn} = 3.49 in.^{2} - 2·(0.813 in. + 1/16 in.)·(0.515 in.) = 2.59 in.^{2}
- F_y/F_u = (36 ksi)/(58 ksi) = 0.62 < 0.8 **therefore Y_t = 1.0**
- $F_u \cdot A_{fn}$ = (58 ksi)·(2.59 in.^{2}) = 150.22 kips
- $Y_t \cdot F_y \cdot A_{fg}$ = 1.0·(36 ksi)·(3.49 in.^{2}) = 125.64 kips
- $F_u \cdot A_{fn}$ = 150.22 kips < $Y_t \cdot F_y \cdot A_{fg}$ = 125.64 kips **NOT OK**
- $M_n/\Omega = F_y \cdot S_x/\Omega$ = (36 ksi)·(61.5 in.^{3}) / 12 / 1.67
- M_n/Ω = **110.48 kips-ft > 6.67 kips-ft OK**

where:

- A_{fg} = 3.49 in.^{2} – gross area of tension flange (Section B4.3a)
- A_{fn} = 2.59 in.^{2} – net area of tension flange (Section B4.3b)
- F_u = 58 ksi – specified minimum tensile strength
- F_y = 36 ksi – specified minimum yield strength
- Y_t = 1.0 – beam flexural factor (= 1.0 for F_y/F_u ≤ 0.8, otherwise = 1.1)
- b_f = 6.77 in. – beam flange width
- t_f = 0.515 in. – beam flange thickness
- S_x = 61.5 in.^{3} – minimum elastic section modulus of beam
- Ω = 1.67 – safety factor (ASD)

F.3.4 Column

F.3.4.1 Web panel zone shear

$R_n = 0.60 \cdot F_y \cdot d_c \cdot t_w$ for $P_r < 0.40 \cdot P_c$ (AISC 360-16, Eq. J10-9)

- $P_y = F_y \cdot A_g$ = (36 ksi)·(28.2 in.^{2}) = 1015.2 kips
- $P_r = 0 < 0.40 \cdot P_y$ = 0.40·(1,015.2 kips) = 406.08 kips
- $R_n = 0.60 \cdot F_y \cdot d_c \cdot t_w$
- R_n = 0.60·(36 ksi)·(12.7 in.)·(0.55 in.) = 150.88 kips
- R_n/Ω = (150.88 kips) / 1.67 = **90.35 kips > 4.15 kips**

where:

- A_g = 28.2 in.^{2} – column cross-sectional area
- P_r = 0 kips – required axial strength
- P_y = 1015.2 kips – axial yield strength of the column
- d_c = 12.7 in. – depth of column
- t_w = 0.55 in. – thickness of column web
- F_y = 36 ksi – specified minimum yield strength
- Ω = 1.67 – safety factor (ASD)

F.3.4.2 Flange local bending

$R_n = 6.25 \cdot (F_{yf}) \cdot (t_f)^2$ (AISC 360-16, Eq. J10-1)

- R_n = 6.25·(36 ksi)·(0.9 in.^{2})^2
- R_n = 182.25 kips
- R_n/Ω = (182.25 kips) / 1.67
- R_n/Ω =**109.13 kips > 4.15 kips**

where:

- F_{yf} = 36 ksi – specified minimum yield stress of the flange
- t_f = 0.9 in.^{2} – thickness of the column flange
- Ω = 1.67 – safety factor (ASD)

F.3.4.3 Web local yielding

$R_n = (F_{yw}) \cdot (t_w) \cdot (5 \cdot k + l_b)$ (AISC 360-16, Eq. J10-2)

- R_n = (36 ksi)·(0.55 in.)·(5·(1.5 in.) + 0.75 in.)
- R_n =163.35 kips
- R_n/Ω = 163.35 kips / 1.5 = **108.9 kips > 4.15 kips**

where:

- F_{yw} = 36 ksi – specified minimum yield strength of the web material
- t_w = 0.55 in.^{2} – thickness of column web
- k = 1.5 in. – distance from outer face of the flange to the web toe of the fillet
- l_b = 0.75 in. – length of bearing (not less than k for end beam reactions)
- Ω = 1.5 – safety factor (ASD)

F.3.4.4 Web local crippling

$R_n = 0.80\cdot(t_w)^2\cdot[1+3\cdot(l_b/d)(t_w/t_f)^{1.5}]\cdot[E\cdot(F_{yw})\cdot(t_f)/(t_w)]^{1/2}\cdot Q_f$ (AISC 360-16, Eq. J10-4)

- $R_n = 0.80\cdot(0.55\text{ in.})^2\cdot[1+3\cdot(0.9\text{ in.}^2/12.7\text{ in.})(0.55\text{ in.}^2/0.9\text{ in.}^2)^{1.5}]\cdot[2.9E+7\cdot(36\text{ ksi})\cdot(0.9\text{ in.}^2)/(0.55)]^{1/2}\cdot(1.0) = 343.08$ kips
- R_n/Ω = (343.08 kips) / 2 = **171.54 kips > 4.15 kips**

where:

- F_{yw} = 36 ksi – specified minimum yield strength of the web material
- t_w = 0.55 in.^{2} – thickness of column web
- t_f = 0.9 in.^{2} – thickness of column flange
- l_b = 0.75 in. – length of bearing (not less than k for end beam reactions)
- d = 12.7 in. – full nominal depth of the member
- E = 2.9E+7 – Elastic Modulus
- Q_f = 1.0 – 1.0 for wide-flange sections and for HSS (connecting surface) in tension = as given in Table K3.2 (AISC 360-16) for all other HSS conditions
- Ω = 2 – safety factor (ASD)

Appendix G. IDEA StatiCa Model

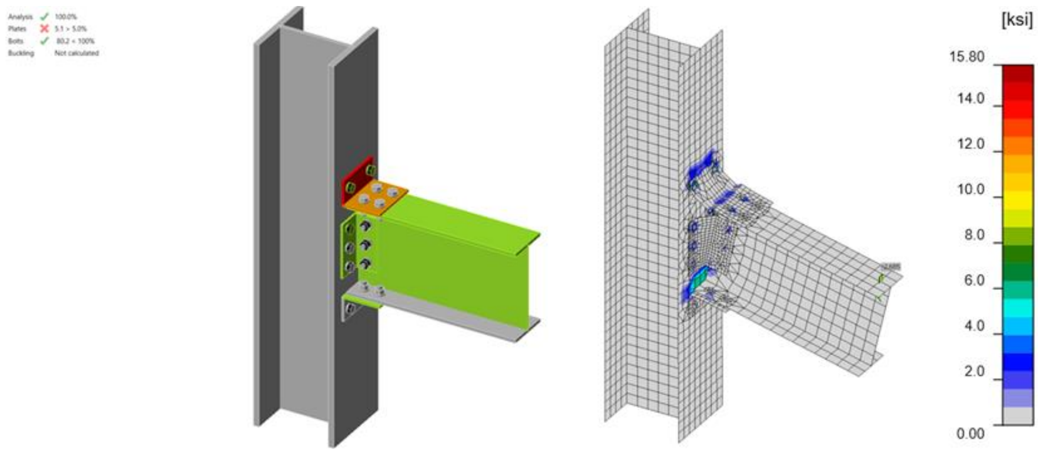


Figure G.1: IDEA StatiCa Test No. 14S1 (LRFD)

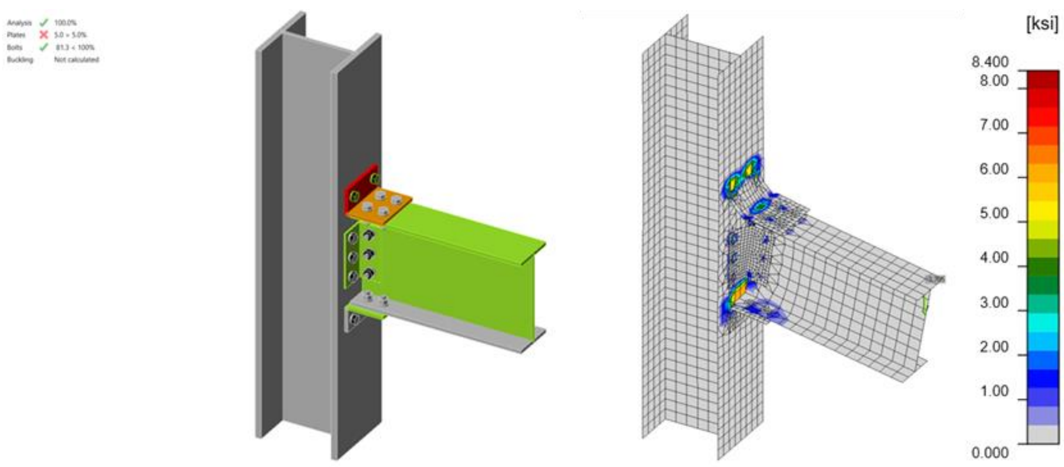


Figure G.2: IDEA StatiCa Test No. 14S2 (LRFD)

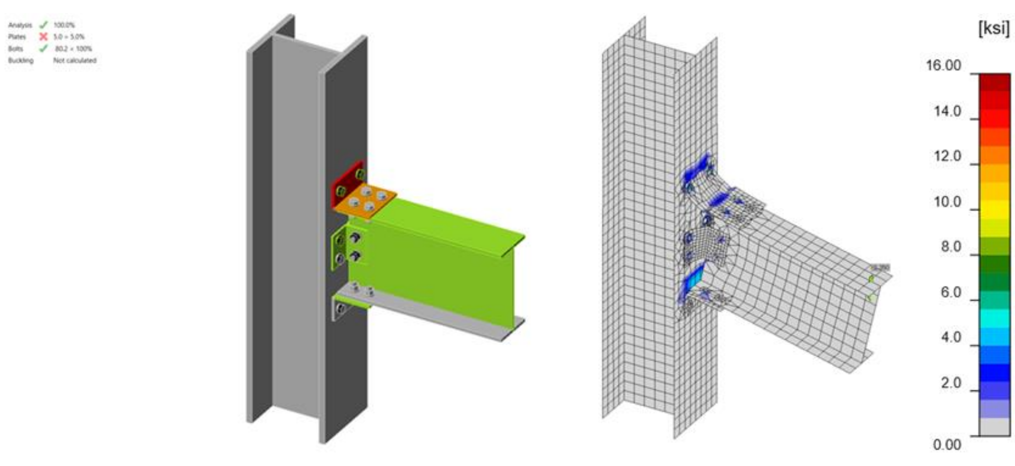


Figure G.3: IDEA StatiCa Test No. 14S3 (LRFD)

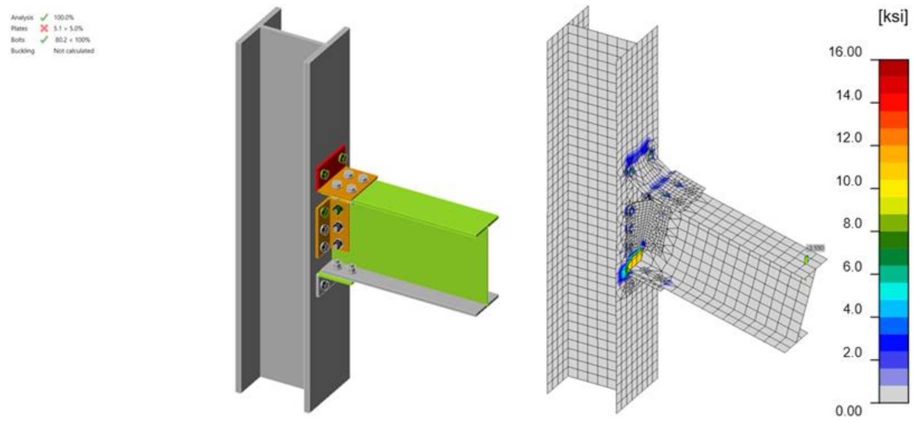


Figure G.4: IDEA StatiCa Test No. 14S4 (LRFD)

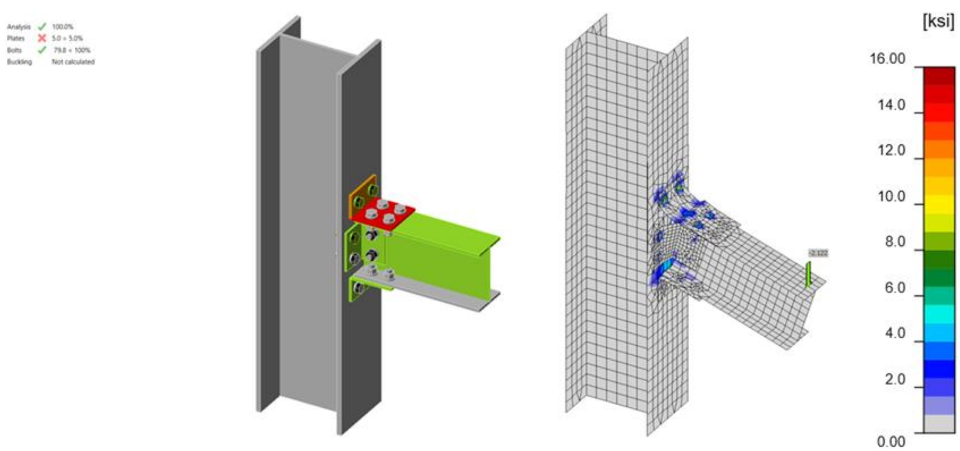


Figure G.5: IDEA StatiCa Test No. 8S1 (LRFD)

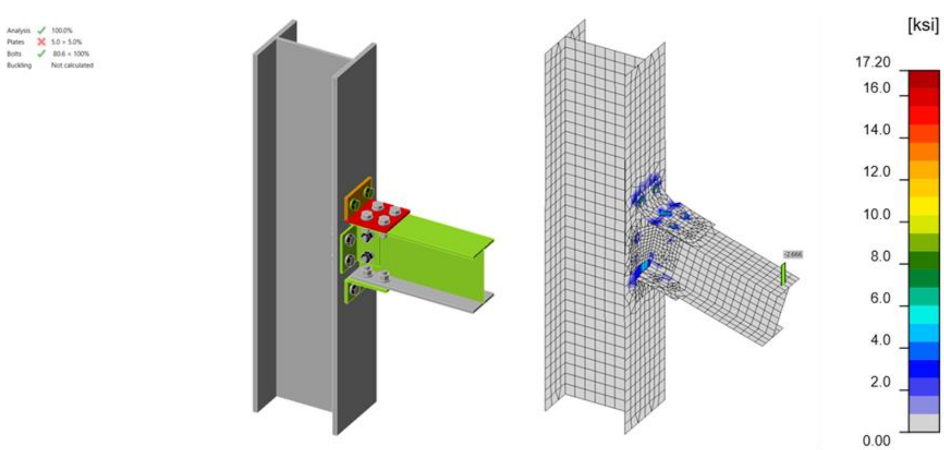


Figure G.6: IDEA StatiCa Test No. 8S2 (LRFD)

