AISC – verification examples

IDEA StatiCa

November 2, 2020

1 Bolted flange plate moment connection – LRFD

A beam with cross-section W12×40 is connected to a column with cross-section W10×45. The joint is designed as a moment connection and is realized as bolted flange plate moment connection. All steel is grade A36 ($f_y = 36$ ksi, $f_u = 58$ ksi) and bolts are grade A307 ($f_y = 50$ ksi, $f_u = 65$ ksi). Fin plates at the beam flanges are with the thickness of 5/8" and the fin plates at the beam web are with the thickness of 3/8". The column is stiffened at the location of fin plates at the beam flanges and are with the thickness of 5/8". The column is loaded by compressive force 200 kip, the beam by bending moment 800 kip-in and shear force 30 kip.

1.1 Geometry

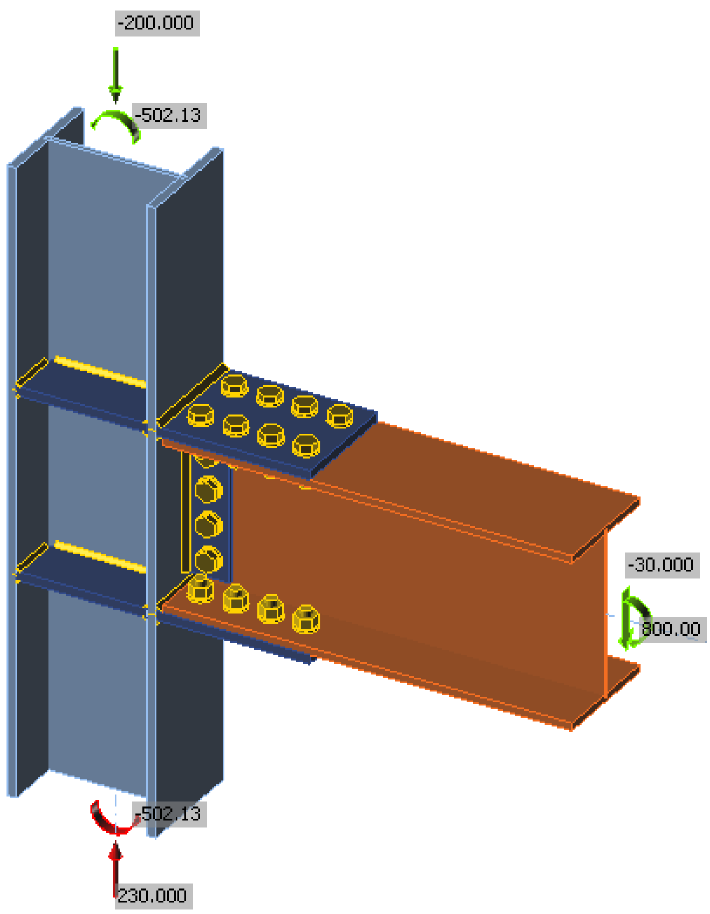

Figure 1: Investigated connection

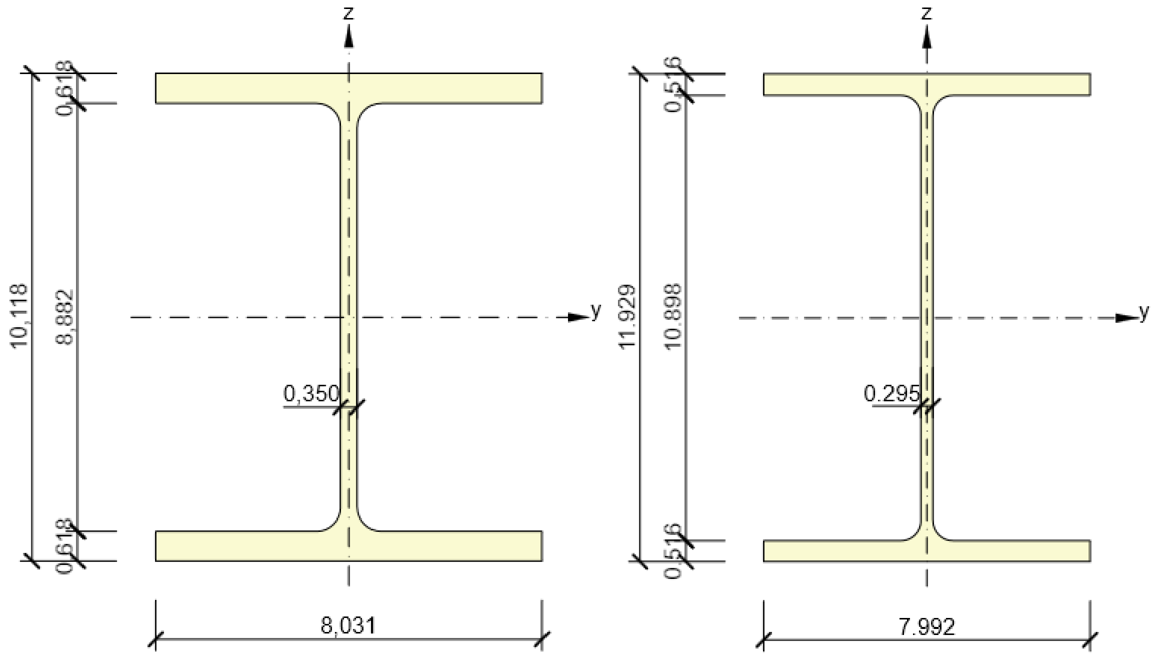

Figure 2: Cross-sections of column (left) and beam (right)

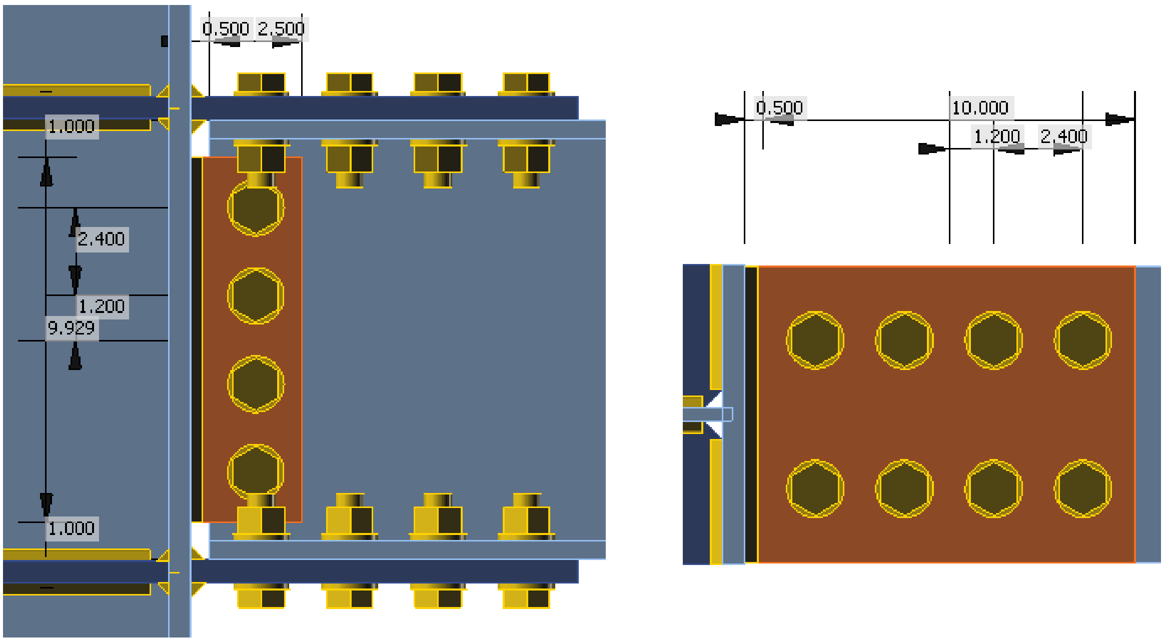

Figure 3: Geometry of fin plates

1.2 Manual assessment

Manual assessment is provided according to AISC 360-16. For simplification, the bending moment is considered to be transferred only by the flanges and the shear force only by the web. The shear force is assumed to be acting at the face of the column. The following checks are required:

- Bolt strength in shear – J3.6
- Bearing and hole tearout strength at bolt holes – J3.10
- Block shear strength – J4.3
- Tensile strength of connected elements – J4.1
- Shear strength of connected elements – J4.2
- Weld strength – J2.4

The design of beam and column is assumed to be checked elsewhere.

1.2.1 Distribution of forces

The bending moment is transferred via bolts on the beam flange. The distance between shear planes is 11.929”. The force acting on the group of bolts at flanges is 67.06 kip.

The bending moment is further transferred via welds connecting fin plates to the column flange. The distance between centers of gravity of welds is increased by the thickness of the fin plate, i.e. $11.929 + 5/8 = 12.554$”. The welds are loaded by force 63.72 kip.

The bolts at the web are loaded by the shear force 30 kip and by a small shear force resulting from the bending moment caused by the eccentricity of assumed shear force acting at the column face, 1.75”. This shear force is neglected here because the utilization of bolts at the beam web is not expected to be very high and there is enough reserve.

The welds at the fin plate connecting the beam web are loaded by shear force 30 kip.

1.2.2 Bolt check

Bolts at the beam flange: The shear force 67.06 kip is assumed to be evenly distributed between 8 bolts 3/4” A307.