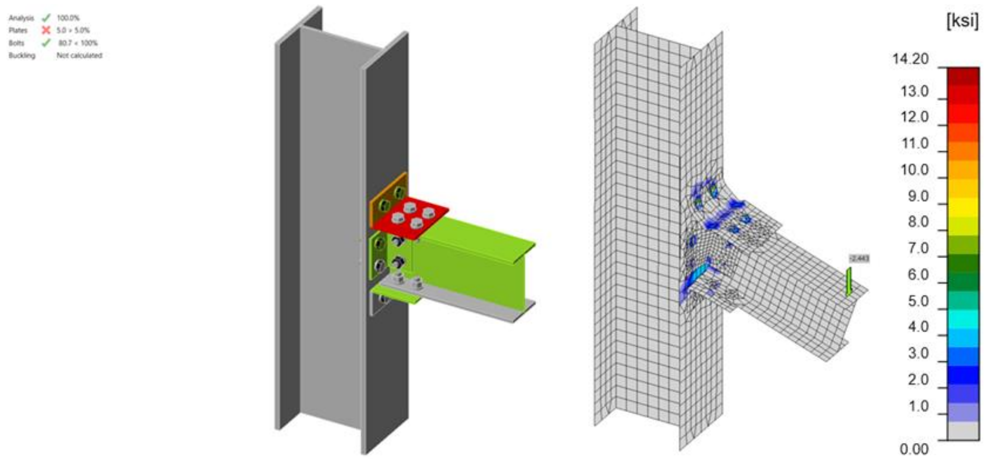


Figure G.7: IDEA StatiCa Test No. 8S3 (LRFD)

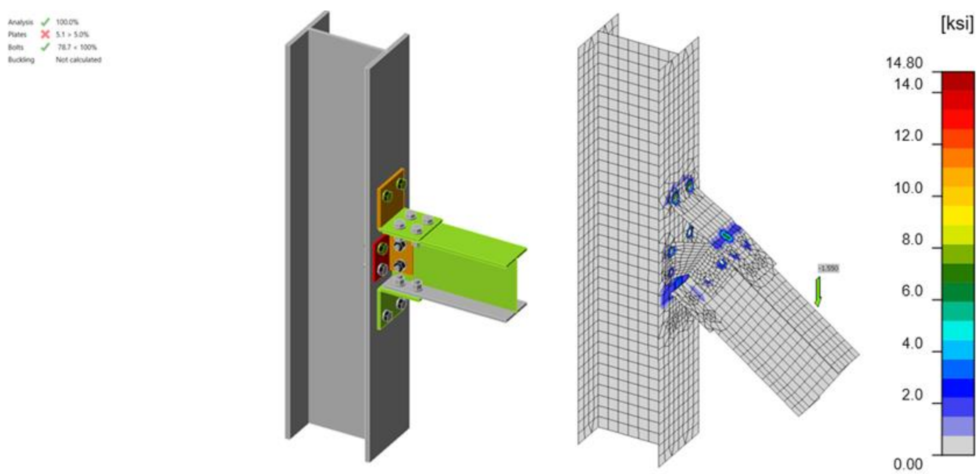


Figure G.8: IDEA StatiCa Test No. 8S4 (LRFD)

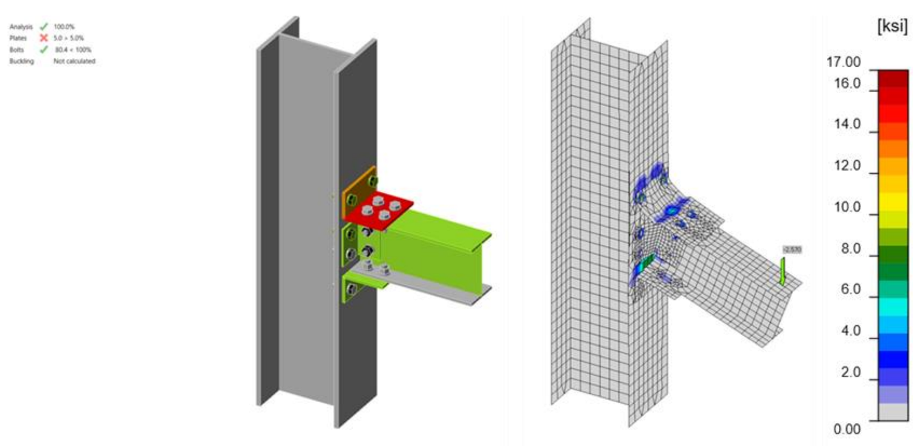


Figure G.9: IDEA StatiCa Test No. 8S5 (LRFD)

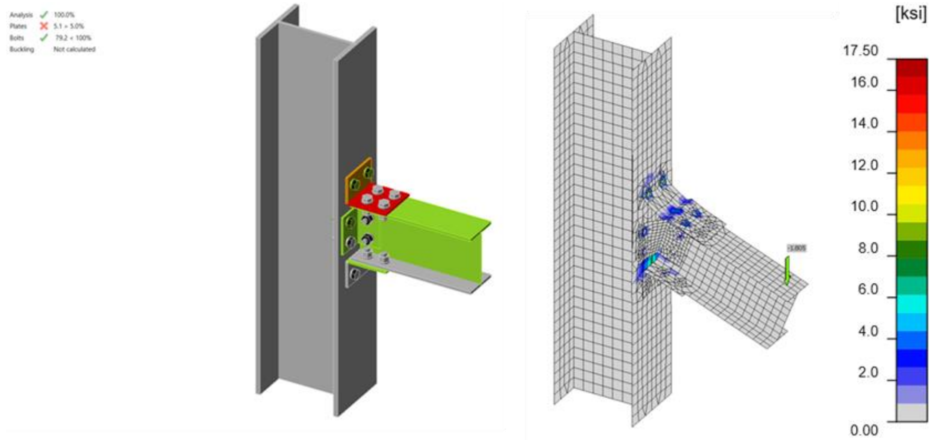


Figure G.10: IDEA StatiCa Test No. 8S6 (LRFD)

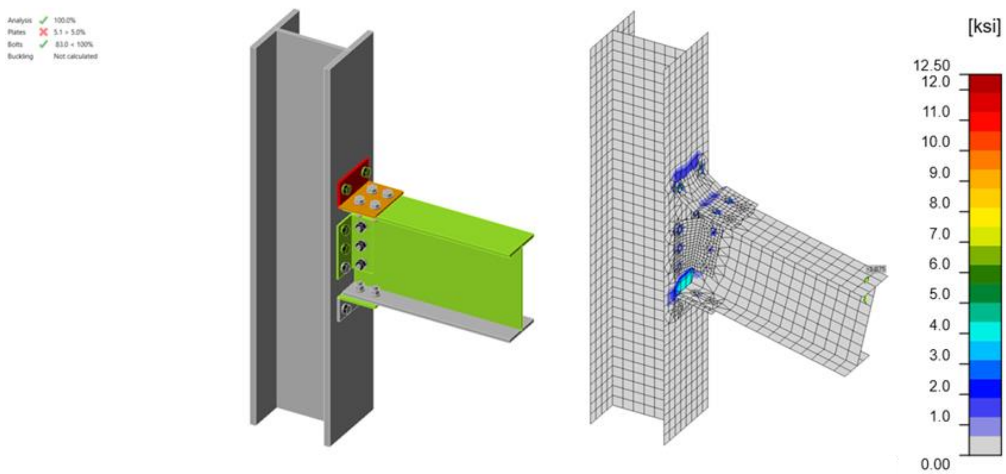


Figure G.11: IDEA StatiCa Test No. 14S1 (ASD)

Appendix H. Comparison of IDEA StatiCa and ABAQUS Models

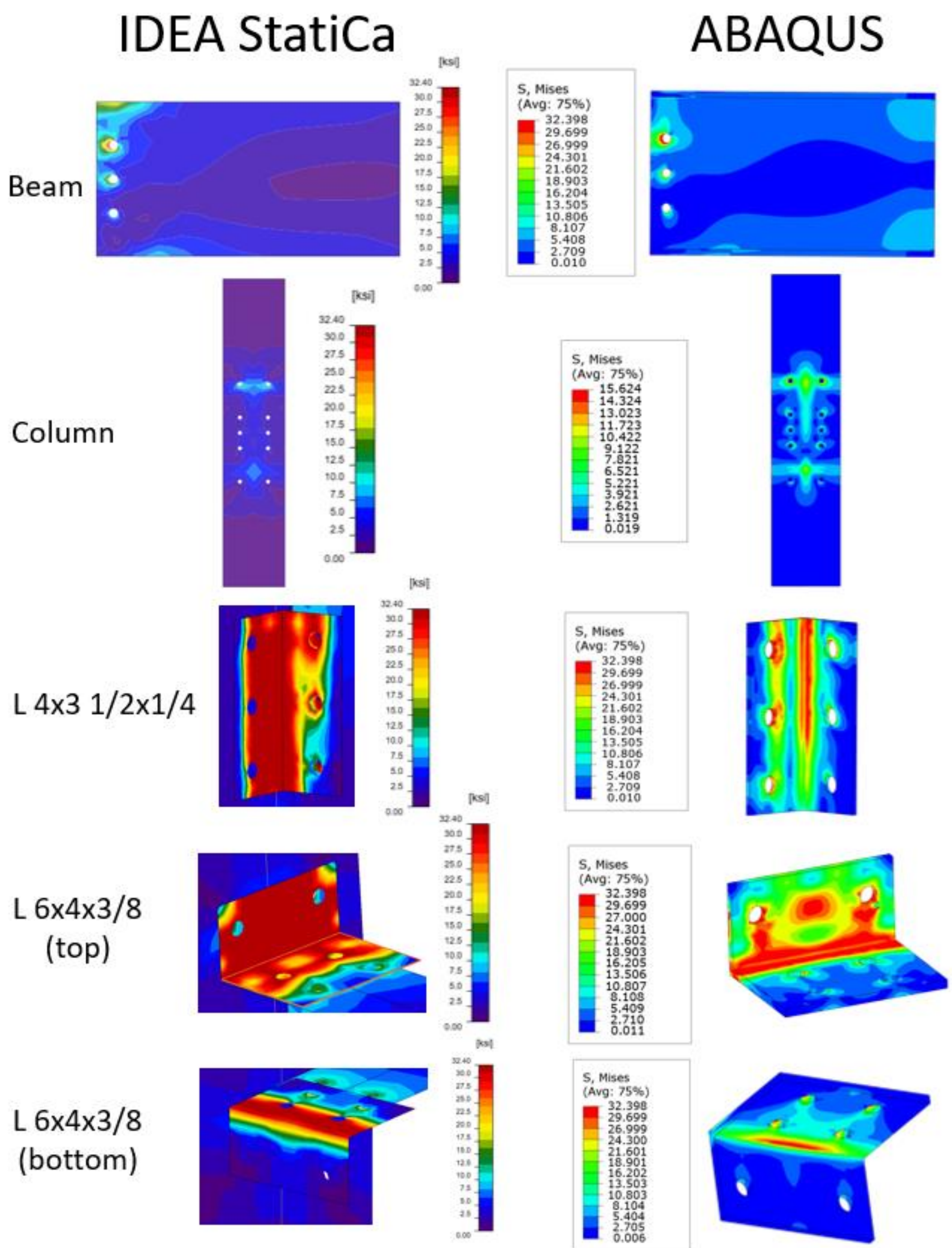


Figure H.1: Predicted stress between IDEA StatiCa and ABAQUS

- **Note:** In ABAQUS, node set was defined in different positions (e.g., shear plains) in order to extract data (i.e., nodal forces, shear forces, etc.) from the model and calculate the bolt loads.

Appendix I. LRFD Strength Calculations for Rigid Connection Test Specimen (Test No. BFP)

I.1 Loading and Assumptions

It is assumed that the double web-angle carries the applied shear force and the top- and seat-angle resist the applied moment.

Vertical shear:

P_D = 10 kips (dead load)

P_L = 30 kips (live load)

Strong-axis moment:

The shear loads are applied 6 in. away from the boltline.

M_D = (10 kips)·(6 in.) = 60 kips-in. (dead load)

M_L = (60 kips)·(6 in.) = 180 kips-in. (live load)

I.1.1 LRFD Load Demand or Design Loads

According to the Load and Resistance Factored Design (LRFD) procedure (AISC 360-16 or AISC Manual 2017), LRFD design load, factored load or demand, P_u under gravity loads is:

$P_u = 1.2P_D + 1.6P_L$ (LRFD)

P_u = 1.2·(10 kips) + 1.6·(30 kips) = 60 kips (LRFD)

$M_u = 1.2M_D + 1.6M_L$ (LRFD)

M_u = 1.2·(60 kips) + 1.6·(180 kips) = 360 kips-in. (LRFD)

Then, the required strength, R_u using the LRFD load combinations will be set equal to the design shear force, V_u applied on the connection. Then, for the cantilever beam and connection specimen:

$R_u = V_u = P_u = 1.2P_D + 1.6P_L$ (LRFD)

I.2 Properties of Test Specimen (Test No. BFP)

For the following analysis, consider:

- t_w = web thickness of beam
- d = depth of beam
- T = clear distance between web fillets

- F_y = specified minimum yield strength
- F_u = specified minimum tensile strength
- t_f = flange thickness of column
- L = length of the angles

- Beam: W30x108
  - Specified minimum yield strength, F_y = 50 ksi   ASTM A992
  - Specified minimum tensile strength, F_u = 65 ksi ASTM A992
  - Web thickness of beam, t_w = 0.545 in.
  - Width of beam, b = 10.475 in.
  - Flange thickness of beam, t_f = 0.76 in.
  - Depth of beam, d = 29.83 in.

    clear distance between web fillets, T = 26.5 in.   (Table 1-1, AISC Manual)

- Column: W14x233
  - Specified minimum yield strength, F_y = 50 ksi   ASTM A992
  - Specified minimum tensile strength, F_u = 65 ksi ASTM A992
  - Flange thickness of column, t_f = 1.72 in.
  - Web thickness of column, t_w = 1.07 in.
  - Depth of column, d = 16.04 in.   (Table 1-1, AISC Manual)

- Flange plates: 1.5’’ x 12’’ x 24’’
  - ASTM A36 F_y = 36 ksi F_u = 58 ksi
  - Number of bolts: 2x7
  - Diameter of bolts: 1 in.
  - Type of bolts: A325-X (threads are excluded)

- Web plate: 23.5’’ x 4.5’’ x 3/8’’
  - ASTM A36 F_y = 36 ksi F_u = 58 ksi
  - Number of bolts: 8
  - Bolt spacing = 3 in.
  - Diameter of bolts: ¾ in.
  - Type of bolts: A325-X (threads are excluded)

I.3. Single Plate Web Connection Geometric Checks

For the following analysis, consider:

- t_{ang} = angle thickness
- L_{min} = minimum angle length
- L_{max} = maximum angle length
- L_{emin} = minimum edge distance
- L_{ev} = vertical edge distance of angle
- L_{eh} = horizontal edge distance of angle

- L_{svmin} = minimum vertical center to center bolt distance
- L_{sv} = vertical center to center bolt distance
- L_{smax} = maximum center to center bolt distance
- L_{wmin} = minimum weld length
- d = bolt diameter
- T = clear distance between web fillets of the beam

I.3.1 Plate Length

$L_{min} = T/2$ (Page 10-104, AISC Manual)

- L_{min} = (26.5 in.)/2 = 13.25 in.
- L = 23.5 in. ≥ L_{min} = 13.25 in. **OK**

$L_{max} = T$ (Page 10-9, AISC Manual)

- L_{max} = 26.5 in.
- L = 23.5 in. ≤ L_{max} = 26.5 in. **OK**

where:

- T = 26.5 in. – clear distance between web fillets of the beam
- L_{min} = 6.14 in. – minimum plate length
- L_{max} = 12.27 in. – maximum plate length

I.3.2 Plate Edge Distance Check

L_{emin} = 1 in. for bolt diameter of ¾ in. (Table J3.4, AISC 360-16)

- L_{ev} = 1.25 in. > L_{emin} = 1 in. **OK**
- L_{eh} = 1.25 in. > L_{emin} = 1 in. **OK**

where:

- L_{emin} = 1 in. – minimum edge distance
- L_{ev} = 1.25 in. – vertical edge distance of plate
- L_{eh} = 1.25 in. – horizontal edge distance of plate

I.3.3 Plate Bolt Spacing Check

$L_{svmin} = (8/3)\cdot d$ (Section J3.3, AISC 360-16)

- L_{svmin} = (8/3)·(¾ in.)
- L_{svmin} = 2 in.
- L_{sv} = 3 in. > L_{svmin} = 2 in. **OK**

L_{smax} = minimum of ($24\cdot t_{ang}$) and 12 in. (Section J3.5, AISC 360-16)

- L_{smax} = 24·(0.375 in.)

- L_{smax} = 9 in.
- L_{sv} = 3 in. < L_{smax} = 9 in. **OK**

where:

- L_{svmin} = 2 in. – minimum vertical center to center bolt distance
- L_{sv} = 3 in. – vertical center to center bolt distance
- L_{smax} = 6 in. – maximum center to center bolt distance
- d = ¾ in. – bolt diameter

I.3.4 Plate Edge Distance Check

L_{emin} = 1 in. for bolt diameter of ¾ in. (Table J3.4, AISC 360-16)

- L_{ev} = 1.25 in. > L_{emin} = 1 in. **OK**
- L_{eh} = 1.25 in. > L_{emin} = 1 in. **OK**

where:

- L_{emin} = 1 in. – minimum edge distance
- L_{ev} = 1.50 in. – vertical edge distance of plate
- L_{eh} = 1.50 in. – horizontal edge distance of plate

I.3.5 Weld Size Check

$w_{min} = (5/8)\cdot(t_{ang})$ for t_{ang} = 3/8 in. (Page 10-87, AISC Manual)

- $w_{min} = (5/8)\cdot(3/8) = 0.23$
- w = 0.25 in. > w_{min} = 0.23 in. **OK**

where:

- w = ¼ in. – weld size
- w_{max} = 0.31 in. – maximum weld size
- w_{min} = 0.19 in. – minimum weld size
- t_{ang} = 3/8 in. – angle thickness

I.3.6 Weld Length Check

$L_{wmin} = 4\cdot w$ (Section J2.2b, AISC 360-16)

- L_{wmin} = 4·0.25 in.
- L_{wmin} = 1 in.
- L_w = 23.5 in. > L_{wmin} = 1 in.

where:

- L_{wmin} = 1 in. – minimum weld length
- L_w = 23.5 in. – weld length

I.4. Single Plate Web Connection LRFD Design Checks

In this section, design of the plate, bolts, and beam are checked for the single plate web connection. The LRFD design strength ϕR_n is calculated following the requirements of AISC Construction Manual (2017) and AISC 360-16. The calculated design strength ϕR_n is then compared with the design demand R_u calculated from structural analysis using the factored external loads.

For single web plate, bolt shear, bolt bearing, bolt tearout, shear yielding, shear rupture, block shear, and weld failure limit states are checked in Sections F.4.1.1 through F.4.1.7.

For beam, bolt bearing, bolt tearout, and shear yielding failure limit states are checked in Sections F.4.2.1 through F.4.2.3.

For the following analysis, consider:

- F_{nv} = nominal shear strength of fasteners
- A_b = nominal bolt area
- ϕ = strength factor (LRFD)
- F_{nt} = nominal tensile strength of fasteners
- d = nominal bolt diameter
- t = thickness of the beam
- F_u = specified minimum tensile strength of beam
- l_c = clear distance
- d_h = nominal hole dimension
- L_{sv} = vertical center to center bolt distance
- L_{ev} = vertical edge distance of angle
- L_{csv} = clear vertical distance between bolts
- L_{cev} = clear vertical edge distance
- A_{nv} = net area subject to shear, in.^{2}
- L_h = hole dimension for tension and shear net area
- L_e = effective length
- n = number of the bolts
- U_{bs} = stress index for uniform tension stress
- A_{nt} = net area subject to tension
- A_{gv} = gross area subject to shear
- A_{nv} = net area subject to shear
- L_{ev} = vertical edge distance of angle
- L_{eh} = horizontal edge distance of angle
- L = length of the angles

I.4.1 Web Plate

I.4.1.1 Bolt shear

$R_n = F_n \cdot A_b$ (Eq. J3-1, AISC 360-16)

- Shear strength of one bolt:
- ϕR_n = 0.75·68·0.442 = **22.54 kips/bolt**
- Total shear strength of eight bolts:
- ϕR_n = (8 bolts)·22.54 kips = **180.32 kips > 60 kips OK**

where:

- F_{nv} = 68 ksi – nominal shear strength of fasteners (Table J3.2, AISC 360-16)
- A_b = 0.442 in.^{2} – nominal bolt area
- ϕ = 0.75 – strength factor (LRFD)

I.4.1.2 Bolt bearing

The nominal bearing strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 2.4 \cdot d \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6a)

- R_n =2.4·(3/4 in.)·(0.375 in.)·(65 ksi)
- R_n = 43.88 kips/bolt
- ϕR_n = 0.75·(8 bolts)·43.88 kips
- ϕR_n = **263.25 kips/connection > 60 kips OK**

where:

- d = 3/4 in. – nominal bolt diameter
- t = 0.375 in. – thickness of angle
- F_u = 65 ksi – specified minimum tensile stress of plate
- ϕ = 0.75 – strength factor (LRFD)

I.4.1.3 Bolt tearout

The available tearout strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 1.2 \cdot l_c \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6c)

- $L_{csv} = L_{sv} - d_h$
- L_{csv} = 3 in. - 13/16 in. = 2.19 in.
- $R_{n\text{-}span}$ = 1.2·(2.19 in.)·(0.375 in.)·(65 ksi)
- $R_{n\text{-}span}$ = 64.06 kips/bolt
- $L_{cev} = L_{ev} - d_h /2$
- L_{cev} = 1.25 in. - (13/16)/2 in. = 0.84 in.
- $R_{n\text{-}end}$ = 1.2·(0.84 in.)·(0.375 in.)·(65 ksi)
- $R_{n\text{-}end}$ = 24.57 kips/bolt

- ϕR_n = 0.75·[7·(64.06 kips) + (24.57 kips)]
- ϕR_n = **465.30 kips/conn. > 60 kips OK**

where:

- l_c : clear distance
- d_h = 13/16 in. – nominal hole dimension (3/4-in.-diameter bolt)
- L_{sv} = 3 in. – vertical center to center bolt distance
- L_{ev} = 1.25 in. – vertical edge distance of angle
- L_{csv} = 2.19 in. – clear vertical distance between bolts
- L_{cev} = 0.84 in. – clear vertical edge distance
- ϕ = 0.75 – strength factor (LRFD)

I.4.1.4 Shear yielding

$R_n = 0.60 \cdot F_y \cdot A_{gv}$ ϕ = 1.00 (LRFD) (AISC 360-16, Eq. J4-3)

- A_{gv} = (23.5 in.)·(0.375 in.) = 8.81 in.^{2}
- ϕR_n = 1.00·0.60·(50 kips)·(8.81 in.^{2})
- ϕR_n = **264.30 kips > 60 kips OK**

where:

- A_{gv} = 8.81 in.^{2} – gross area subject to shear
- ϕ = 1.00 – strength factor (LRFD)
- L = 23.5 in. – length of angle
- t = 0.375 in. – thickness of angle
- F_y = 50 ksi – specified minimum yield strength of plate

I.4.1.5 Shear rupture

$R_n = 0.60 \cdot F_u \cdot A_{nv}$ (AISC 360-16, Eq. J4-4)

- $L_h = d_h$ + 1/16 (Section B4-3b, AISC Manual)
- L_h = 0.813 + 1/16 = 0.875 in.
- $L_e = L - n \cdot L_h$
- L_e = 23.5 - 8·0.875 = 16.5 in.
- $A_{nv} = L_e \cdot t_p$
- A_{nv} = (16.5 in.)·(0.375 in.) = 6.19 in.^{2}
- ϕR_n = (0.75)·0.60·(65 kips)·(6.19 in.^{2})
- ϕR_n = **181.06 kips > 60 kips OK**

where:

- A_{nv} = 6.19 in.^{2} – net area subject to shear, in.^{2}

- L_h = 0.875 in. – hole dimension for tension and shear net area
- L_e = 16.5 in. – effective length
- ϕ = 0.75 – strength factor (LRFD)
- n = 8 – number of the bolts
- t = 0.375 in. – thickness of angle

I.4.1.6 Block shear

$R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$ (AISC 360-16, Eq. J4-5)

- $A_{gv} = (L - L_{ev}) \cdot (t_p)$
- A_{gv} = (23.5 in. - 1.25 in.)·(0.375 in.) = 8.34 in.^{2}
- $A_{nv} = A_{gv}$ - (n-0.5)·(d_h + 1/16 in.)·(t_p)
- A_{nv} = 8.34 in.^{2} - (8-0.5)·(0.813 + 1/16 in.)·(0.375 in.)
- A_{nv} = 5.88 in.^{2}
- A_{nt} = [$\mathrm{L_{eh}}$ - 0.5·(d_h + 1/16 in.)]·(t_p)
- A_{nt} = [1.5 in. - 0.5·(0.813 in. + 1/16 in.)]·(0.375 in.)
- A_{nt} = 0.39 in.^{2}
- U_{bs} = 1 (as the tension stress is uniform)
- $R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$
- R_n = 0.60·(65 kips/in.^{2})·(5.88 in.^{2}) + 1·(65 kips/in.^{2})·(0.39 in.^{2}) ≤ 0.60·(50 kips/in.^{2})·(8.34 in.^{2}) + 1·(65 kips/in.^{2})·(0.39 in.^{2})
- R_n = **254.67 kips** ≤ 275.55 kips
- ϕR_n = 0.75·(254.67 kips)
- **ϕR_n = 191.0 kips > 60 kips OK**

where:

- U_{bs} = 1 – Stress index for uniform tension stress
- A_{nt} = 0.39 in.^{2} – net area subject to tension
- A_{gv} = 8.34 in.^{2} – gross area subject to shear
- A_{nv} = 5.88 in.^{2} – net area subject to shear
- L_{ev} = 1.25 in. – vertical edge distance of angle
- L_{eh} = 1.5 in. – horizontal edge distance of angle
- ϕ = 0.75 – strength factor (LRFD)

I.4.1.7 Weld Shear (web plate to the column flange)

$\phi R_n = (2 \text{ welds}) \cdot (0.75) \cdot [(F_{EXX}) \cdot 0.6 \cdot (D/16) \cdot L \cdot (2^{1/2}/2)]$ (Eq. 8-2a/8-2b, AISC Manual)

- ϕR_n = (2 welds)·(1.392·D·L)
- ϕR_n = (2 welds)·(1.392·4·23.5)
- **ϕR_n = 261.70 kips > 60 kips OK**

where:

- D = 4 – number of sixteenths-of-an-inch in the weld size
- L = 23.5 in. – length of the angles
- F_{EXX} = 70 ksi – filler metal electrode classification strength

I.4.2 Beam

I.4.2.1 Bolt bearing

The nominal bearing strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 2.4 \cdot d \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6a)

- R_n =2.4·(3/4 in.)·(0.545 in.)·(65 ksi)
- R_n = 63.77 kips/bolt
- ϕR_n = 0.75·(8 bolts)·63.77 kips
- ϕR_n = **382.59 kips/connection > 60 kips OK**

where:

- d = 3/4 in. – nominal bolt diameter
- t_w = 0.545 in. – thickness of beam web
- F_u = 65 ksi – specified minimum tensile stress of angle
- ϕ = 0.75 – strength factor (LRFD)

I.4.2.2 Bolt tearout

The available tearout strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 1.2 \cdot l_c \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6c)

- $L_{csv} = L_{sv} - d_h$
- L_{csv} = 3 in. - 13/16 in. = 2.19 in.
- $R_{n\text{-}span}$ = 1.2·(2.19 in.)·(0.545 in.)·(65 ksi)
- $R_{n\text{-}span}$ = 93.10 kips/bolt
- $L_{cev} = L_{ev} - d_h/2$
- L_{cev} = 4.4 in. - (13/16)/2 in. = 3.99 in.
- $R_{n\text{-}end}$ = 1.2·(3.99 in.)·(0.545 in.)·(65 ksi)
- $R_{n\text{-}end}$ = 169.62 kips/bolt
- ϕR_n = 0.75·[7·(93.10 kips) + (169.62 kips)]
- ϕR_n = **615.98 kips/conn. > 60 kips OK**

where:

- l_c : clear distance
- d_h = 13/16 in. – nominal hole dimension (3/4-in.-diameter bolt)
- L_{sv} = 3 in. – vertical center to center bolt distance
- L_{ev} = 1.25 in. – vertical edge distance of angle
- L_{csv} = 2.19 in. – clear vertical distance between bolts
- L_{cev} = 3.99 in. – clear vertical edge distance
- ϕ = 0.75 – strength factor (LRFD)
- t_w = 0.545 in. – thickness of beam web
- F_u = 65 ksi – specified minimum tensile stress of beam

I.4.2.3 Shear yielding

$R_n = 0.60 \cdot F_y \cdot A_{gv}$ $\phi = 1.00$ (LRFD) (AISC 360-16, Eq. J4-3)

- A_{gv} = (29.83 in.)·(0.545 in.) = 16.26 in.^{2}
- ϕR_n = 1.00·0.60·(50 kips)·(16.26 in.^{2})
- ϕR_n = **487.72 kips > 60 kips OK**

where:

- A_{gv} = 16.26 in.^{2} – gross area subject to shear
- ϕ = 1.00 – strength factor (LRFD)
- d = 29.83 in. – beam depth
- t_w = 0.545 in. – thickness of beam web
- F_y = 50 ksi – specified minimum yield strength

I.5. Flange Plate Connection Geometric Checks

For the following analysis, consider:

- L_{emin} = minimum edge distance
- L_{el} = longitudinal edge distance of angle
- L_{et} = transverse edge distance of angle
- L_{smax} = maximum center to center bolt distance
- d = bolt diameter
- L_{st} = transfer center to center bolt distance
- L_{sl} = longitudinal center to center bolt distance

I.5.1 Plate Edge Distance Check

L_{emin} = 1 in. for bolt diameter of 1 in. (Table J3.4, AISC 360-16)

- L_{el} = 3 in. > L_{emin} = 1.25 in. **OK**
- L_{et} = 3 in. > L_{emin} = 1.25 in. **OK**

where:

- L_{emin} = 1.25 in. – minimum edge distance
- L_{el} = 3 in. – longitudinal edge distance of angle
- L_{et} = 3 in. – transverse edge distance of angle

I.5.2 Plate Bolt Spacing Check

$L_{smin} = (8/3) \cdot d$ (Section J3.3, AISC 360-16)

- $L_{svmin} = (8/3) \cdot (1 \text{ in.})$
- L_{svmin} = 2.67 in.
- L_{st} = 6 in. > L_{svmin} = 2.67 in. **OK**
- L_{sl} = 3 in. > L_{svmin} = 2.67 in. **OK**

L_{smax} = minimum of $(24 \cdot t_p)$ and 12 in. (Section J3.5, AISC 360-16)

- L_{smax} = 12 in. < $24 \cdot (0.76 \text{ in.})$ = 18.24 in.
- L_{smax} = 12 in.
- L_{st} = 6 in. < L_{smax} = 12 in. **OK**
- L_{sl} = 3 in. < L_{smax} = 12 in. **OK**

where:

- L_{st} = 6 in. – transfer center to center bolt distance
- L_{sl} = 3 in. – longitudinal center to center bolt distance
- L_{smin} = 2.67 in. – minimum center to center bolt distance
- L_{smax} = 12 in. – maximum center to center bolt distance
- d = 1 in. – bolt diameter

I.6. Flange Plate Connection LRFD Design Checks

In this section, design of the flange plates, bolts, beam, and column are checked for the flange plate connection. The LRFD design strength ϕR_n is calculated following the requirements of AISC Construction Manual (2017) and AISC 360-16. The calculated design strength ϕR_n is then compared with the design demand R_u calculated from structural analysis using the factored external loads.