Shear strength:

$$\phi R_n = \phi F_{nv} A_b = 0.75 \cdot 27 \cdot 0.442 = 8.938\,\texttt{kip} \tag{1}$$

Bearing strength:

$$\phi R_n = \phi 2.4 d t F_u = 0.75 \cdot 2.4 \cdot 0.75 \cdot 0.516 \cdot 58 = 40.394\,\texttt{kip} \tag{2}$$

Hole tearout strength:

$$\phi R_n = \phi 1.2 l_c t F_u = 0.75 \cdot 1.2 \cdot (1.4 - 0.406) \cdot 0.516 \cdot 58 = 26.77\,\texttt{kip} \tag{3}$$

The shear resistance of one bolt is 8.938 kip, i.e. the resistance of a group of 8 bolts is 67.184 kip. The resistance is sufficient to transfer shear force 67.06 kip.

Block shear strength:

$$\phi R_n = \phi(0.6 F_u A_{nv} + U_{bs} F_u A_{nt}) \leq \phi(0.6 F_y A_{gv} + U_{bs} F_u A_{nt}) \tag{4}$$

$$\phi R_n = 0.75 \cdot (0.6 \cdot 58 \cdot 2.97 + 1 \cdot 58 \cdot 0.82) \leq 0.75 \cdot (0.6 \cdot 36 \cdot 4.44 + 1 \cdot 58 \cdot 0.82) = 143\,\texttt{kip} \quad (5)$$

This example shows the block shear strength of the upper flange of the beam. The expected rupture is presumed to span across 4 bolts next to the beam web. Thus, it must resist half the load acting on the bolt group, i.e. 30.03 kip. The reserve is very high.

Tensile yielding of the fin plate:

$$\phi R_n = \phi F_y A_g = 0.9 \cdot 36 \cdot 5.00 = 162\,\texttt{kip} \quad (6)$$

Tensile rupture of the fin plate:

$$\phi R_n = \phi F_u A_n = 0.75 \cdot 58 \cdot 3.98 = 173\,\texttt{kip} \quad (7)$$

The plate is utilized at 41 %.

Bolts at the beam web: The shear force 30 kip is assumed to be evenly distributed between 4 bolts 3/4" A307.

Shear strength:

$$\phi R_n = \phi F_{nv} A_b = 0.75 \cdot 27 \cdot 0.442 = 8.938\,\texttt{kip} \quad (8)$$

Bearing strength:

$$\phi R_n = \phi 2.4 d t F_u = 0.75 \cdot 2.4 \cdot 0.75 \cdot 0.295 \cdot 58 = 23.1\,\texttt{kip} \quad (9)$$

Hole tearout strength:

$$\phi R_n = \phi 1.2 l_c t F_u = 0.75 \cdot 1.2 \cdot (1.365 - 0,406) \cdot 0.375 \cdot 58 = 17.81\,\texttt{kip} \quad (10)$$

The shear resistance of one bolt is 8.938 kip, i.e. the resistance of a group of 4 bolts is 36 kip. The resistance is sufficient to transfer shear force 30 kip.

Shear yielding of the fin plate:

$$\phi R_n = \phi 0.6 F_y A_{gv} = 1 \cdot 0.6 \cdot 36 \cdot 3.72 = 80\,\texttt{kip} \quad (11)$$

Shear rupture of the fin plate:

$$\phi R_n = \phi 0.6 F_u A_{nv} = 0.75 \cdot 0.6 \cdot 58 \cdot 2.50 = 65\,\texttt{kip} \quad (12)$$

The shear strength of the fin plate, i.e. 65 kip is sufficient to transfer the shear load 30 kip.

1.2.3 Weld check

Welds near the beam flange: Welds connecting the fin plate at the beam flanges to the column flange are required to transfer 63.72 kip. Welds are loaded at an angle 90°. Weld electrode E70XX is used and its size is 3/8".

$$F_{nw} = 0.6 F_{EXX} (1 + 0.5 \sin^{1.5}\theta) = 0.6 \cdot 70 \cdot (1 + 0.5 \sin^{1.5} 90°) = 63\,\texttt{ksi} \quad (13)$$

$$\phi R_n = \phi F_{nw} A_{we} = 0.75 \cdot 63 \cdot 4.213 = 199\,\texttt{kip} \quad (14)$$

The weld strength is sufficient.

Welds near the beam web: Welds connecting the fin plate at the beam web to the column flange are required to transfer 30 kip. Welds are loaded at an angle 0°. Weld electrode E70XX is used and its size is 5/16”.

$$F_{nw} = 0.6F_{EXX}(1 + 0.5\sin^{1.5}\theta) = 0.6 \cdot 70 \cdot (1 + 0.5\sin^{1.5} 0°) = 42\,\texttt{ksi} \tag{15}$$

$$\phi R_n = \phi F_{nw} A_{we} = 0.75 \cdot 42 \cdot 4.374 = 138\,\texttt{kip} \tag{16}$$

The weld strength is sufficient.

1.3 Check in IDEA StatiCa

The plates are checked by finite element analysis. The bilinear material model is used with the yield strength multiplied by steel resistance factor $\phi = 0.9$. The forces acting on other components of the connection, i.e. bolts and welds, are also determined by finite element analysis but their resistance is checked using standard formulas from AISC 360-16. The most stressed weld element is checked and with further loading, the stress in weld is spreading into further weld elements. Therefore, the ultimate weld resistance is higher than simply dividing the force by weld utilization.