Flange bolt shear force and flange tension force corresponding the design moment are calculated in Sections F.6.1.1 and F.6.1.2.

For flange plates, bolt shear, bolt bearing, bolt tearout, tensile yielding, tensile rupture, block shear, and compression failure limit states are checked in Sections F.6.2.1 through F.6.2.7.

For beam, bolt bearing, bolt tearout, and block shear, and flexural failure limit states are checked in Sections F.6.3.1 through F.6.3.4.

For column, web panel zone, flange local bending, web local yielding and web local crippling are checked in Sections F.6.4.1 through F.6.4.4.

For the following analysis, consider:

- d_m = depth of beam
- P_{uf} = flange bolt shear
- F_{nv} = nominal shear strength of fasteners
- A_b = nominal bolt area
- L = length of angle
- t = flange thickness
- l_c = clear distance
- d_h = nominal hole dimension
- L_{sv} = vertical center to center bolt distance
- L_{ev} = vertical edge distance of beam
- L_{csv} = clear vertical distance between bolts
- L_{cev} = clear vertical edge distance
- U_{bs} = stress index for uniform tension stress
- A_{nt} = net area subject to tension
- A_{gv} = gross area subject to tension
- A_{nv} = net area subject to tension
- L_{ev} = vertical edge distance of plate
- L_{eh} = horizontal edge distance of plate
- ϕ = strength factor (LRFD)
- g = gage between transfer bolts of plate
- b = width of plate
- A_{fg} = gross area of tension flange (Section B4.3a)
- A_{fn} = net area of tension flange (Section B4.3b)
- F_u = specified minimum tensile strength
- F_y = specified minimum yield strength
- Y_t = beam flexural factor (= 1.0 for $F_y/F_u \leq 0.8$, otherwise = 1.1)
- b_f = beam flange width
- t_f = beam flange thickness
- S_x = minimum elastic section modulus of beam
- A_g = column cross-sectional area
- P_r = required axial strength using LRFD
- P_y = axial yield strength of the column
- d_c = depth of column
- t_w = thickness of column web

I.6.1 Design Forces

I.6.1.1 Flange Bolt Shear Force

The moment arm between flange forces is equal to the depth of the beam.

$P_{uf} = M_u/d_m$

- P_{uf} = 360 kips-in. / 29.83 in. = **12.07 kips**

where:

- d_m = 29.83 in. – depth of beam
- P_{uf} = 12.07 kips – flange bolt shear

I.6.1.2 Flange Tension Force

The moment arm between flange forces is equal to the depth of the beam plus one plate thickness.

$P_{uf} = M_u/(d + t)$

- P_{uf} = 360 kips-in. / (29.83 in. + 1.5 in.) = **11.49 kips**

where:

- d_m = 29.83 in – depth of beam
- t = 1.5 in. – plate thickness
- P_{uf} = 11.49 kips – flange tension force

I.6.2 Flange Plate

I.6.2.1 Bolt shear

$R_n = F_n \cdot A_b$ (Eq. J3-1, AISC 360-16)

- Shear strength of one bolt:
- ϕR_n = 0.75·84·0.785 = 49.46 kips/bolt
- Total shear strength of eight bolts:
- ϕR_n = (14 bolts)·49.46 kips = **692.44 kips > 12.07 kips OK**

where:

- F_{nv} = 84 ksi – nominal shear strength of fasteners (Table J3.2, AISC 360-16)
- A_b = 0.785 in.^{2} – nominal bolt area
- ϕ = 0.75 – strength factor (LRFD)

I.6.2.2 Bearing

The nominal bearing strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 2.4 \cdot d \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6a)

- R_n = 2.4·(1 in.)·(1.5 in.)·(65 ksi)
- R_n = 234 kips/bolt
- ϕR_n = 0.75·(14 bolts)·234 kips
- ϕR_n = **2457.0 kips/connection > 12.07 kips OK**

where:

- d = 1 in. – nominal bolt diameter
- t = 1.5 in. – plate thickness
- F_u = 65 ksi – specified minimum tensile stress of plate
- ϕ = 0.75 – strength factor (LRFD)

I.6.2.3 Tearout

The available tearout strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 1.2 \cdot l_c \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6c)

- $L_{csv} = L_{sv} - d_h$
- L_{csv} = 3 in. - 1.125 in. = 1.875 in.
- $R_{n\text{-}span}$ = 1.2·(1.875 in.)·(1.5 in.)·(65 ksi)
- $R_{n\text{-}span}$ = 219.38 kips/bolt
- $L_{cev} = L_{ev} - d_h/2$
- L_{cev} = 3 in. - (1.125 in.)/2 = 2.44 in.
- $R_{n\text{-}end}$ = 1.2·(2.44 in.)·(1.5 in.)·(65 ksi)
- $R_{n\text{-}end}$ = 285.48 kips/bolt
- ϕR_n = 0.75·(2)·[6·(219.38 kips) + (285.48 kips)]
- ϕR_n = **2402.64 kips/conn. > 12.07 kips OK**

where:

- l_c : clear distance
- d_h = 1.125 in. – nominal hole dimension (1-in.-diameter bolt)
- L_{sv} = 3 in. – vertical center to center bolt distance
- L_{ev} = 3 in. – vertical edge distance of angle
- L_{csv} = 1.875 in. – clear vertical distance between bolts
- L_{cev} = 2.44 in. – clear vertical edge distance
- ϕ = 0.75 – strength factor (LRFD)

I.6.2.4 Tensile yielding

$R_n = 0.60 \cdot F_y \cdot A_{gv}$ ϕ = 1.00 (LRFD) (AISC 360-16, Eq. J4-3)

- A_{gv} = (12 in.)·(1.5 in.) = 18 in.^{2}
- ϕR_n = 0.90·(50 kips)·(18 in.^{2})
- ϕR_n = **810.0 kips > 11.49 kips OK**

where:

- A_{gv} = 18 in.^{2} – gross area subject to shear
- ϕ = 0.90 – strength factor (LRFD)
- L = 12 in. – length of angle
- t = 1.5 in. – flange thickness
- F_y = 50 ksi – specified minimum yield strength of plate

I.6.2.5 Tension rupture

$R_n = F_u \cdot A_e$ (AISC 360-16, Eq. J4-2)

- A_g = (12 in.)·(1.5 in.) = 18 in.^{2}
- $L_h = d_h + 1/16$ (Section B4-3b, AISC Manual)
- L_h = 1.125 + 1/16 = 1.19 in.
- $A_n = (L - n \cdot L_h) \cdot t_p$
- A_n = (12 in. - 2·1.19 in.)·(1.5 in.) = 14.44 in.^{2}
- $A_e = A_n \leq 0.85 A_g$ = 15.3 in.^{2} (AISC 360-16, Sec. J4-1)
- A_e = 14.44 in.^{2}
- R_n = (65 ksi)·(14.44 in.^{2}) = 938.6 kips
- ϕR_n = 0.75·(938.6 ksi) = **703.95 kips > 11.49 kips** **OK**

where:

- A_g = 18 in.^{2} – gross area
- A_e = 14.44 in.^{2} – effective net area
- L_h = 1.19 in. – hole dimension
- F_u = 65 ksi – specified minimum tensile stress of plate
- ϕ = 0.75 – strength factor (LRFD)

I.6.2.6 Block shear

$R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$ (AISC 360-16, Eq. J4-5)

- $A_{gv} = (L - L_{ev}) \cdot (t)$
- A_{gv} = (2 planes)·(24 in. - 3 in.)·(1.5 in.) = 63 in.^{2}
- $A_{nv} = A_{gv}$ - (n-0.5)·(d_h + 1/16 in.)·(t)
- A_{nv} = 63 in.^{2} - (2 planes)·(7-0.5)·(1.125 + 1/16 in.)·(1.5 in.)
- A_{nv} = 39.84 in.^{2}
- A_{nt} = [min(g, b - g) - 1.0·(d_h + 1/16 in.)]·(t)
- A_{nt} = [6 in. - 1.0·(1.125 in. + 1/16 in.)]·(1.5 in.)
- A_{nt} = 7.22 in.^{2}
- U_{bs} = 1 (as the tension stress is uniform)
- $R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$
- R_n = 0.60·(65 kips/in.^{2})·(39.84 in.^{2}) + 1·(65 kips/in.^{2})·(7.22 in.^{2}) ≤ 0.60·(50 kips/in.^{2})·(63 in.^{2}) + 1·(65 kips/in.^{2})·(7.22 in.^{2})

- R_n = **2023.06 kips** ≤ 2359.30 kips
- ϕR_n = 0.75·(2023.06 kips)
- ϕR_n **= 1517.30 kips > 11.49 kips** **OK**

where:

- U_{bs} = 1 – Stress index for uniform tension stress
- A_{nt} = 7.22 in.^{2} – net area subject to tension
- A_{gv} = 63 in.^{2} – gross area subject to tension
- A_{nv} = 39.84 in.^{2} – net area subject to tension
- L_{ev} = 3 in. – vertical edge distance of plate
- L_{eh} = 3 in. – horizontal edge distance of plate
- ϕ = 0.75 – strength factor (LRFD)
- g = 6 in. – gage between transfer bolts of plate
- b = 12 in. – width of plate

I.6.2.7 Compression

$P_n = F_y \cdot A_g$ when $L_c/r \leq 25$ (AISC 360-16, Sec. J4-4)

- $r = t_p/(12)^{1/2}$ = 1.5 in./$(12)^{1/2}$
- r = 0.43 in.
- $L_c = K \cdot L/r$ = 0.65·(3 in.)/(0.43 in.)
- L_c = 4.43 < 25 **NOT OK**
- $A_g = L_p \cdot t_p$ = (12 in.)·(1.5 in.)
- A_g = 18 in.^{2}
- $\phi P_n = \phi \cdot F_y \cdot A_g$ = 0.9·(50 ksi)·(18 in.^{2})
- ϕP_n = **810.0 kips > 11.49 kips** **OK**

where:

- K = 0.65 – effective length factor
- L = 3 in. – distance between adjacent bolt holes
- L_c = 4.43 – effective length
- A_g = 18 in.^{2} – gross area
- ϕ = 0.9 – strength factor (LRFD)

I.6.3 Beam

I.6.3.1 Bearing

The nominal bearing strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 2.4 \cdot d \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6a)

- R_n = 2.4·(1 in.)·(0.76 in.)·(65 ksi)
- R_n = 118.56 kips/bolt
- ϕR_n = 0.75·(14 bolts)·118.56 kips
- ϕR_n = **1244.88 kips/connection > 12.07 kips OK**

where:

- d = 1 in. – nominal bolt diameter
- t_p = 0.76 in. – thickness of beam flange
- F_u = 58 ksi – specified minimum tensile stress of beam
- ϕ = 0.75 – strength factor (LRFD)

I.6.3.2 Tearout

The available tearout strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 1.2 \cdot l_c \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6c)

- $L_{csv} = L_{sv} - d_h$
- L_{csv} = 3 in. - 1.125 in. = 1.875 in.
- $R_{n\text{-}span}$ = 1.2·(1.875 in.)·(0.76 in.)·(65 ksi)
- $R_{n\text{-}span}$ = 111.15 kips/bolt
- $L_{cev} = L_{ev} - d_h/2$
- L_{cev} = 2 in. - (1.125 in.)/2 = 1.44 in.
- $R_{n\text{-}end}$ = 1.2·(1.44 in.)·(0.76 in.)·(65 ksi)
- $R_{n\text{-}end}$ = 85.22 kips/bolt
- ϕR_n = 0.75·(2)·[6·(111.15 kips) + (85.22 kips)]
- ϕR_n = **1128.18 kips/conn. > 12.07 kips OK**

where:

- l_c : clear distance
- d_h = 1.125 in. – nominal hole dimension (1-in.-diameter bolt)
- L_{sv} = 3 in. – vertical center to center bolt distance
- L_{ev} = 2 in. – vertical edge distance of beam
- L_{csv} = 1.875 in. – clear vertical distance between bolts
- L_{cev} = 2.44 in. – clear vertical edge distance
- ϕ = 0.75 – strength factor (LRFD)

I.6.3.3 Block shear

$R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$ (AISC 360-16, Eq. J4-5)

- $A_{gv} = (L_{ev}+(n\text{-}1) \cdot s) \cdot (t)$

- A_{gv} = (2 planes)·(2 in. +(7-1)·3 in.)·(0.76 in.) = 30.4 in.^{2}
- $A_{nv} = A_{gv}$ - (n-0.5)·(d_h + 1/16 in.)·(t)
- A_{nv} = 30.4 in.^{2} - (2)·(7-0.5)·(1.125 + 1/16 in.)·(0.76 in.)
- A_{nv} = 18.67 in.^{2}
- A_{nt} = [(b - g) - 1.0·(d_h + 1/16 in.)]·(t)
- A_{nt} = [(10.5 in. - 6 in.) - 1.0·(1.125 in. + 1/16 in.)]·(0.76 in.)
- A_{nt} = 2.52 in.^{2}
- U_{bs} = 1 (as the tension stress is uniform)
- $R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$
- R_n = 0.60·(65 kips/in.^{2})·(18.67 in.^{2}) + 1·(65 kips/in.^{2})·(2.52 in.^{2}) ≤ 0.60·(50 kips/in.^{2})·(30.4 in.^{2}) + 1·(65 kips/in.^{2})·(2.52 in.^{2})
- R_n = **891.93 kips** ≤ 1075.8 kips
- ϕR_n = 0.75·(891.93 kips)
- ϕR_n **= 668.95 kips > 12.07 kips OK**

where:

- U_{bs} = 1 – Stress index for uniform tension stress
- A_{nt} = 2.52 in.^{2} – net area subject to tension
- A_{gv} = 30.4 in.^{2} – gross area subject to tension
- A_{nv} = 18.67 in.^{2} – net area subject to tension
- L_{ev} = 2 in. – vertical edge distance of plate
- L_{eh} = 3 in. – horizontal edge distance of plate
- ϕ = 0.75 – strength factor (LRFD)
- g = 6 in. – gage between transfer bolts of plate
- b = 10.5 in. – width of beam flange
- s = 3 in. – spacing between bolts

I.6.3.4 Flexural strength

$M_n = S_x \cdot F_u \cdot A_{fn} / A_{fg}$ when $F_u \cdot A_{fn} < Y_t \cdot F_y \cdot A_{fg}$ (AISC 360-16, Eq. F13-1)

Otherwise, the limit state of tensile rupture does not apply

- $A_{fg} = (b_f) \cdot (t_f)$
- A_{fg} = (10.5 in.)·(0.76 in.) = 7.98 in.^{2}
- $A_{fn} = A_{fg}$ - n·(d_h + 1/16 in.)·(t_p)
- A_{fn} = 7.98 in.^{2} - 2·(1.125 in. + 1/16 in.)·(0.76 in.) = 6.18 in.^{2}
- F_y/F_u = (50 ksi)/(65 ksi) = 0.77 < 0.8 **therefore Y_t = 1.0**
- $F_u \cdot A_{fn}$ = (65 ksi)·(6.18 in.^{2}) = 401.7 kips
- $Y_t \cdot F_y \cdot A_{fg}$ = 1.0·(50 ksi)·(7.98 in.^{2}) = 399 kips
- $F_u \cdot A_{fn}$ = 401.7 kips < $Y_t \cdot F_y \cdot A_{fg}$ = 399 kips **NOT OK**
- $\phi M_n = \phi \cdot F_y \cdot S_x$ = 0.90·(50 ksi)·(346 in.^{3}) / 12

- ϕM_n = **1297.5 kips-ft > 12.07 kips OK**

where:

- A_{fg} = 7.98 in.^{2} – gross area of tension flange (Section B4.3a)
- A_{fn} = 6.18 in.^{2} – net area of tension flange (Section B4.3b)
- F_u = 65 ksi – specified minimum tensile strength
- F_y = 50 ksi – specified minimum yield strength
- Y_t = 1.0 – beam flexural factor (= 1.0 for $F_y/F_u \leq 0.8$, otherwise = 1.1)
- b_f = 10.5 in. – beam flange width
- t_f = 0.76 in. – beam flange thickness
- S_x = 346 in.^{3} – minimum elastic section modulus of beam
- ϕ = 0.9 – strength factor (LRFD)

I.6.4 Column

I.6.4.1 Web panel zone shear

$R_n = 0.60 \cdot F_y \cdot d_c \cdot t_w$ for $P_r < 0.40 \cdot P_c$ (AISC 360-16, Eq. J10-9)

- $P_y = F_y \cdot A_g$ = (50 ksi)·(68.5 in.^{2}) = 3425 kips
- $P_r = 0 < 0.40 \cdot P_y$ = 0.40·(3425 kips) = 1370 kips
- $R_n = 0.60 \cdot F_y \cdot d_c \cdot t_w$
- R_n = 0.60·(50 ksi)·(16 in.)·(1.07 in.) = 513.6 kips
- ϕR_n = 0.9·(513.6 kips) = **462.24 kips > 11.49 kips OK**

where:

- A_g = 68.5 in.^{2} – column cross-sectional area
- P_r = 0 kips – required axial strength using LRFD
- P_y = 3425 – axial yield strength of the column
- d_c = 16 in. – depth of column
- t_w = 1.07 in. – thickness of column web
- F_y = 50 ksi – specified minimum yield strength
- ϕ = 0.9 – strength factor (LRFD)

I.6.4.2 Flange local bending

$R_n = 6.25 \cdot (F_{yf}) \cdot (t_f)^2$ (AISC 360-16, Eq. J10-1)

- R_n = 6.25·(50 ksi)·(1.72 in.^{2})^2
- R_n = 924.5 kips
- ϕR_n = (0.9)·924.5 kips
- ϕR_n = **832.05 kips > 11.49 kips OK**

where:

- F_{yf} = 50 ksi – specified minimum yield stress of the flange
- t_f = 1.72 in.^{2} – thickness of the column flange
- ϕ = 0.9 – strength factor (LRFD)

I.6.4.3 Web local yielding

$R_n = (F_{yw})\cdot(t_w)\cdot(5\cdot k + l_b)$ (AISC 360-16, Eq. J10-2)

- R_n = (50 ksi)·(1.07 in.)·(5·(2.32 in.) + 1.5 in.)
- R_n =700.85 kips
- ϕR_n = (1.0)·163.35 kips = **700.85 kips > 11.49 kips OK**

where:

- F_{yw} = 50 ksi – specified minimum yield strength of the web material
- t_w = 1.07 in.^{2} – thickness of column web
- k = 2.32 in. – distance from outer face of the flange to the web toe of the fillet
- l_b = 1. 5 in. – length of bearing (not less than k for end beam reactions)
- ϕ = 1.0 – strength factor (LRFD)

I.6.4.4 Web local crippling

$R_n = 0.80\cdot(t_w)^2\cdot[1+3\cdot(l_b/d)\cdot(t_w/t_f)^{1.5}]\cdot[E\cdot(F_{yw})\cdot(t_f)/(t_w)]^{1/2}\cdot Q_f$ (AISC 360-16, Eq. J10-4)

- R_n = 0.80·(1.07 in.)^2·[1+3·(1.5 in./16.04 in.)·(1.07 in./1.72 in.)$^{1.5}$]·[(2.9E+4 ksi)·(50 ksi)·(1.72 in.)/(1.07 in.)]$^{1/2}$·(1.0) = 1590.83 kips
- ϕR_n = 0.75·(1590.83 kips) = **1193.12 kips > 11.49 kips OK**

where:

- F_{yw} = 50 ksi – specified minimum yield strength of the web material
- t_w = 1.07 in. – thickness of column web
- t_f = 1.72 in. – thickness of column flange
- l_b = 1. 5 in. – length of bearing (not less than k for end beam reactions)
- d = 16.04 in. – full nominal depth of the member
- ϕ = 0.75 – strength factor (LRFD)
- E = 2.9E+4 ksi – Elastic Modulus
- Q_f = 1.0 – 1.0 for wide-flange sections and for HSS (connecting surface) in tension = as given in Table K3.2 (AISC 360-16) for all other HSS conditions

Appendix J. ASD Strength Calculations for Rigid Connection Test Specimen (Test No. BFP)

J.1 Loading and Assumptions

It is assumed that the double web-angle carries the applied shear force and the top and seat angles resist the applied moment. It is assumed that the following dead and live loads are applied 120 in. far away from the column centerline.

Vertical shear:

P_D = 10 kips (dead load)

P_L = 30 kips (live load)

Strong-axis moment:

The shear loads are applied 6 in. away from the boltline.

M_D = (10 kips)·(6 in.) = 60 kips-in. (dead load)

M_L = (60 kips)·(6 in.) = 180 kips-in. (live load)

J.1.1 ASD Load Demand or Design Loads

According to the Load and Resistance Factored Design (ASD) procedure (AISC 360-16 or AISC Manual 2017), ASD design load, factored load or demand, P_u under gravity loads is:

$P_a = P_D + P_L$ (ASD)

P_u = 10 kips + 30 kips = 40 kips (ASD)

$M_u = M_D + M_L$ (ASD)

M_u = 60 kips + 180 kips = 240 kips-in. (ASD)

Then, the required strength, R_u using the ASD load combinations will be set equal to the design shear force, V_u applied on the connection. Then, for the cantilever beam and connection specimen:

$R_u = V_u = P_u = P_D + P_L$ (ASD)

J.2. Single Plate Web Connection ASD Design Checks

J.2.1 Web Plate

J.2.1.1 Bolt shear

$R_n = F_n \cdot A_b$ (Eq. J3-1, AISC 360-16)

- Shear strength of one bolt:
- R_n/Ω = 68·0.442 / 2 = **15.03 kips/bolt**
- Total shear strength of eight bolts:
- R_n/Ω = (8 bolts)·15.03 kips = **120.22 kips > 40 kips OK**

where:

- F_{nv} = 68 ksi – nominal shear strength of fasteners (Table J3.2, AISC 360-16)
- A_b = 0.442 in.^{2} – nominal bolt area
- Ω = 2 – safety factor (ASD)

J.2.1.2 Bolt bearing

The nominal bearing strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 2.4 \cdot d \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6a)

- R_n =2.4·(3/4 in.)·(0.375 in.)·(65 ksi)
- R_n = 43.88 kips/bolt
- R_n/Ω = (8 bolts)·(43.88 kips) / 2
- R_n/Ω = **175.52 kips/connection > 40 kips OK**

where:

- d = 3/4 in. – nominal bolt diameter
- t = 0.375 in. – thickness of angle
- F_u = 58 ksi – specified minimum tensile stress of angle
- Ω = 2 – safety factor (ASD)

J.2.1.3 Bolt tearout

The available tearout strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 1.2 \cdot l_c \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6c)

- $L_{csv} = L_{sv} - d_h$
- L_{csv} = 3 in. - 13/16 in. = 2.19 in.
- $R_{n\text{-}span}$ = 1.2·(2.19 in.)·(0.375 in.)·(65 ksi)
- $R_{n\text{-}span}$ = 64.06 kips/bolt
- $L_{cev} = L_{ev} - d_h/2$
- L_{cev} = 1.25 in. - (13/16)/2 in. = 0.84 in.
- $R_{n\text{-}end}$ = 1.2·(0.84 in.)·(0.375 in.)·(65 ksi)
- $R_{n\text{-}end}$ = 24.57 kips/bolt

- R_n/Ω = [7·(64.06 kips) + (24.57 kips)] / 2
- R_n/Ω = **236.50 kips/conn. > 40 kips OK**

where:

- l_c : clear distance
- d_h = 13/16 in. – nominal hole dimension (3/4-in.-diameter bolt)
- L_{sv} = 3 in. – vertical center to center bolt distance
- L_{ev} = 1.25 in. – vertical edge distance of angle
- L_{csv} = 2.19 in. – clear vertical distance between bolts
- L_{cev} = 0.84 in. – clear vertical edge distance
- Ω = 2 – safety factor (ASD)

J.2.1.4 Shear yielding

$R_n = 0.60 \cdot F_y \cdot A_{gv}$ (AISC 360-16, Eq. J4-3)

- A_{gv} = (23.5 in.)·(0.375 in.) = 8.81 in.^{2}
- R_n/Ω = 0.60·(50 kips)·(8.81 in.^{2}) / 1.5
- R_n/Ω = **176.2 kips > 40 kips OK**

where:

- A_{gv} = 8.81 in.^{2} – gross area subject to shear
- Ω = 1.5 – safety factor (ASD)
- L = 23.5 in. – length of angle
- t = 0.375 in. – thickness of angle
- F_y = 50 ksi – specified minimum yield strength of plate

J.2.1.5 Shear rupture

$R_n = 0.60 \cdot F_u \cdot A_{nv}$ (AISC 360-16, Eq. J4-4)

- $L_h = d_h + 1/16$ (Section B4-3b, AISC Manual)
- L_h = 0.813 + 1/16 = 0.875 in.
- $L_e = L - n \cdot L_h$
- L_e = 23.5 - 8·0.875 = 16.5 in.
- $A_{nv} = L_e \cdot t_p$
- A_{nv} = (16.5 in.)·(0.375 in.) = 6.19 in.^{2}
- R_n/Ω = 0.60·(65 kips)·(6.19 in.^{2}) / 2
- R_n/Ω = **120.71 kips > 40 kips OK**

where:

- A_{nv} = 6.19 in.^{2} – net area subject to shear, in.^{2}

- L_h = 0.875 in. – hole dimension for tension and shear net area
- L_e = 16.5 in. – effective length
- Ω = 2 – safety factor (ASD)
- n = 8 – number of the bolts
- t = 0.375 in. – thickness of angle

J.2.1.6 Block shear

$R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$ (AISC 360-16, Eq. J4-5)

- $A_{gv} = (L\text{-}L_{ev}) \cdot (t_p)$
- A_{gv} = (23.5 in. - 1.25 in.)·(0.375 in.) = 8.34 in.^{2}
- $A_{nv} = A_{gv}$ - (n-0.5)·(d_h + 1/16 in.)·(t_p)
- A_{nv} = 8.34 in.^{2} - (8-0.5)·(0.813 + 1/16 in.)·(0.375 in.)
- A_{nv} = 5.88 in.^{2}
- A_{nt} = [L_{eh} - 0.5·(d_h + 1/16 in.)]·(t_p)
- A_{nt} = [1.5 in. - 0.5·(0.813 in. + 1/16 in.)]·(0.375 in.)
- A_{nt} = 0.39 in.^{2}
- U_{bs} = 1 (as the tension stress is uniform)
- $R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$
- R_n = 0.60·(65 kips/in.^{2})·(5.88 in.^{2}) + 1·(65 kips/in.^{2})·(0.39 in.^{2}) ≤ 0.60·(50 kips/in.^{2})·(8.34 in.^{2}) + 1·(65 kips/in.^{2})·(0.39 in.^{2})
- R_n = **254.67 kips** ≤ 275.55 kips
- R_n/Ω = (254.67 kips) / 2
- R_n/Ω **= 127.34 kips > 40 kips OK**

where:

- U_{bs} = 1 – Stress index for uniform tension stress
- A_{nt} = 0.39 in.^{2} – net area subject to tension
- A_{gv} = 8.34 in.^{2} – gross area subject to shear
- A_{nv} = 5.88 in.^{2} – net area subject to shear
- L_{ev} = 1.25 in. – vertical edge distance of angle
- L_{eh} = 1.5 in. – horizontal edge distance of angle
- Ω = 2 – safety factor (ASD)

J.2.1.7 Weld Shear (web plate to the column flange)

- $R_n/\Omega = (2 \text{ welds}) \cdot [(F_{EXX}) \cdot 0.6 \cdot (D/16) \cdot L \cdot (2^{1/2}/2)] / 2$ (Eq. 8-2a/8-2b, AISC Manual)
- R_n/Ω = (2 welds)·(0.928·D·L)
- R_n/Ω = (2 welds)·(0.928·4·23.5)
- R_n/Ω = **174.46 kips > 40 kips OK**

where:

- $D = 4$ – number of sixteenths-of-an-inch in the weld size
- $L = 23.5$ in. – length of the angles
- $F_{EXX} = 70$ ksi – filler metal electrode classification strength
- $\Omega = 2$ – safety factor (ASD)

J.2.2 Beam

J.2.2.1 Bolt bearing

The nominal bearing strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 2.4 \cdot d \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6a)

- $R_n = 2.4 \cdot (3/4 \text{ in.}) \cdot (0.545 \text{ in.}) \cdot (58 \text{ ksi})$
- $R_n = 56.90$ kips/bolt
- $R_n/\Omega = (8 \text{ bolts}) \cdot (56.90 \text{ kips}) / 2$
- $R_n/\Omega =$ **227.6 kips/connection > 40 kips OK**

where:

- $d = 3/4$ in. – nominal bolt diameter
- $t_w = 0.545$ in. – thickness of beam web
- $F_u = 58$ ksi – specified minimum tensile stress of angle
- $\Omega = 2$ – safety factor (ASD)

J.2.2.2 Bolt tearout

The available tearout strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 1.2 \cdot l_c \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6c)

- $L_{csv} = L_{sv} - d_h$
- $L_{csv} = 3 \text{ in.} - 13/16 \text{ in.} = 2.19$ in.
- $R_{n\text{-}span} = 1.2 \cdot (2.19 \text{ in.}) \cdot (0.545 \text{ in.}) \cdot (65 \text{ ksi})$
- $R_{n\text{-}span} = 93.10$ kips/bolt
- $L_{cev} = L_{ev} - d_h/2$
- $L_{cev} = 4.4 \text{ in.} - (13/16)/2 \text{ in.} = 3.99$ in.
- $R_{n\text{-}end} = 1.2 \cdot (3.99 \text{ in.}) \cdot (0.545 \text{ in.}) \cdot (65 \text{ ksi})$
- $R_{n\text{-}end} = 169.62$ kips/bolt
- $R_n/\Omega = [7 \cdot (93.10 \text{ kips}) + (169.62 \text{ kips})] / 2$
- $R_n/\Omega =$ **410.66 kips/conn. > 40 kips OK**

where:

- l_c : clear distance
- d_h = 13/16 in. – nominal hole dimension (3/4-in.-diameter bolt)
- L_{sv} = 3 in. – vertical center to center bolt distance
- L_{ev} = 1.25 in. – vertical edge distance of angle
- L_{csv} = 2.19 in. – clear vertical distance between bolts
- L_{cev} = 3.99 in. – clear vertical edge distance
- $\Omega = 2$ – safety factor (ASD)
- t_w = 0.545 in. – thickness of beam web
- F_u = 65 ksi – specified minimum tensile stress of beam

J.2.2.3 Shear yielding

$R_n = 0.60 \cdot F_y \cdot A_{gv}$ (AISC 360-16, Eq. J4-3)

- A_{gv} = (29.83 in.)·(0.545 in.) = 16.26 in.^{2}
- R_n/Ω = 0.60·(50 kips)·(16.26 in.^{2}) / 1.5
- R_n/Ω = **325.2 kips > 40 kips OK**

where:

- A_{gv} = 16.26 in.^{2} – gross area subject to shear
- $\Omega = 1.5$ – safety factor (ASD)
- d = 29.83 in. – beam depth
- t_w = 0.545 in. – thickness of beam web
- F_y = 50 ksi – specified minimum yield strength

J.3. Flange Plate Connection ASD Design Checks

J.3.1 Design Forces

J.3.1.1 Flange Bolt Shear Force

The moment arm between flange forces is equal to the depth of the beam.