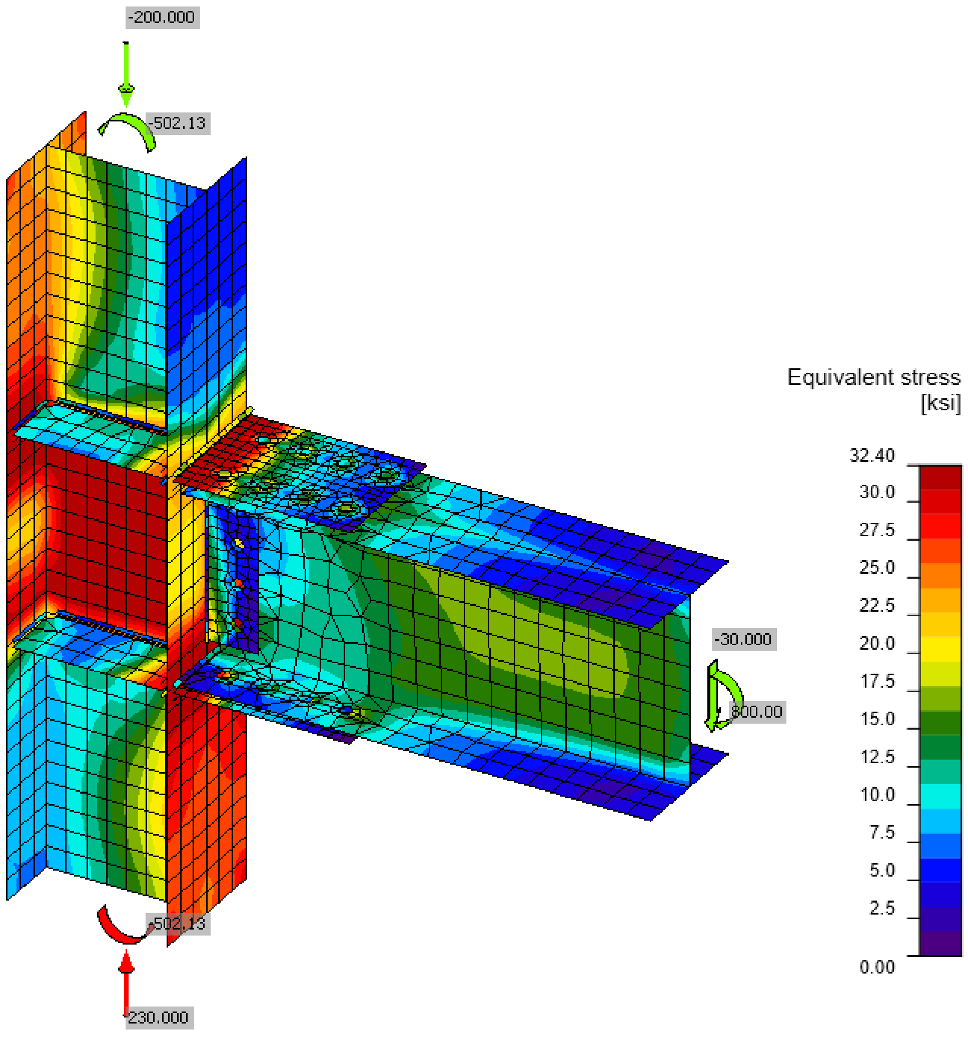

Figure 4: Von Mises stress

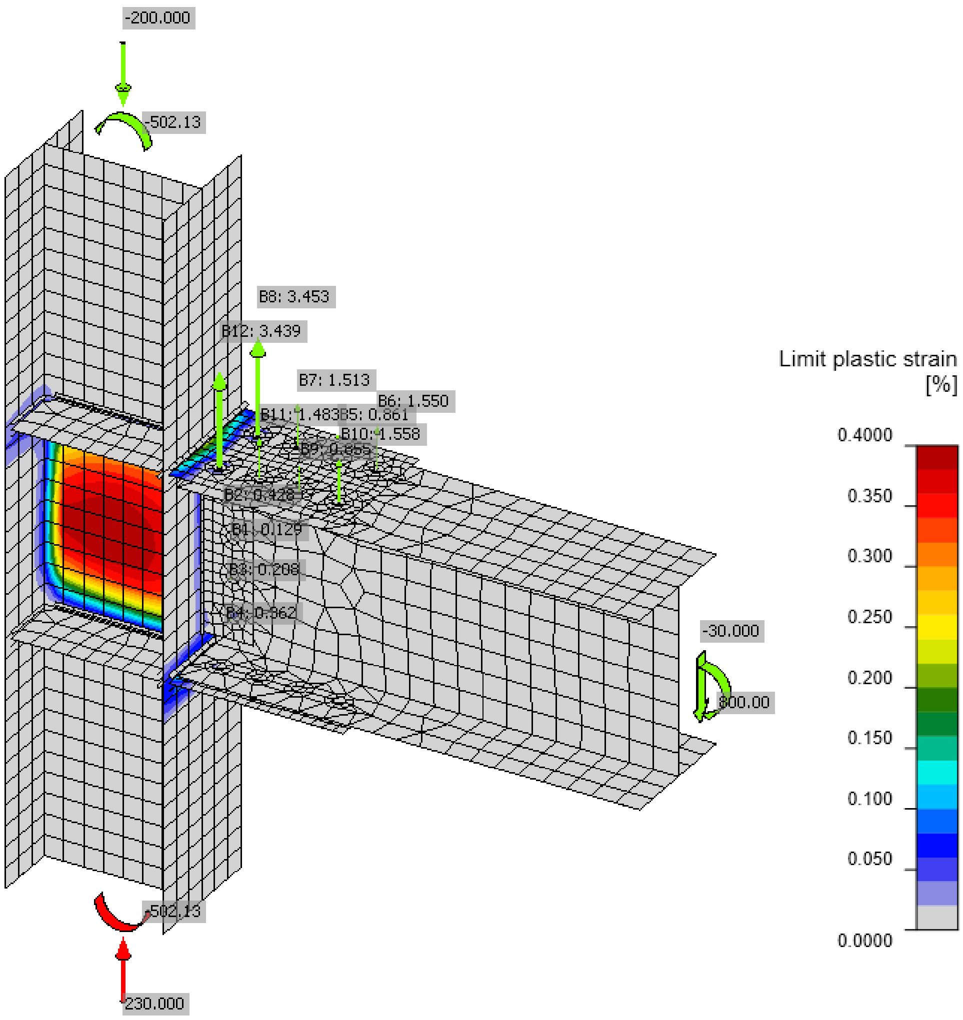

Figure 5: Plastic strain including the tensile forces in bolts

Check of members and steel plates for extreme load effect

	Status	Item	Th [in]	Loads	σ,Ed [ksi]	ε,Pl [%]
>	✓	C-bfl 1	5/8"	LE1	32.5	0.2
	✓	C-tfl 1	5/8"	LE1	32.4	0.1
	✓	C-w 1	3/8"	LE1	32.5	0.4
	✓	B-bfl 1	1/2"	LE1	20.4	0.0
	✓	B-tfl 1	1/2"	LE1	29.0	0.0
	✓	B-w 1	5/16"	LE1	20.4	0.0
	✓	FP1	3/8"	LE1	28.5	0.0
	✓	FP2	5/8"	LE1	32.5	0.2
	✓	FP3	5/8"	LE1	32.5	0.4
	✓	STIFF1a	5/8"	LE1	31.5	0.0
	✓	STIFF1b	5/8"	LE1	31.4	0.0
	✓	STIFF2a	5/8"	LE1	32.4	0.0
	✓	STIFF2b	5/8"	LE1	32.4	0.0

Design data

	Grade	Fy [ksi]	ε,lim [%]
>	A36	36.0	5.0

Figure 6: Check of stress and strain of plates

Check of bolts for extreme load effect

		Status	Item	Loads	Ft [kip]	V [kip]	Bearing φRn [kip]	Utt [%]	Uts [%]	Utts [%]	Detailing
>	+		B1	LE1	0.129	0.803	23.126	0.9	9.0	-	
	+		B2	LE1	0.428	2.606	13.719	2.9	29.1	-	
	+		B3	LE1	0.208	1.501	23.126	1.4	16.8	-	
	+		B4	LE1	0.362	3.120	23.126	2.4	34.9	-	
	+		B5	LE1	0.861	7.812	40.394	5.8	87.4	-	
	+		B6	LE1	1.550	7.869	32.441	10.4	88.0	-	
	+		B7	LE1	1.513	7.830	40.394	10.2	87.6	-	
	+		B8	LE1	3.453	7.662	26.760	23.2	85.7	-	
	+		B9	LE1	0.855	7.810	40.394	5.7	87.4	-	
	+		B10	LE1	1.558	7.867	32.438	10.5	88.0	-	
	+		B11	LE1	1.483	7.825	40.394	10.0	87.5	-	
	+		B12	LE1	3.439	7.651	26.760	23.1	85.6	-	
	+		B13	LE1	0.146	7.506	40.394	1.0	84.0	-	
	+		B14	LE1	0.338	7.574	40.394	2.3	84.7	-	
	+		B15	LE1	0.162	7.606	40.394	1.1	85.1	-	
	+		B16	LE1	0.000	7.739	40.394	0.0	86.6	-	
	+		B17	LE1	0.149	7.506	40.394	1.0	84.0	-	
	+		B18	LE1	0.344	7.576	40.394	2.3	84.8	-	
	+		B19	LE1	0.184	7.600	40.394	1.2	85.0	-	
	+		B20	LE1	0.000	7.724	40.394	0.0	86.4	-	

Design data

	Grade	Tension φRn [kip]	Shear φRn [kip]
>	3/4 A307 - 1	14.897	8.938

Figure 7: Check of bolts

Check of welds for extreme load effect (Plastic redistribution)

		Status	Item	Edge	Xu	Th [in]	Ls [in]	L [in]	Lc [in]	Loads	Fn [kip]	φRn [kip]	Ut [%]	Detailing
>	+	✓	C-bfl 1	FP1	E70xx	◢1/4"◣	◢5/16"◣	9.898	0.619	LE1	4.011	5.741	69.9	✓
	+	✓			E70xx	◢1/4"◣	◢5/16"◣	9.898	0.619	LE1	4.023	5.720	70.3	✓
	+	✓	C-bfl 1	FP2	E70xx	◢1/4"◣	◢3/8"◣	7.945	0.662	LE1	5.774	8.226	70.2	✓
	+	✓			E70xx	◢1/4"◣	◢3/8"◣	7.945	0.662	LE1	1.359	8.187	16.6	✓
	+	✓	C-bfl 1	FP3	E70xx	◢1/4"◣	◢3/8"◣	7.945	0.662	LE1	6.255	8.249	75.8	✓
	+	✓			E70xx	◢1/4"◣	◢3/8"◣	7.945	0.662	LE1	3.033	8.017	37.8	✓
	+	✓	C-bfl 1	STIFF1a	E70xx	◢3/16"◣	◢1/4"◣	3.335	1.112	LE1	6.867	9.130	75.2	✓
	+	✓			E70xx	◢3/16"◣	◢1/4"◣	3.335	1.112	LE1	2.021	8.200	24.6	✓
	+	✓	C-w 1	STIFF1a	E70xx	◢3/16"◣	◢1/4"◣	7.868	1.124	LE1	2.909	7.659	38.0	✓
	+	✓			E70xx	◢3/16"◣	◢1/4"◣	7.868	1.124	LE1	2.561	8.305	30.8	✓
	+	✓	C-tfl 1	STIFF1a	E70xx	◢3/16"◣	◢1/4"◣	3.335	1.112	LE1	2.438	6.736	36.2	✓
	+	✓			E70xx	◢3/16"◣	◢1/4"◣	3.335	1.112	LE1	3.861	8.407	45.9	✓
	+	✓	C-bfl 1	STIFF1b	E70xx	◢3/16"◣	◢1/4"◣	3.335	1.112	LE1	2.014	8.211	24.5	✓
	+	✓			E70xx	◢3/16"◣	◢1/4"◣	3.335	1.112	LE1	6.868	9.130	75.2	✓
	+	✓	C-w 1	STIFF1b	E70xx	◢3/16"◣	◢1/4"◣	7.868	1.124	LE1	2.562	8.304	30.9	✓
	+	✓			E70xx	◢3/16"◣	◢1/4"◣	7.868	1.124	LE1	2.912	7.658	38.0	✓
	+	✓	C-tfl 1	STIFF1b	E70xx	◢3/16"◣	◢1/4"◣	3.335	1.112	LE1	3.863	8.409	45.9	✓
	+	✓			E70xx	◢3/16"◣	◢1/4"◣	3.335	1.112	LE1	2.446	6.732	36.3	✓
	+	✓	C-bfl 1	STIFF2a	E70xx	◢3/16"◣	◢1/4"◣	3.335	1.112	LE1	2.213	7.053	31.4	✓
	+	✓			E70xx	◢3/16"◣	◢1/4"◣	3.329	1.110	LE1	6.803	9.268	73.4	✓
	+	✓	C-w 1	STIFF2a	E70xx	◢3/16"◣	◢1/4"◣	7.868	1.124	LE1	1.945	7.776	25.0	✓
	+	✓			E70xx	◢3/16"◣	◢1/4"◣	7.868	1.124	LE1	2.450	8.821	27.8	✓
	+	✓	C-tfl 1	STIFF2a	E70xx	◢3/16"◣	◢1/4"◣	3.335	1.112	LE1	2.997	9.245	32.4	✓
	+	✓			E70xx	◢3/16"◣	◢1/4"◣	3.335	1.112	LE1	2.369	8.651	27.4	✓
	+	✓	C-bfl 1	STIFF2b	E70xx	◢3/16"◣	◢1/4"◣	3.329	1.110	LE1	6.816	9.268	73.5	✓
	+	✓			E70xx	◢3/16"◣	◢1/4"◣	3.335	1.112	LE1	2.239	6.742	33.2	✓
	+	✓	C-w 1	STIFF2b	E70xx	◢3/16"◣	◢1/4"◣	7.868	1.124	LE1	2.465	8.762	28.1	✓
	+	✓			E70xx	◢3/16"◣	◢1/4"◣	7.868	1.124	LE1	1.939	7.784	24.9	✓
	+	✓	C-tfl 1	STIFF2b	E70xx	◢3/16"◣	◢1/4"◣	3.335	1.112	LE1	2.370	8.635	27.5	✓
	+	✓			E70xx	◢3/16"◣	◢1/4"◣	3.335	1.112	LE1	2.997	9.245	32.4	✓

Figure 8: Check of welds

1.4 Comparison

It is clear that the finite element analysis shows different distribution of internal forces than simple assumptions. Shear force is also partially transferred via fin plates at the beam flanges as can be seen from the tensile forces in bolts and high stresses caused by bending of the fin plate near the column flange. The individual strengths of bolts and welds show perfect match but the loads and load directions are different.