$P_{uf} = M_u/d_m$

- P_{uf} = 240 kips-in. / 29.83 in. = **8.05 kips**

where:

- d_m = 29.83 in. – depth of beam
- P_{uf} = 12.07 kips – flange bolt shear

J.3.1.2 Flange Tension Force

The moment arm between flange forces is equal to the depth of the beam plus one plate thickness.

$P_{uf} = M_u/(d + t)$

- P_{uf} = 240 kips-in. / (29.83 in. + 1.5 in.) = **7.66 kips**

where:

- d_m = 29.83 in – depth of beam
- t = 1.5 in. – plate thickness
- P_{uf} = 11.49 kips – flange tension force

J.3.2 Flange Plate

J.3.2.1 Bolt shear

$R_n = F_n \cdot A_b$ (Eq. J3-1, AISC 360-16)

- Shear strength of one bolt:
- R_n/Ω = (84 ksi)·(0.785 in.^{2}) / 2 = 32.97 kips/bolt
- Total shear strength of eight bolts:
- R_n/Ω = (14 bolts)·(32.97 kips) = **461.58 kips > 8.05 kips OK**

where:

- F_{nv} = 68 ksi – nominal shear strength of fasteners (Table J3.2, AISC 360-16)
- A_b = 0.785 in.^{2} – nominal bolt area
- Ω = 2 – safety factor (ASD)

J.3.2.2 Bearing

The nominal bearing strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 2.4 \cdot d \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6a)

- R_n = 2.4·(1 in.)·(1.5 in.)·(65 ksi)
- R_n = 234 kips/bolt
- R_n/Ω = (14 bolts)·(234 kips) / 2
- R_n/Ω = **1638.0 kips/connection > 8.05 kips OK**

where:

- d = 1 in. – nominal bolt diameter
- t = 1.5 in. – plate thickness
- F_u = 65 ksi – specified minimum tensile stress of plate
- Ω = 2 – safety factor (ASD)

J.3.2.3 Tearout

The available tearout strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 1.2 \cdot l_c \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6c)

- $L_{csv} = L_{sv} - d_h$

- L_{csv} = 3 in. - 1.125 in. = 1.875 in.
- $R_{n\text{-}span}$ = 1.2·(1.875 in.)·(1.5 in.)·(65 ksi)
- $R_{n\text{-}span}$ = 219.38 kips/bolt
- $L_{cev} = L_{ev} - d_h/2$
- L_{cev} = 3 in. - (1.125 in.)/2 = 2.44 in.
- $R_{n\text{-}end}$ = 1.2·(2.44 in.)·(1.5 in.)·(65 ksi)
- $R_{n\text{-}end}$ = 285.48 kips/bolt
- R_n/Ω = (2)·[6·(219.38 kips) + (285.48 kips)] / 2
- R_n/Ω = **1601.76 kips/conn. > 8.05 kips OK**

where:

- l_c : clear distance
- d_h = 1.125 in. – nominal hole dimension (1-in.-diameter bolt)
- L_{sv} = 3 in. – vertical center to center bolt distance
- L_{ev} = 3 in. – vertical edge distance of angle
- L_{csv} = 1.875 in. – clear vertical distance between bolts
- L_{cev} = 2.44 in. – clear vertical edge distance
- Ω = 2 – safety factor (ASD)

J.3.2.4 Tensile yielding

$R_n = 0.60 \cdot F_y \cdot A_{gv}$ (AISC 360-16, Eq. J4-3)

- A_{gv} = (12 in.)·(1.5 in.) = 18 in.^{2}
- R_n/Ω = (50 kips)·(18 in.^{2}) / 1.67
- R_n/Ω = **538.92 kips > 7.66 kips OK**

where:

- A_{gv} = 18 in.^{2} – gross area subject to shear
- Ω = 1.67 – safety factor (ASD)
- L = 12 in. – length of angle
- t = 1.5 in. – flange thickness
- F_y = 50 ksi – specified minimum yield strength

J.3.2.5 Tension rupture

$R_n = F_u \cdot A_e$ (AISC 360-16, Eq. J4-2)

- A_g = (12 in.)·(1.5 in.) = 18 in.^{2}
- $L_h = d_h + 1/16$ (Section B4-3b, AISC Manual)
- L_h = 1.125 + 1/16 = 1.19 in.
- $A_n = (L - n \cdot L_h) \cdot t_p$

- A_n = (12 in. - 2·1.19 in.)·(1.5 in.) = 14.44 in.^{2}
- $A_e = A_n \leq 0.85A_g$ = 15.3 in.^{2} (AISC 360-16, Sec. J4-1)
- A_e = 14.44 in.^{2}
- R_n = (65 ksi)·(14.44 in.^{2}) = 938.6 kips
- R_n/Ω = (938.6 ksi) / 2 = **469.3 kips** **> 7.66 kips** **OK**

where:

- A_g = 18 in.^{2} – gross area
- A_e = 14.44 in.^{2} – effective net area
- L_h = 1.19 in. – hole dimension
- F_u = 65 ksi – specified minimum tensile stress of plate
- Ω = 2 – safety factor (ASD)

J.3.2.6 Block shear

$R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$ (AISC 360-16, Eq. J4-5)

- $A_{gv} = (L\text{-}L_{ev}) \cdot (t)$
- A_{gv} = (2 planes)·(24 in. - 3 in.)·(1.5 in.) = 63 in.^{2}
- $A_{nv} = A_{gv}$ - (n-0.5)·(d_h + 1/16 in.)·(t)
- A_{nv} = 63 in.^{2} - (2 planes)·(7-0.5)·(1.125 + 1/16 in.)·(1.5 in.)
- A_{nv} = 39.84 in.^{2}
- A_{nt} = [min(g, b - g) - 1.0·(d_h + 1/16 in.)]·(t)
- A_{nt} = [6 in. - 1.0·(1.125 in. + 1/16 in.)]·(1.5 in.)
- A_{nt} = 7.22 in.^{2}
- U_{bs} = 1 (as the tension stress is uniform)
- $R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$
- R_n = 0.60·(65 kips/in.^{2})·(39.84 in.^{2}) + 1·(65 kips/in.^{2})·(7.22 in.^{2}) ≤ 0.60·(50 kips/in.^{2})·(63 in.^{2}) + 1·(65 kips/in.^{2})·(7.22 in.^{2})
- R_n = **2023.06 kips** ≤ 2359.30 kips
- R_n/Ω = (2023.06 kips) / 2
- R_n/Ω **= 1011.53 kips > 7.66 kips** **OK**

where:

- U_{bs} = 1 – Stress index for uniform tension stress
- A_{nt} = 7.22 in.^{2} – net area subject to tension
- A_{gv} = 63 in.^{2} – gross area subject to tension
- A_{nv} = 39.84 in.^{2} – net area subject to tension
- L_{ev} = 3 in. – vertical edge distance of plate
- L_{eh} = 3 in. – horizontal edge distance of plate
- Ω = 2 – safety factor (ASD)

- g = 6 in. – gage between transfer bolts of plate
- b = 12 in. – width of plate

J.3.2.7 Compression

$P_n = F_y{\cdot}A_g$ when $L_c/r \leq 25$ (AISC 360-16, Sec. J4-4)

- $r = t_p/(12)^{1/2}$ = 1.5 in./$(12)^{1/2}$
- r = 0.43 in.
- $L_c = K{\cdot}L/r$ = 0.65·(3 in.)/(0.43 in.)
- L_c = 4.43 < 25 **NOT OK**
- $A_g = L_p{\cdot}t_p$ = (12 in.)·(1.5 in.)
- A_g = 18 in.^{2}
- $R_n/\Omega = F_y{\cdot}A_g$ = (50 ksi)·(18 in.^{2}) / 1.67
- R_n/Ω = **538.02 kips > 7.66 kips OK**

where:

- K = 0.65 – effective length factor
- L = 3 in. – distance between adjacent bolt holes
- L_c = 4.43 – effective length
- A_g = 18 in.^{2} – gross area
- Ω = 1.67 – safety factor (ASD)

J.3.3 Beam

J.3.3.1 Bearing

The nominal bearing strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 2.4{\cdot}d{\cdot}t{\cdot}F_u$ (AISC 360-16, Eq. J3-6a)

- R_n = 2.4·(1 in.)·(0.76 in.)·(65 ksi)
- R_n = 118.56 kips/bolt
- R_n/Ω = (14 bolts)·(118.56 kips) / 2
- R_n/Ω = **829.92 kips/connection > 8.05 kips OK**

where:

- d = 1 in. – nominal bolt diameter
- t_p = 0.76 in. – thickness of beam flange
- F_u = 58 ksi – specified minimum tensile stress of beam
- Ω = 2 – safety factor (ASD)

J.3.3.2 Tearout

The available tearout strength of the beam web per bolt is determined from AISC Specification Section J3.10.

$R_n = 1.2 \cdot l_c \cdot t \cdot F_u$ (AISC 360-16, Eq. J3-6c)

- $L_{csv} = L_{sv} - d_h$
- L_{csv} = 3 in. - 1.125 in. = 1.875 in.
- $R_{n\text{-}span}$ = 1.2·(1.875 in.)·(0.76 in.)·(65 ksi)
- $R_{n\text{-}span}$ = 111.15 kips/bolt
- $L_{cev} = L_{ev} - d_h/2$
- L_{cev} = 2 in. - (1.125 in.)/2 = 1.44 in.
- $R_{n\text{-}end}$ = 1.2·(1.44 in.)·(0.76 in.)·(65 ksi)
- $R_{n\text{-}end}$ = 85.22 kips/bolt
- R_n/Ω = (2)·[6·(111.15 kips) + (85.22 kips)] / 2
- R_n/Ω = **752.12 kips/conn. > 8.05 kips** **OK**

where:

- l_c : clear distance
- d_h = 1.125 in. – nominal hole dimension (1-in.-diameter bolt)
- L_{sv} = 3 in. – vertical center to center bolt distance
- L_{ev} = 2 in. – vertical edge distance of beam
- L_{csv} = 1.875 in. – clear vertical distance between bolts
- L_{cev} = 2.44 in. – clear vertical edge distance
- Ω = 2 – safety factor (ASD)

J.3.3.3 Block shear

$R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$ (AISC 360-16, Eq. J4-5)

- $A_{gv} = (L_{ev}+(n\text{-}1) \cdot s) \cdot (t)$
- A_{gv} = (2 planes)·(2 in. +(7-1)·3 in.)·(0.76 in.) = 30.4 in.^{2}
- $A_{nv} = A_{gv}$ - (n-0.5)·(d_h + 1/16 in.)·(t)
- A_{nv} = 30.4 in.^{2} - (2)·(7-0.5)·(1.125 + 1/16 in.)·(0.76 in.)
- A_{nv} = 18.67 in.^{2}
- A_{nt} = [(b - g) - 1.0·(d_h + 1/16 in.)]·(t)
- A_{nt} = [(10.5 in. - 6 in.) - 1.0·(1.125 in. + 1/16 in.)]·(0.76 in.)
- A_{nt} = 2.52 in.^{2}
- U_{bs} = 1 (as the tension stress is uniform)
- $R_n = 0.60 \cdot F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \leq 0.60 \cdot F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$
- R_n = 0.60·(65 kips/in.^{2})·(18.67 in.^{2}) + 1·(65 kips/in.^{2})·(2.52 in.^{2}) ≤ 0.60·(50 kips/in.^{2})·(30.4 in.^{2}) + 1·(65 kips/in.^{2})·(2.52 in.^{2})
- R_n = **891.93 kips** ≤ 1075.8 kips

- R_n/Ω = (891.93 kips) / 2
- R_n/Ω **= 445.97 kips > 8.05 kips** **OK**

where:

- U_{bs} = 1 – Stress index for uniform tension stress
- A_{nt} = 2.52 in.^{2} – net area subject to tension
- A_{gv} = 30.4 in.^{2} – gross area subject to tension
- A_{nv} = 18.67 in.^{2} – net area subject to tension
- L_{ev} = 2 in. – vertical edge distance of plate
- L_{eh} = 3 in. – horizontal edge distance of plate
- Ω = 2 – safety factor (ASD)
- g = 6 in. – gage between transfer bolts of plate
- b = 10.5 in. – width of beam flange
- s = 3 in. – spacing between bolts

J.3.3.3 Flexural strength

$M_n = S_x \cdot F_u \cdot A_{fn}/A_{fg}$ when $F_u \cdot A_{fn} < Y_t \cdot F_y \cdot A_{fg}$ (AISC 360-16, Eq. F13-1)

Otherwise, the limit state of tensile rupture does not apply

- $A_{fg} = (b_f)\cdot(t_f)$
- A_{fg} = (10.5 in.)·(0.76 in.) = 7.98 in.^{2}
- $A_{fn} = A_{fg}$ - n·(d_h + 1/16 in.)·(t_p)
- A_{fn} = 7.98 in.^{2} - 2·(1.125 in. + 1/16 in.)·(0.76 in.) = 6.18 in.^{2}
- F_y/F_u = (50 ksi)/(65 ksi) = 0.77 < 0.8 **therefore Y_t = 1.0**
- $F_u \cdot A_{fn}$ = (65 ksi)·(6.18 in.^{2}) = 401.7 kips
- $Y_t \cdot F_y \cdot A_{fg}$ = 1.0·(50 ksi)·(7.98 in.^{2}) = 399 kips
- $F_u \cdot A_{fn}$ = 401.7 kips < $Y_t \cdot F_y \cdot A_{fg}$ = 399 kips **NOT OK**
- $M_n/\Omega = F_y \cdot S_x/\Omega$ = [(50 ksi)·(346 in.^{3}) / 12] / 1.67
- M_n/Ω = **863.27 kips-ft > 8.05 kips** **OK**

where:

- A_{fg} = 7.98 in.^{2} – gross area of tension flange (Section B4.3a)
- A_{fn} = 6.18 in.^{2} – net area of tension flange (Section B4.3b)
- F_u = 65 ksi – specified minimum tensile strength
- F_y = 50 ksi – specified minimum yield strength
- Y_t = 1.0 – beam flexural factor (= 1.0 for $F_y/F_u \leq 0.8$, otherwise = 1.1)
- b_f = 10.5 in. – beam flange width
- t_f = 0.76 in. – beam flange thickness
- S_x = 346 in.^{3} – minimum elastic section modulus of beam
- Ω = 1.67 – safety factor (ASD)

J.3.4 Column

J.3.4.1 Web panel zone shear

$R_n = 0.60{\cdot}F_y{\cdot}d_c{\cdot}t_w$ for $P_r < 0.40{\cdot}P_c$ (AISC 360-16, Eq. J10-9)

- $P_y = F_y{\cdot}A_g$ = (50 ksi)·(68.5 in.^{2}) = 3425 kips
- $P_r = 0 < 0.40{\cdot}P_y$ = 0.40·(3425 kips) = 1370 kips
- $R_n = 0.60{\cdot}F_y{\cdot}d_c{\cdot}t_w$
- R_n = 0.60·(50 ksi)·(16 in.)·(1.07 in.) = 513.6 kips
- R_n/Ω = (513.6 kips) / 1.67 = **307.54 kips > 7.66 kips** **OK**

where:

- A_g = 68.5 in.^{2} – column cross-sectional area
- P_r = 0 kips – required axial strength
- P_y = 3425 – axial yield strength of the column
- d_c = 16 in. – depth of column
- t_w = 1.07 in. – thickness of column web
- F_y = 50 ksi – specified minimum yield strength
- Ω = 1.67 – safety factor (ASD)

J.3.4.2 Flange local bending

$R_n = 6.25{\cdot}(F_{yf}){\cdot}(t_f)^2$ (AISC 360-16, Eq. J10-1)

- R_n = 6.25·(50 ksi)·(1.72 in.^{2})^2
- R_n = 924.5 kips
- R_n/Ω = (924.5 kips) / 1.67
- R_n/Ω = **553.59 kips > 7.66 kips** **OK**

where:

- F_{yf} = 50 ksi – specified minimum yield stress of the flange
- t_f = 1.72 in.^{2} – thickness of the column flange
- Ω = 1.67 – safety factor (ASD)

J.3.4.3 Web local yielding

$R_n = (F_{yw}){\cdot}(t_w){\cdot}(5{\cdot}k + l_b)$ (AISC 360-16, Eq. J10-2)

- R_n = (50 ksi)·(1.07 in.)·(5·(2.32 in.) + 1.5 in.)
- R_n =700.85 kips
- R_n/Ω = (700.85 kips) / 1.5 = **467.23 kips > 7.66 kips** **OK**

where:

- F_{yw} = 50 ksi – specified minimum yield strength of the web material
- t_w = 1.07 in.[2] – thickness of column web
- k = 2.32 in. – distance from outer face of the flange to the web toe of the fillet
- l_b = 1.5 in. – length of bearing (not less than k for end beam reactions)
- Ω = 1.5 – safety factor (ASD)

J.3.4.4 Web local crippling

$R_n = 0.80 \cdot (t_w)^2 \cdot [1+3 \cdot (l_b/d) \cdot (t_w/t_f)^{1.5}] \cdot [E \cdot (F_{yw}) \cdot (t_f)/(t_w)]^{1/2} \cdot Q_f$ (AISC 360-16, Eq. J10-4)

- $R_n = 0.80 \cdot (1.07 \text{ in.})^2 \cdot [1+3 \cdot (1.5 \text{ in.}/16.04 \text{ in.}) \cdot (1.07 \text{ in.}/1.72 \text{ in.})^{1.5}] \cdot [(2.9E+4 \text{ ksi}) \cdot (50 \text{ ksi}) \cdot (1.72 \text{ in.})/(1.07 \text{ in.})]^{1/2} \cdot (1.0)$ = 1590.83 kips
- R_n/Ω = (1590.83 kips) / 2 = **795.42 kips > 7.66 kips** **OK**

where:

- F_{yw} = 50 ksi – specified minimum yield strength of the web material
- t_w = 1.07 in. – thickness of column web
- t_f = 1.72 in. – thickness of column flange
- l_b = 1. 5 in. – length of bearing (not less than k for end beam reactions)
- d = 16.04 in. – full nominal depth of the member
- Ω = 2 – safety factor (ASD)
- E = 2.9E+4 ksi – Elastic Modulus
- Q_f = 1.0 – 1.0 for wide-flange sections and for HSS (connecting surface) in tension = as given in Table K3.2 (AISC 360-16) for all other HSS conditions

Appendix K. IDEA StatiCa Model

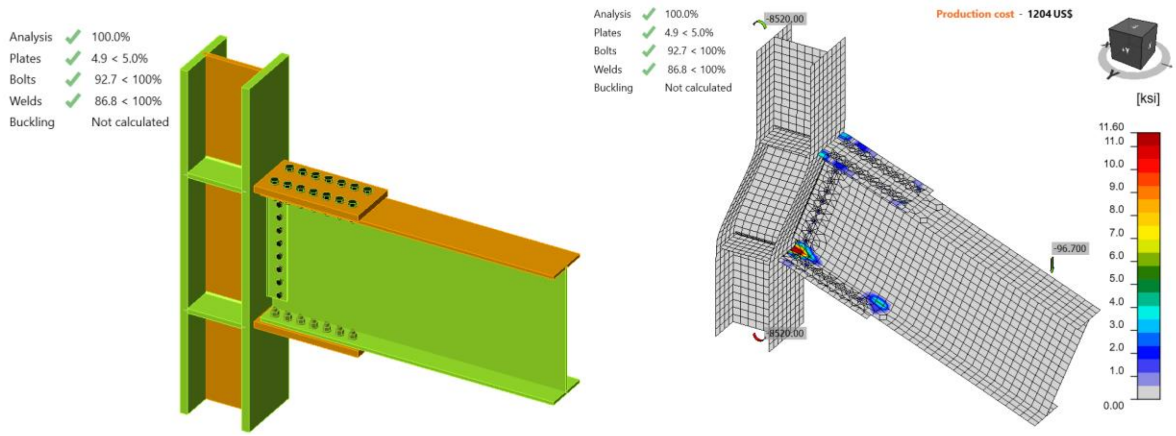


Figure K.1: IDEA StatiCa BFP and stress in contact distribution

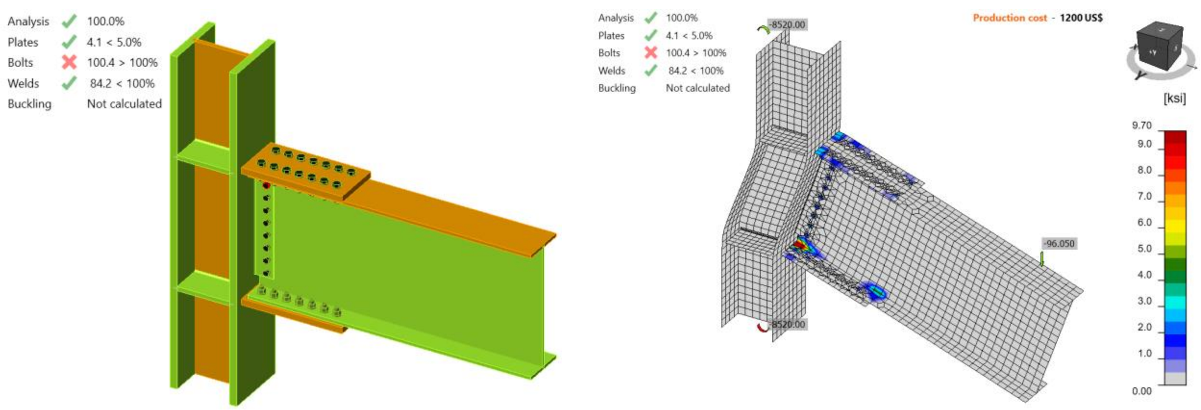


Figure K.2: IDEA StatiCa Model 1 and stress in contact distribution (LRFD)

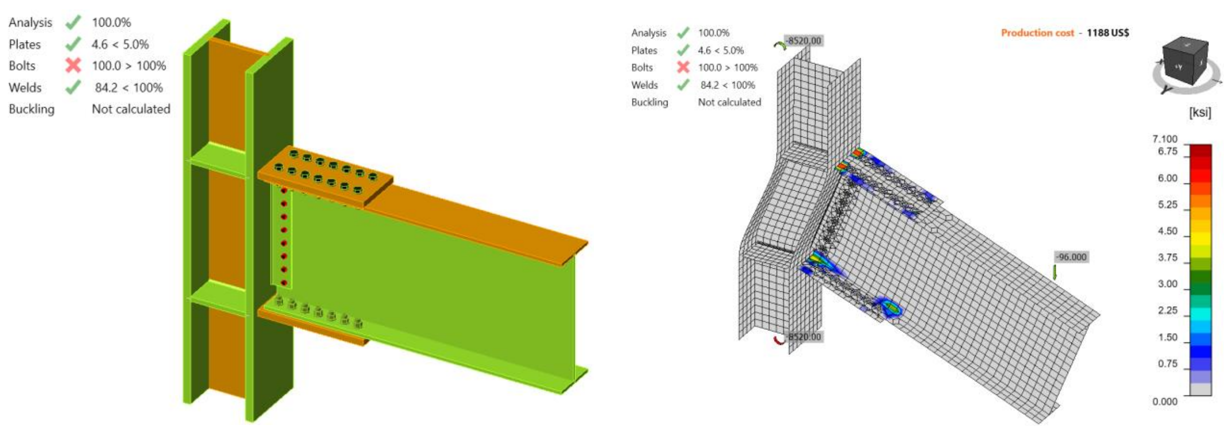


Figure K.3: IDEA StatiCa Model 2 and stress in contact distribution (LRFD)

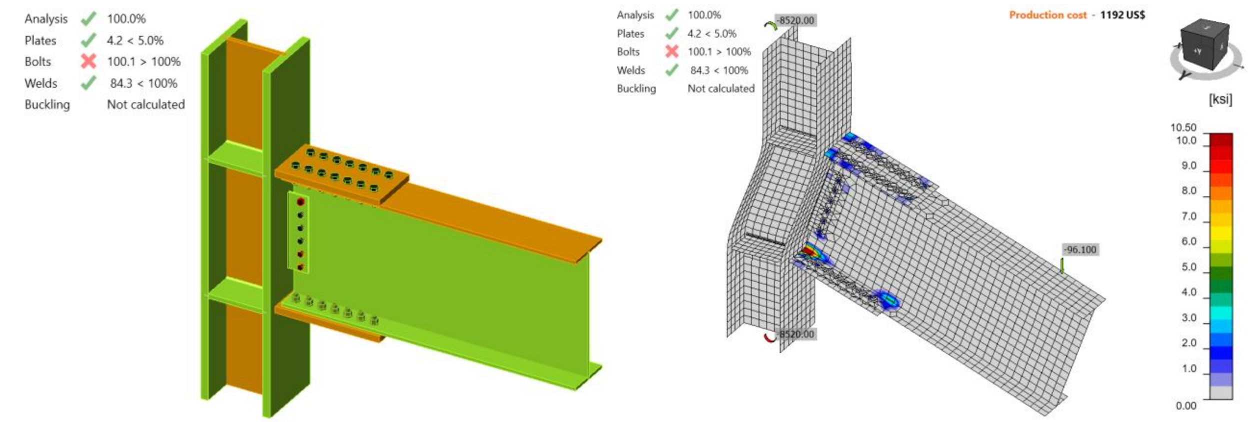


Figure K.4: IDEA StatiCa Model 3 and stress in contact distribution (LRFD)

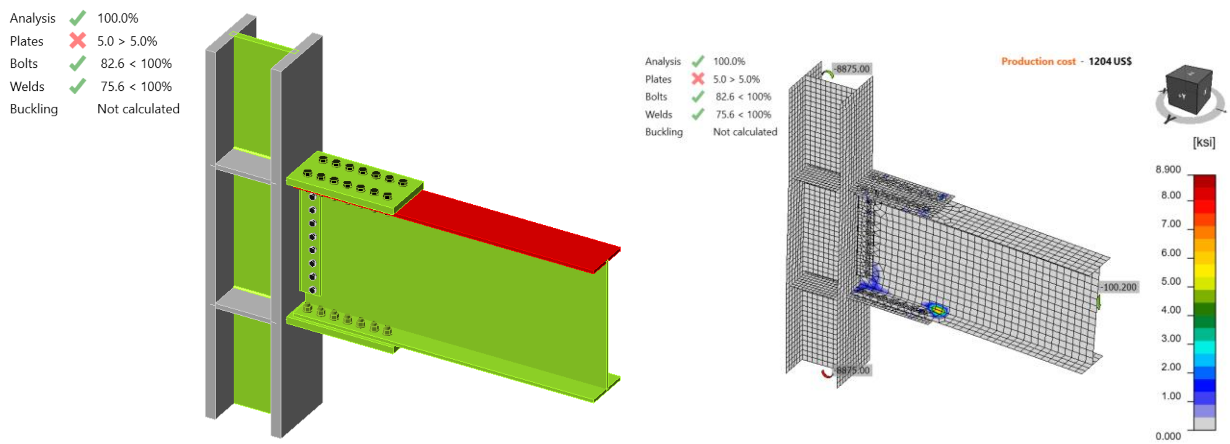


Figure K.5: IDEA StatiCa Model 4 and stress in contact distribution (LRFD)

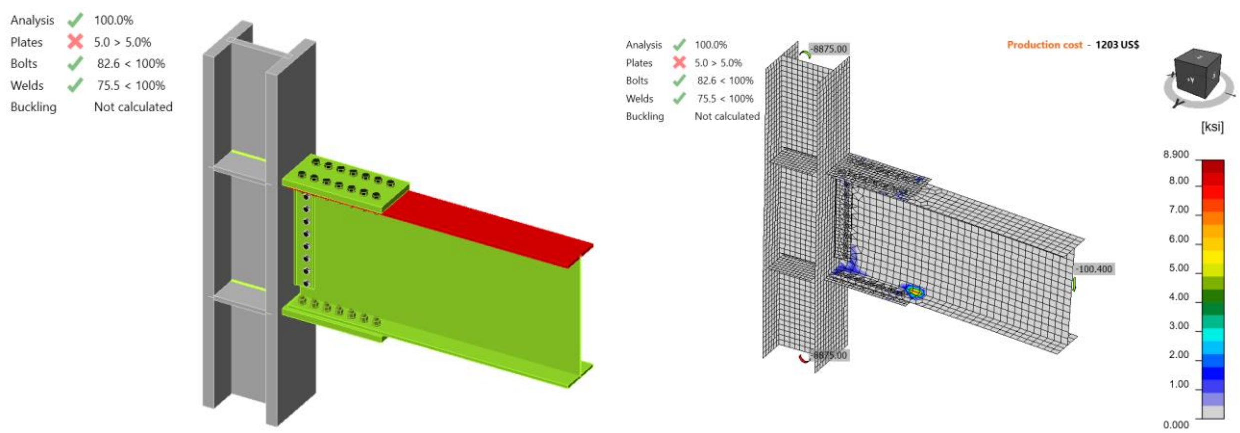


Figure K.6: IDEA StatiCa Model 5 and stress in contact distribution (LRFD)

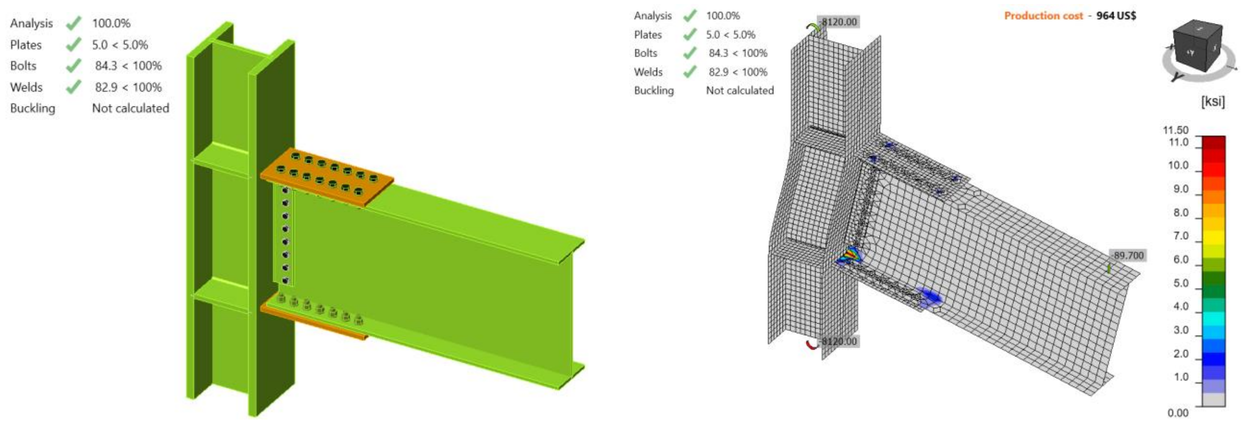


Figure K.7: IDEA StatiCa Model 6 and stress in contact distribution (LRFD)

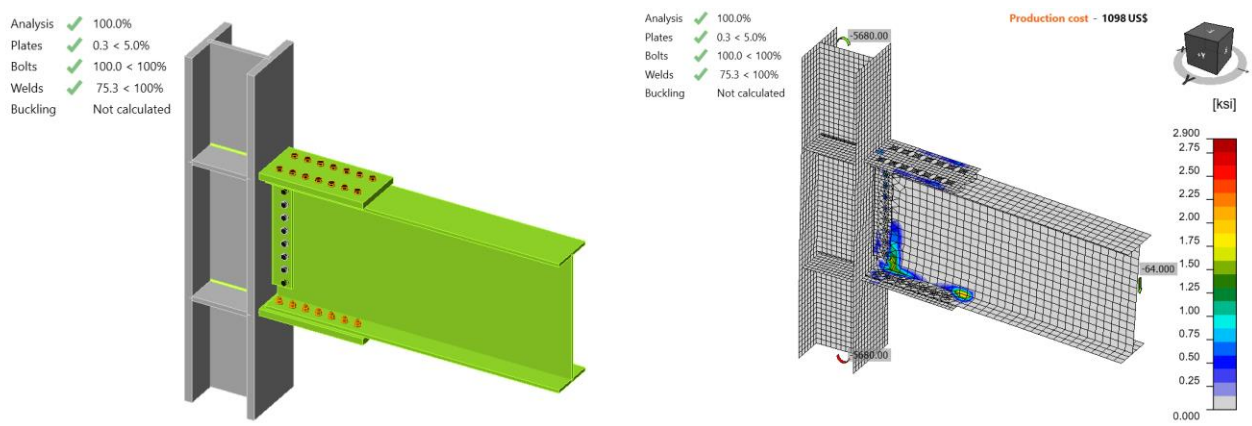


Figure K.8: IDEA StatiCa Model 7 and stress in contact distribution (LRFD)

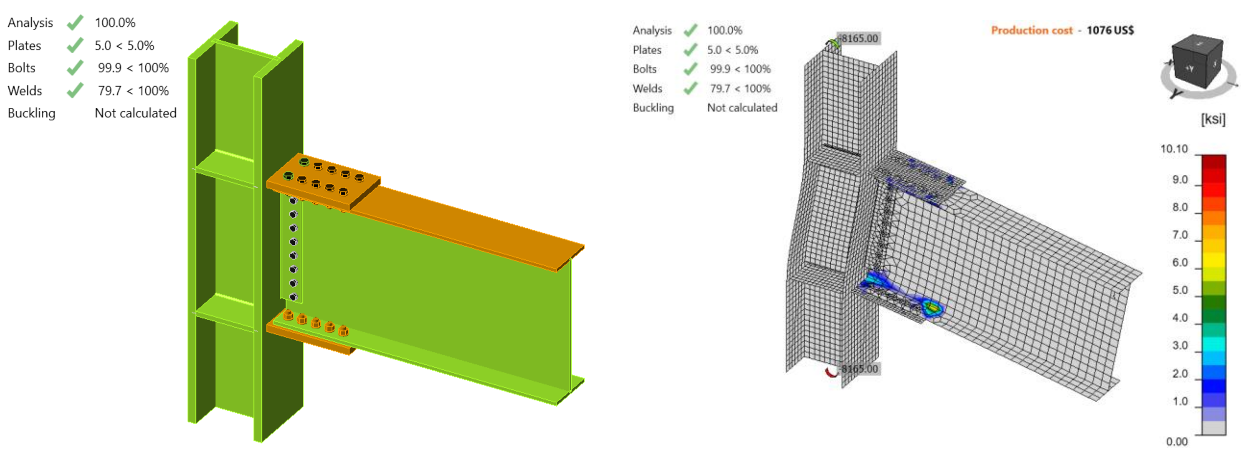


Figure K.9: IDEA StatiCa Model 8 and stress in contact distribution (LRFD)

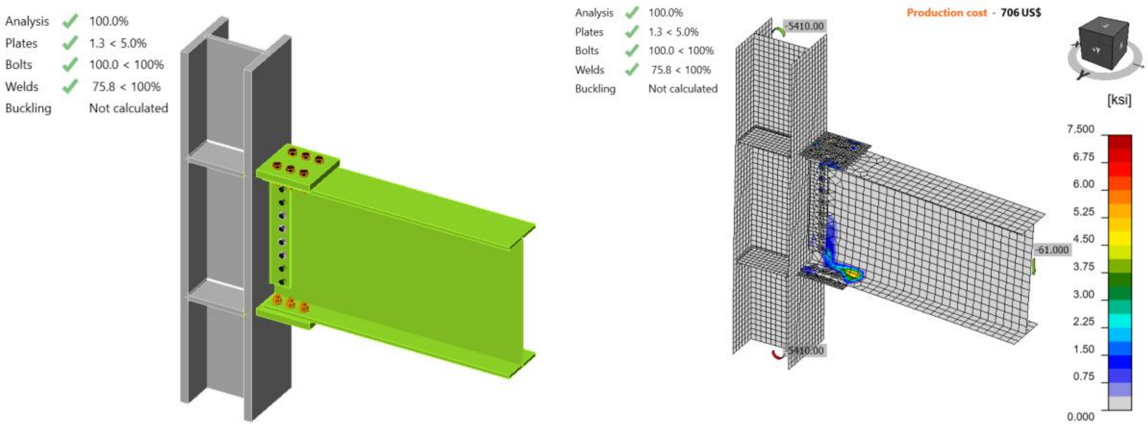


Figure K.10: IDEA StatiCa Model 9 and stress in contact distribution (LRFD)

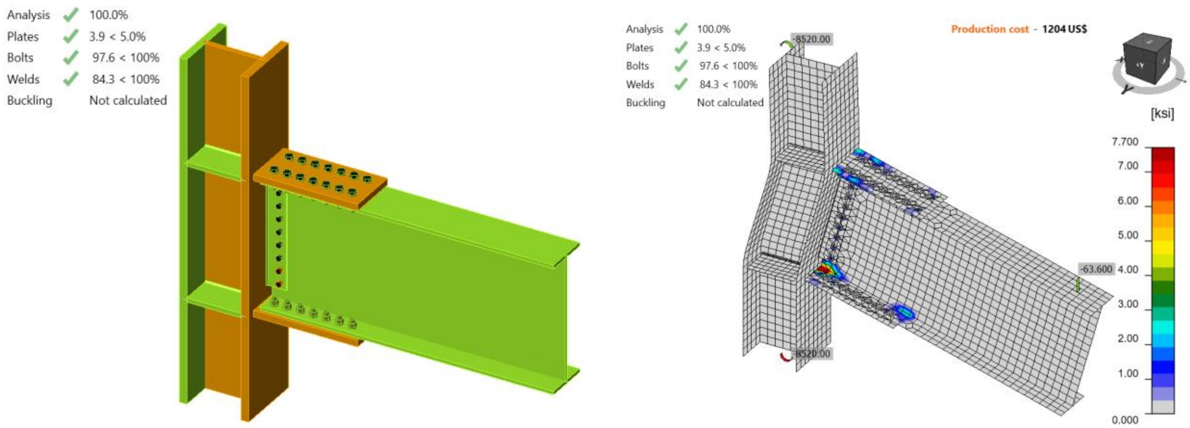
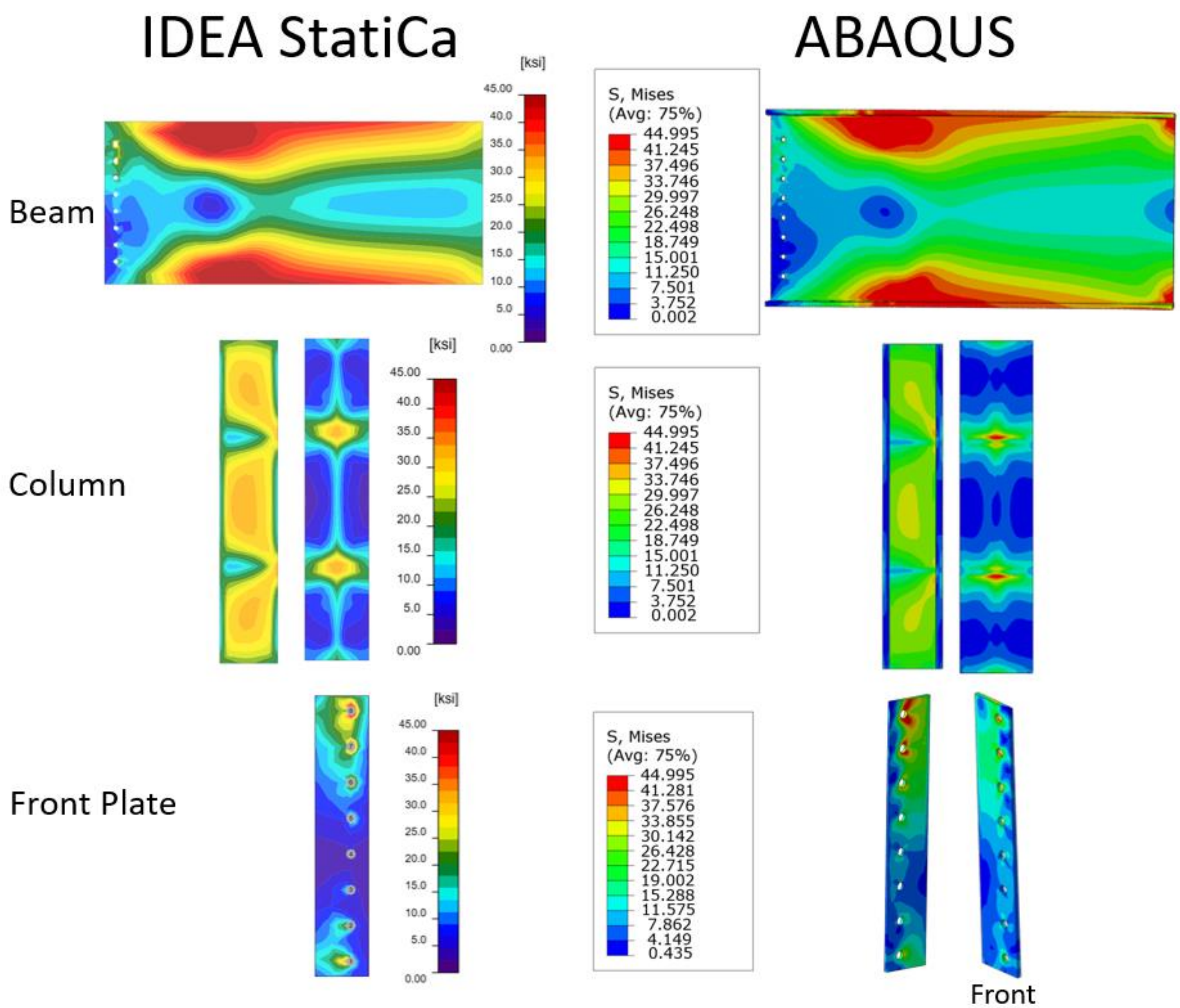


Figure K.11: IDEA StatiCa BFP and stress in contact distribution (ASD)

Appendix L. Comparison of ABAQUS and IDEA StatiCa Models



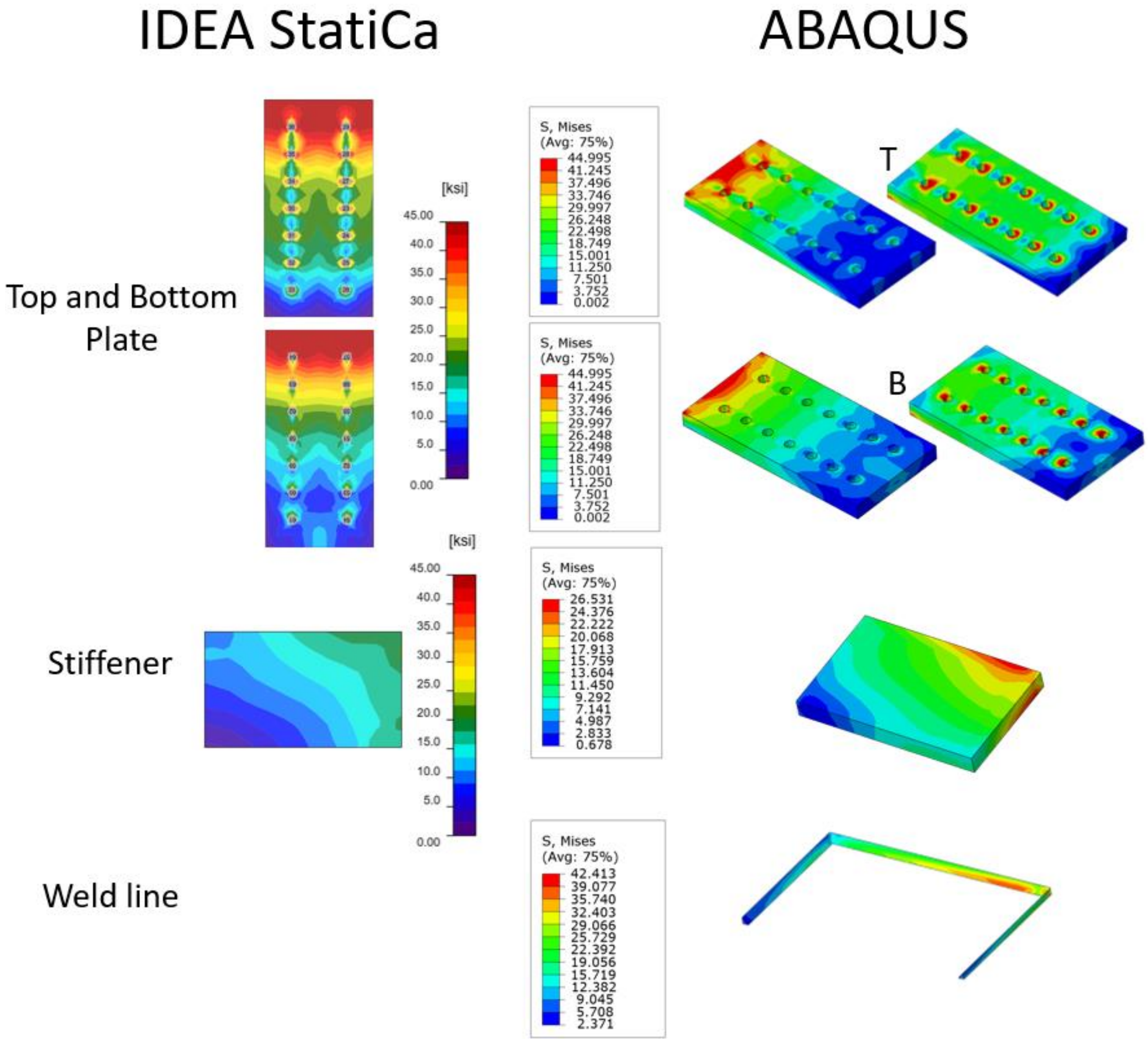


Figure L.1: Predicted stress between IDEA StatiCa and ABAQUS

Note 1: In ABAQUS, node set was defined in different positions (e.g., shear plains) in order to extract data (i.e., nodal forces, shear forces, etc.) from the model and calculate the bolt loads.

Note 2: In ABAQUS, to calculate the weld capacity, first the element with maximum stress was identified (critical element). Then, the resultant force and its angle with the weld longitudinal axis was obtained and approximated, respectively.