While the manual check is showing that the joint is fully utilized due to shear strength of bolts at beam flanges, IDEA still shows some reserve. The loads can be increased by 10 % to achieve full utilization in IDEA. This can be expected due to the simplification in load distribution in manual assessment.

The check in design software IDEA StatiCa Connection is in close agreement with the manual assessment according to AISC 360.

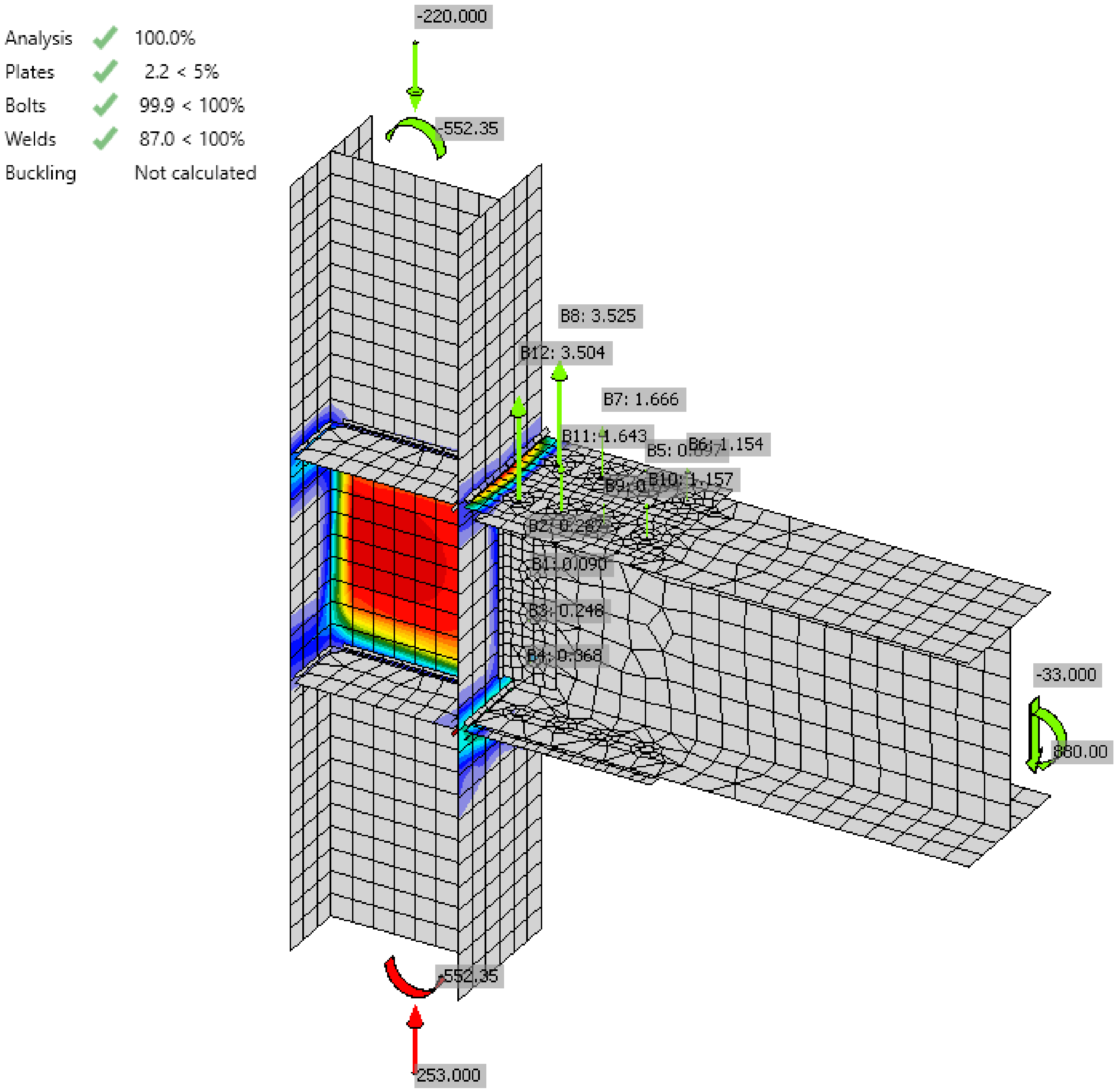

Figure 9: Plastic strain, loads and bolt forces at full utilization

2 Column base plate in braced bay – LRFD

A column with cross-section W12×79 is anchored into a concrete block (concrete compressive strength 4 ksi) by four anchor bolts 3/4" A307 ($f_y = 50$ ksi, $f_u = 65$ ksi). Column base is grouted. A brace is HSS 3.5×0.203 connected by gusset plate and 2 slip-critical bolts 3/4" A490 ($f_y = 130$ ksi, $f_u = 150$ ksi). All steel is grade A36 ($f_y = 36$ ksi, $f_u = 58$ ksi). The shear is transferred via shear lug with cross-section W6×25. Weld electrodes E70XX are selected. The column is loaded by compressive force −160 kip, bending moment 1000 kip-in, and shear force 20 kip. The brace is loaded by tensile force 30 kip.

2.1 Geometry

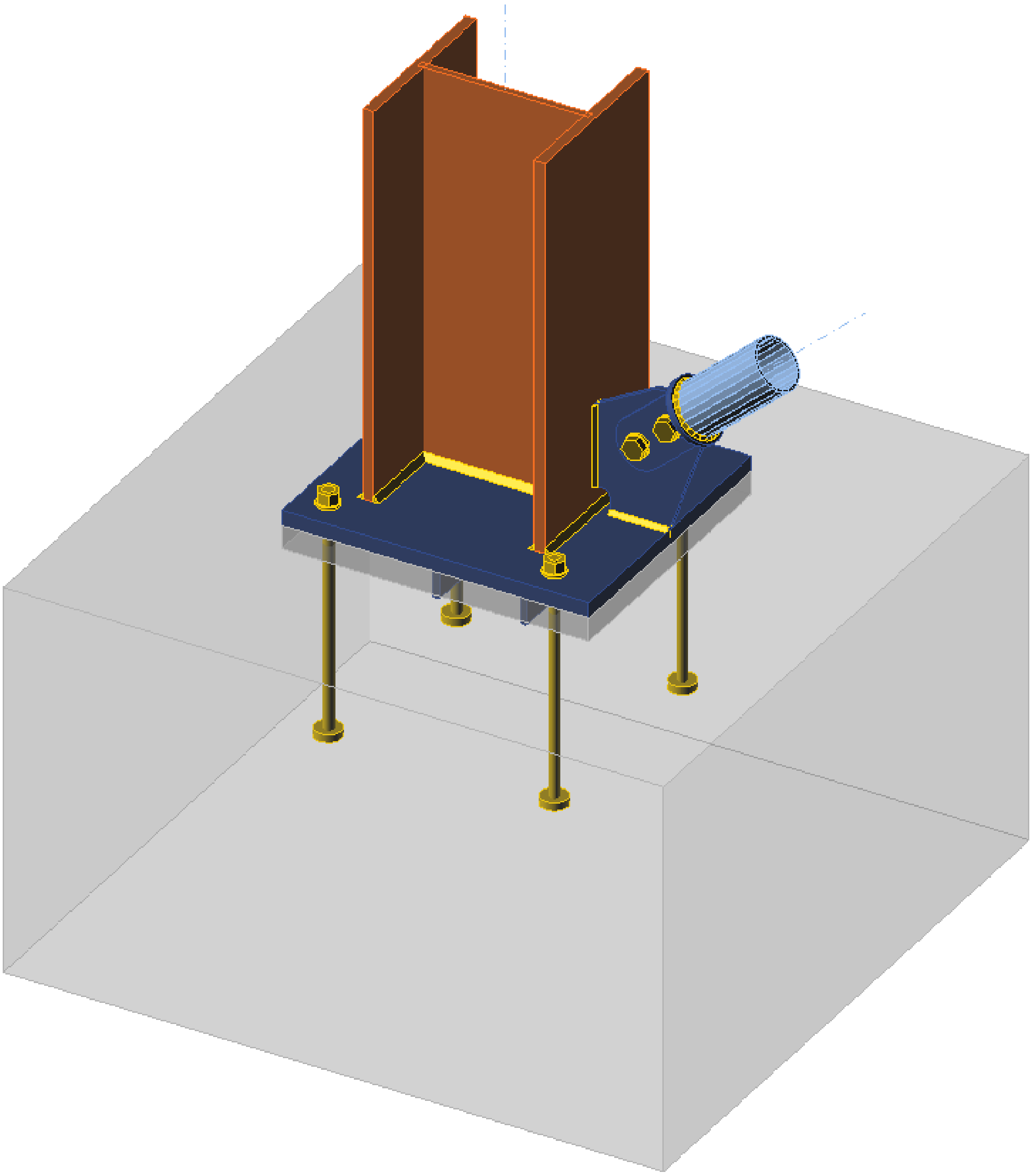

Figure 10: Investigated joint

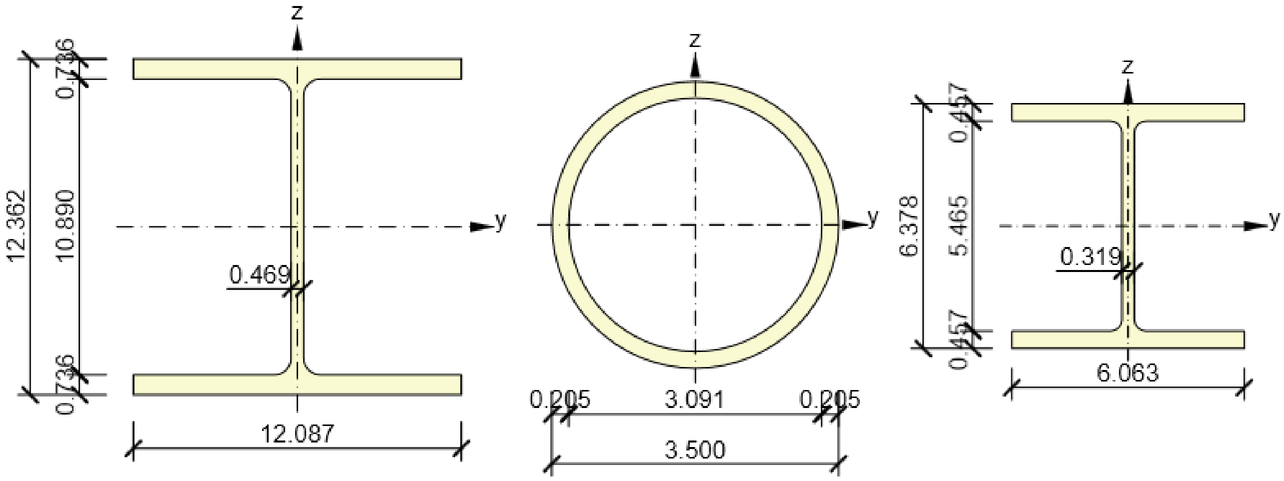

Figure 11: Cross-sections of column (left), brace (middle), and shear lug (right)

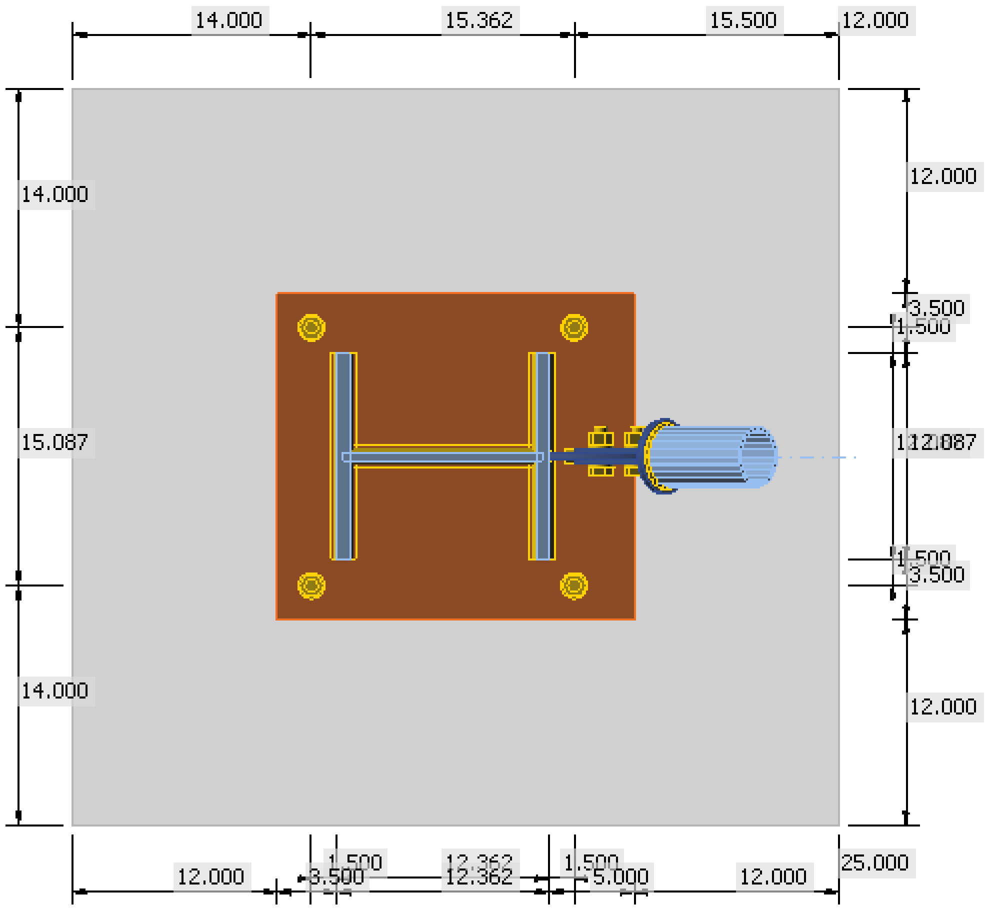

Figure 12: Concrete block dimensions

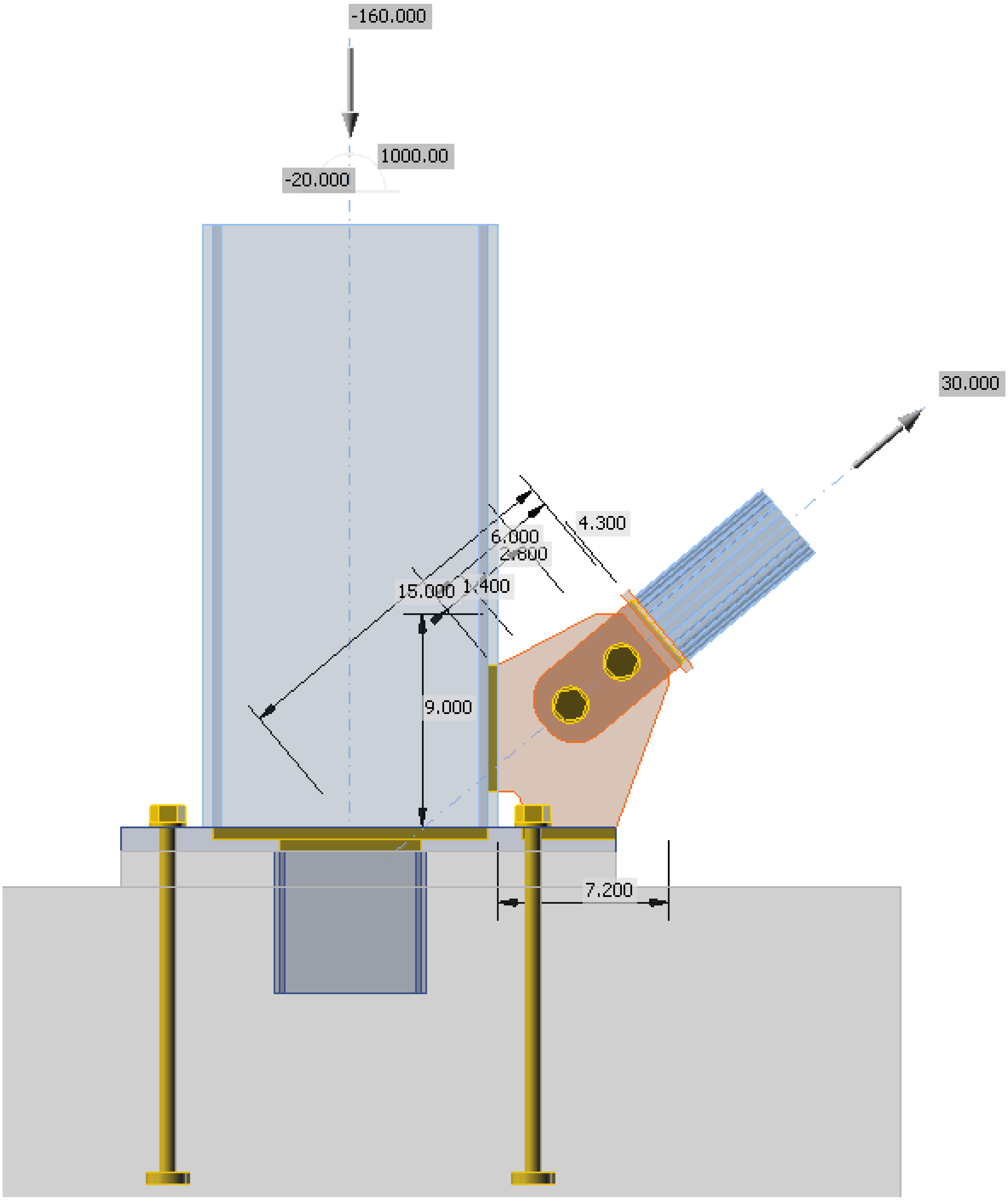

Figure 13: Gusset plate dimensions and loads on a transparent model

2.2 Manual assessment

Manual check of bolts, welds, plates, and concrete in compression is done according to AISC 360-16. The capacity of shear lug is determined according to ACI 349-01. Anchor rods are designed according to AISC 360-16 – J9 and ACI 318-14 – Chapter 17. The following checks are required:

- Slip resistance of bolts in shear – AISC 360-16 – J3.8
- Block shear strength – AISC 360-16 – J4.3
- Tensile strength of connected elements – AISC 360-16 – J4.1
- Weld strength – AISC 360-16 – AISC 360-16 – J2.4
- Shear strength of shear lug – AISC 360-16 – G2
- Bending strength of shear lug – AISC 360-16 – F2.1
- Bearing capacity of shear lug against concrete – ACI 349-01 – B.4.5 and RB11
- Concrete breakout strength of the shear lug – ACI 349 – B11
- Concrete bearing strength in compression – AISC 360-16 – J8
- Steel strength of anchors in tension – ACI 318-14 – 17.4.1
- Concrete breakout strength – ACI 318-14 – 17.4.2
- Concrete pullout strength – ACI 318-14 – 17.4.3
- Concrete side-face blowout strength – ACI 318-14 – 17.4.4

The design of beam and column is assumed to be checked elsewhere.

2.2.1 Distribution of forces

The whole shear force is expected to be transferred via the shear lug into the concrete block. The shear is transferred only in the concrete block and the grout is ineffective. The shear force is the sum of shear force in column and the horizontal component of the tensile force in the brace, i.e. $V = 20 + 30 \cdot \cos(40°) = 43\,\text{kip}$.

The tensile force in the brace, 30 kip, is required to be transferred via two preloaded bolts. The gusset plates and welds needs to be sufficient.

The compressive force, 160 kip, is decreased by the vertical component of the tensile force in the brace. The column base needs to resist compressive force of $160 - 30 \cdot \sin(40°) = 141\,\text{kip}$ and bending moment 1000 kip-in.

2.2.2 Brace connection check

Slip-critical connection The strength of slip-critical connection is determined according to AISC 360-16 – J3.8. The minimum bolt pretension is taken from Table J3.1 as $T_b = 35\,\text{kip}$. The single bolt slip resistance is:

$$\phi R_n = \phi \mu D_u h_f T_b n_s = 1 \cdot 0.3 \cdot 1.13 \cdot 1.0 \cdot 35 \cdot 2 = 24\,\texttt{kip} \tag{17}$$

The slip resistance of 2 bolts, 47 kip, is sufficient to transfer the tensile force 30 kip.

Tensile strength of the tongue The tongue are two plates with the thickness of 1/4" to avoid eccentricity in compressive loading. The dimensions of the tongue can be seen in Figure 14. The gross and net areas in tension are $3.4 \cdot (2 \cdot 1/4) = 1.7\,\text{in}^2$ and $(3.4 - 13/16) \cdot (2 \cdot 1/4) = 1.3\,\text{in}^2$, respectively.

$$\phi R_n = \phi F_y A_g = 0.9 \cdot 36 \cdot 1.7 = 55\,\texttt{kip} \tag{18}$$
$$\phi R_n = \phi F_u A_n = 0.75 \cdot 58 \cdot 1.3 = 57\,\texttt{kip} \tag{19}$$

The strength of the tongue, 55 kip, is sufficient to transfer tensile force, 30 kip. The welds are designed as CJP butt welds and their strength should be the same as the base material.

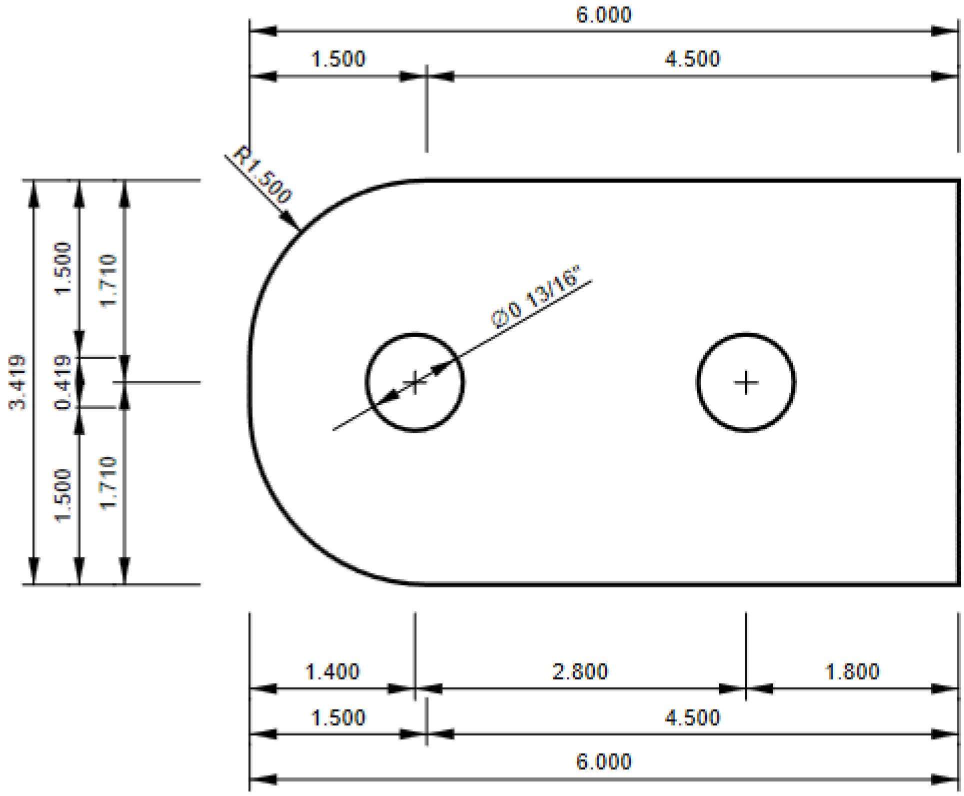

Figure 14: Tongue dimensions

Gusset plate block shear strength The expected yield line at gusset plate for block shear failure is 6.6 in long, the rupture may occur at line shorter by the bolt hole, i.e. 5.8 in. The gusset plate thickness is 3/8".

$$\phi R_n = \phi F_y A_g = 0.9 \cdot 36 \cdot 2.5 = 80\,\texttt{kip} \tag{20}$$
$$\phi R_n = \phi F_u A_n = 0.75 \cdot 58 \cdot 2.2 = 94\,\texttt{kip} \tag{21}$$

The strength of the gusset, 80 kip, is sufficient to transfer tensile force, 30 kip.

Gusset plate weld strength The fillet welds are designed on both sides of the gusset plate with the size 1/4". The lengths of the welds are 5.2 in and 4.0 in. To avoid calculating the eccentricity, it is conservatively assumed that both welds are 4 in long and both welds transfer half of the load. The critical weld is the one loaded at an angle 40°.

$$F_{nw} = 0.6 F_{EXX}(1 + 0.5 \sin^{1.5}\theta) = 0.6 \cdot 70 \cdot (1 + 0.5 \sin^{1.5} 40°) = 53\,\texttt{ksi} \tag{22}$$
$$\phi R_n = \phi F_{nw} A_{we} = 0.75 \cdot 53 \cdot 2.83 = 112\,\texttt{kip} \tag{23}$$

The strength of the welds at the gusset, 224 kip, is sufficient to transfer tensile force, 30 kip.

2.2.3 Column base check

The column base needs to resist compressive force of $P_u = 160 - 30 \cdot \sin(40°) = 141\,\text{kip}$ and bending moment M_u =1000 kip-in. Since the supporting area, A_2, is sufficiently large, the concrete bearing strength is

$$\phi f_{p,(\max)} = \phi 1.7 f_c' = 0.65 \cdot 1.7 \cdot 4 = 4.4\,\texttt{ksi} \tag{24}$$
$$\phi q_{\max} = f_{p,(\max)} B = 4.4 \cdot 19 = 83.6\,\texttt{kip/in} \tag{25}$$

The base plate is elongated due to the gusset connection of the brace. It is conservatively assumed that the compressive force is acting at the column flange, i.e. e =6.18 in from the connection center. The distance between anchor bolt and connection center is f =7.68 in.

$$M_u = eP_r + 2fN_{ua} \tag{26}$$
$$N_{ua} = \frac{M_u - eP_r}{2f} = \frac{1000 - 6.18 \cdot 141}{2 \cdot 7.68} = 8.4\,\texttt{kip} \tag{27}$$
$$Y = \frac{P_r + 2N_{ua}}{q_{\max}} = \frac{141 + 2 \cdot 8.4}{83.6} = 1.9\,\texttt{in} \tag{28}$$

The bearing resistance of the concrete is sufficient, because the base plate is large enough to accommodate bearing area length, Y, and the tensile force in anchor is 8.4 kip. More detailed base plate check with the check of base plate yielding should be provided for the load case with maximum compressive force.

Anchor design Anchors are 3/4”, grade A307, 12 in embedded length in the concrete block with circular washer plates with diameter 1.8 in. Anchors are loaded only in tension because shear is transferred via shear lug. The check of anchors is provided according to ACI 318-14 – Chapter 17. Steel strength and pullout strength is provided for individual anchors and concrete breakout strength and concrete side-face blowout strength is provided for group of anchors because $3h_{ef} \geq s$, where h_{ef} is the embedment depth and s is anchor spacing.

Steel strength in tension of an anchor – 17.4.1

$$\phi N_{sa} = \phi A_{se,N} f_{uta} \tag{29}$$
$$\phi N_{sa} = 0.7 \cdot 0.334 \cdot 60 = 14\,\texttt{kip} \tag{30}$$

Concrete breakout strength – 17.4.2

$$h_{ef} = \min\left(\frac{c_{a,\max}}{1.5}, \frac{s}{3}\right) \leq h_{ef} = \max\left(\frac{14}{1.5}, \frac{15.1}{3}\right) = 9.33 \leq 12\,\texttt{in} \tag{31}$$
$$A_{Nc} = (14 + 1.8/2 + 14) \cdot (14 + 15.1 + 14) = 1245\,\texttt{in}^2 \tag{32}$$
$$A_{Nco} = 9h_{ef}^2 = 9 \cdot 9.33^2 = 783\,\texttt{in}^2 \tag{33}$$
$$N_b = k_c \lambda_a \sqrt{f_c'} h_{ef}^{1.5} = 24 \cdot 1 \cdot \sqrt{4000} \cdot 9.33^{1.5} = 43.3\,\texttt{kip} \tag{34}$$
$$\psi_{ec,N} = \frac{1}{1 + \frac{2e_N'}{3h_{ef}}} = \frac{1}{1 + \frac{2 \cdot 0}{3 \cdot 9.33}} = 1 \tag{35}$$
$$\psi_{ed,N} = \min\left(0.7 + \frac{0.3c_{a,min}}{1.5h_{ef}}, 1\right) = \min\left(0.7 + \frac{0.3 \cdot 14}{1.5 \cdot 9.33}, 1\right) = 1 \tag{36}$$
$$\phi N_{cbg} = \phi \frac{A_{Nc}}{A_{Nco}} \psi_{ec,N} \psi_{ed,N} \psi_{c,N} \psi_{cp,N} N_b \tag{37}$$
$$\phi N_{cbg} = 0.7 \cdot \frac{1245}{783} \cdot 1 \cdot 1 \cdot 1 \cdot 1 \cdot 43.3 = 48\,\texttt{kip} \tag{38}$$

Concrete pullout strength – 17.4.3

$$A_{brg} = \pi\left(\frac{d_{wp}^2 - d_a^2}{4}\right) = \pi\left(\frac{1.8^2 - 0.75^2}{4}\right) = 2.1\,\texttt{in}^2 \tag{39}$$

$$N_p = 8A_{brg}f_c' = 8 \cdot 2.1 \cdot 4 = 67\,\texttt{kip} \tag{40}$$

$$\phi N_{pn} = \phi\psi_{c,P}N_p = 0.7 \cdot 1 \cdot 67 = 47\,\texttt{kip} \tag{41}$$

Concrete side-face blowout strength – 17.4.4

$$red = \frac{1 + \frac{c_{a2}}{c_{a1}}}{4} = \frac{1 + \frac{14}{14}}{4} = 0.5 \tag{42}$$

$$\phi N_{sb} = \phi 160 c_{a1}\sqrt{A_{brg}}\sqrt{f_c'} = 0.7 \cdot 160 \cdot 14 \cdot \sqrt{2.1} \cdot \sqrt{4000} = 144\,\texttt{kip} \tag{43}$$

$$\phi N_{sbg} = n \cdot red \cdot \phi N_{sb} = 2 \cdot 0.5 \cdot 144 = 144\,\texttt{kip} \tag{44}$$

The smallest resistance is that of the anchor steel, 14 kip. It is sufficient to transfer the load 8.4 kip.

Shear lug design The whole shear force is expected to be transferred via the shear lug into the concrete block. The shear is transferred only in the concrete block and the grout is ineffective. The shear force is the sum of shear force in column and the horizontal component of the tensile force in the brace, i.e. $V = 20 + 30 \cdot \cos(40°) = 43$ kip. The shear lug cross-section is W6x25 and it is 6 in long. The grout layer is 1.5 in thick, so the shear lug is embedded 4.5 in in concrete block. The concrete pressure is assumed as uniform in the concrete block. The bending moment acting on shear lug is equal to shear force acting on lever arm $1.5 + 4.5/2 = 3.75$ in, i.e. $M_u = 161$ kip-in. It is expected that fillet welds on shear lug flanges and web are transferring bending moment and shear, respectively. The fillet welds at the flanges needs to transfer $161/5.9 = 27.3$ kip.

Bearing capacity of shear lug against concrete – ACI 349-01 – B4.5 and RB11

$$N_y = nA_{se}F_y = 4 \cdot 0.334 \cdot 36 = 48\,\texttt{kip} \tag{45}$$

$$\phi P_{br} = \phi 1.3 f_c' A_1 + \phi K_c(N_y - P_a) \tag{46}$$

$$\phi P_{br} = 0.7 \cdot 1.3 \cdot 4 \cdot 27.3 + 0.7 \cdot 1.6 \cdot (48 + 141) = 311\,\texttt{kip} \geq 43\,\texttt{kip} \tag{47}$$

Concrete breakout strength of the shear lug – ACI 349-01 – B11

$$A_{Vc} = (18.5 + 6.1 + 18.5) \cdot (4.5 + 20) - 6.1 \cdot 4.5 = 1028\,\texttt{in}^2 \tag{48}$$

$$\phi V_{cb} = A_{Vc}4\phi\sqrt{f_c'} = 1028 \cdot 4 \cdot 0.85 \cdot \sqrt{4000} = 221\,\texttt{kip} \geq 43\,\texttt{kip} \tag{49}$$

Shear strength of shear lug – AISC 360-16 – G2

$$\phi V_n = 0.6F_yA_wC_{v1} = 1 \cdot 0.6 \cdot 36 \cdot 2 \cdot 1 = 44\,\texttt{kip} \geq 43\,\texttt{kip} \tag{50}$$

Fillet welds of shear lug web – AISC 360-16 – J2.4

$$F_{nw} = 0.6F_{EXX}(1 + 0.5\sin^{1.5}\theta) = 0.6 \cdot 70 \cdot (1 + 0.5\sin^{1.5}0°) = 42\,\texttt{ksi} \tag{51}$$

$$\phi R_n = \phi F_{nw}A_{we} = 0.75 \cdot 42 \cdot 1.93 = 61\,\texttt{kip} \geq 43\,\texttt{kip} \tag{52}$$

Bending strength of shear lug – AISC 360-16 – F2.1

$$\phi M_n = \phi M_p = F_yZ_x = 0.9 \cdot 36 \cdot 18.9 = 680.4\,\texttt{kip-in} \geq 161\,\texttt{kip-in} \tag{53}$$

Fillet welds of the shear lug flange – AISC 360-16 – J2.4

$$F_{nw} = 0.6F_{EXX}(1 + 0.5\sin^{1.5}\theta) = 0.6 \cdot 70 \cdot (1 + 0.5\sin^{1.5} 90°) = 63\,\mathtt{ksi} \tag{54}$$

$$\phi R_n = \phi F_{nw} A_{we} = 0.75 \cdot 63 \cdot 2.1 = 100\,\mathtt{kip} \geq 27.3\,\mathtt{kip} \tag{55}$$

The shear and bending strength of the shear lug, weld strength, concrete bearing strength and concrete breakout strength are enough to transfer shear force 43 kip.

2.3 Check in IDEA StatiCa

The plates are checked by finite element analysis. The bilinear material model is used with the yield strength multiplied by steel resistance factor $\phi = 0.9$. The forces acting on other components of the connection, i.e. bolts and welds, are also determined by finite element analysis but their resistance is checked using standard formulas from AISC 360-16, ACI 318-14, and ACI 349-01. The most stressed weld element is checked and with further loading, the stress in weld is spreading into further weld elements. Therefore, the ultimate weld resistance is higher than simply dividing the force by weld utilization.

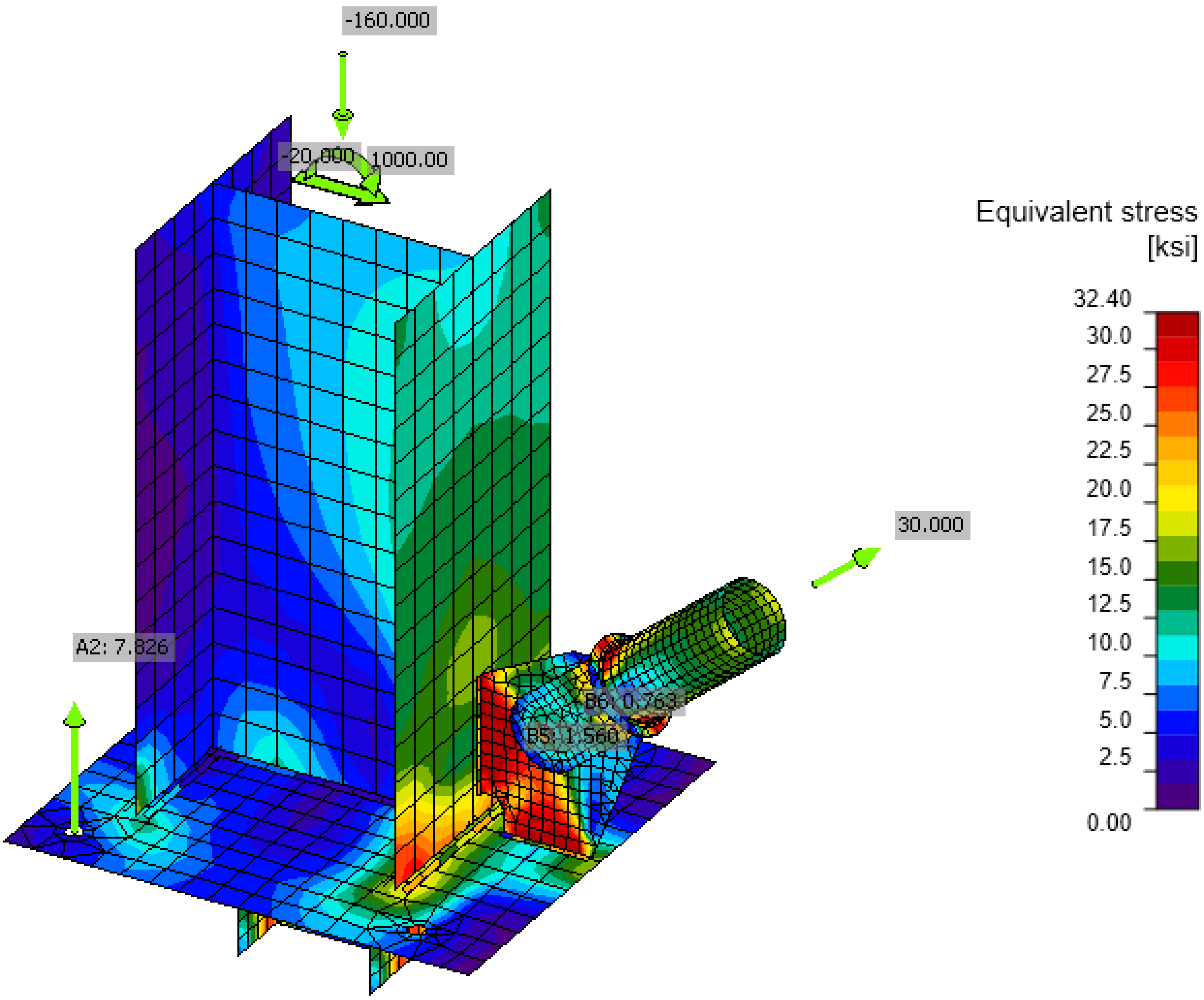

Figure 15: Von Mises stress

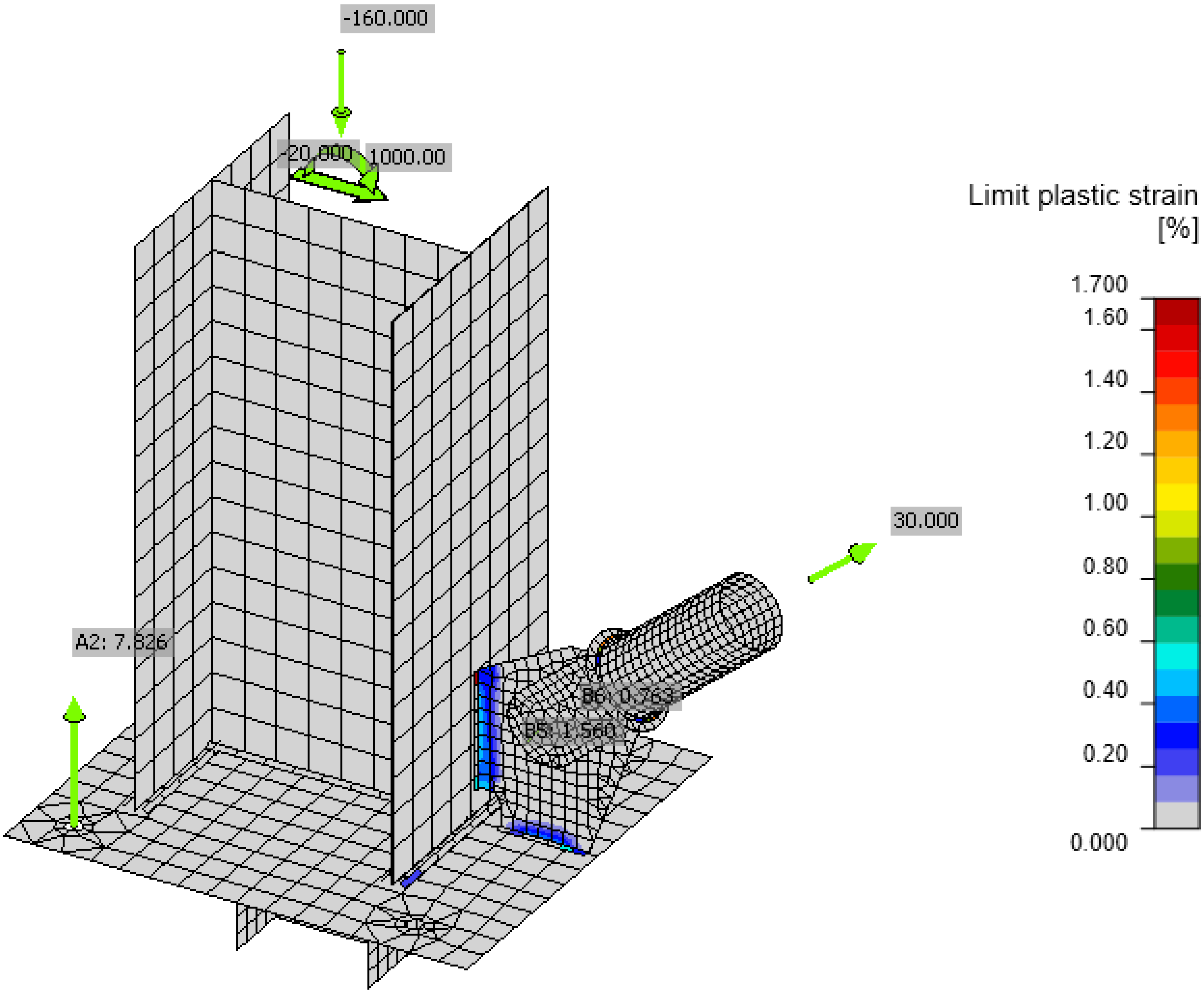

Figure 16: Plastic strain including the tensile forces in anchors

Check of members and steel plates for extreme load effect

	Status	Item	Th [in]	Loads	σ,Ed [ksi]	ε,Pl [%]
>	✓	W12x79-bfl 1	3/4"	LE1	27.2	0.0
	✓	W12x79-tfl 1	3/4"	LE1	14.0	0.0
	✓	W12x79-w 1	7/16"	LE1	23.3	0.0
	✓	HSS3.5x0.203	3/16"	LE1	32.4	0.0
	✓	BP1	1"	LE1	22.4	0.0
	✓	Member 3-bfl 1	7/16"	LE1	32.4	0.1
	✓	Member 3-tfl 1	7/16"	LE1	32.4	0.1
	✓	Member 3-w 1	5/16"	LE1	30.1	0.0
	✓	CPL1a	3/8"	LE1	29.5	0.7
	✓	CPL1b	3/8"	LE1	32.4	0.0
	✓	CPL1c	1/4"	LE1	32.4	0.2
	✓	CPL1d	1/4"	LE1	32.5	0.2

Design data

	Grade	Fy [ksi]	ε,lim [%]
>	A36	36.0	5.0

Figure 17: Check of stress and strain of plates

Check of preloaded bolts for extreme load effect

		Status	Item	Loads	Ft [kip]	V [kip]	Slip φRn [kip]	Utt [%]	Uts [%]	Detailing
>	+	✓	B5	LE1	1.560	7.288/7.287	11.420	5.6	63.8	✓
	+	✓	B6	LE1	0.763	7.712/7.712	11.660	2.7	66.1	✓

Design data

	Grade	Tension φRn [kip]	μ
>	3/4 A490 - 1	37.484	0.3

Figure 18: Check of slip-critical connection

Check of welds for extreme load effect (Plastic redistribution)

		Status	Item	Edge	Xu	Th [in]	Ls [in]	L [in]	Lc [in]	Loads	Fn [kip]	φRn [kip]	Ut [%]	Detailing
>	+	✓	BP1	W12x79-bfl 1	E70xx	◢1/4"◣	◢3/8"◣	12.069	1.341	LE1	11.607	15.390	75.4	✓
	+	✓			E70xx	◢1/4"◣	◢3/8"◣	12.051	1.339	LE1	12.619	16.777	75.2	✓
	+	✓	BP1	W12x79-tfl 1	E70xx	◢1/4"◣	◢3/8"◣	12.069	1.341	LE1	8.901	14.761	60.3	✓
	+	✓			E70xx	◢1/4"◣	◢3/8"◣	12.069	1.341	LE1	5.478	15.377	35.6	✓
	+	✓	BP1	W12x79-w 1	E70xx	◢1/4"◣	◢3/8"◣	11.591	1.288	LE1	3.911	15.740	24.8	✓
	+	✓			E70xx	◢1/4"◣	◢3/8"◣	11.591	1.288	LE1	3.915	15.741	24.9	✓
	+	✓	BP1	Member 3-bfl 1	E70xx	◢3/16"◣	◢1/4"◣	6.047	0.756	LE1	1.577	6.249	25.2	✓
	+	✓			E70xx	◢3/16"◣	◢1/4"◣	6.031	0.754	LE1	3.548	6.205	57.2	✓
	+	✓	BP1	Member 3-tfl 1	E70xx	◢3/16"◣	◢1/4"◣	6.047	0.756	LE1	3.896	6.221	62.6	✓
	+	✓			E70xx	◢3/16"◣	◢1/4"◣	6.031	0.754	LE1	1.305	6.208	21.0	✓
	+	✓	BP1	Member 3-w 1	E70xx	◢3/16"◣	◢1/4"◣	5.907	0.844	LE1	3.026	5.619	53.8	✓
	+	✓			E70xx	◢3/16"◣	◢1/4"◣	5.907	0.844	LE1	3.026	5.619	53.8	✓
	+	✓	W12x79-bfl 1	CPL1a	E70xx	◢3/16"◣	◢1/4"◣	5.282	0.587	LE1	3.628	4.340	83.6	✓
	+	✓			E70xx	◢3/16"◣	◢1/4"◣	5.282	0.587	LE1	3.628	4.340	83.6	✓
	+	✓	CPL1b	HSS3.5x0.203	E70xx	◢1/8"	◢3/16"	10.323	0.430	LE1	2.197	2.694	81.6	✓
	+	✓	BP1	CPL1a	E70xx	◢3/16"◣	◢1/4"◣	3.986	0.569	LE1	3.600	4.711	76.4	✓
	+	✓			E70xx	◢3/16"◣	◢1/4"◣	3.986	0.569	LE1	3.600	4.711	76.4	✓

Figure 19: Check of welds

Check of anchors for extreme load effect

		Status	Item	Loads	Nf [kip]	V [kip]	φNcbg [kip]	φNp [kip]	φNsb [kip]	φVcbg [kip]	φVcp [kip]	Utt [%]	Uts [%]	Utts [%]	Detailing
>	+	✓	A1	LE1	7.826	0.053	47.949	47.105	70.438	-	136.364	55.7	0.8	37.8	✓
	+	✓	A2	LE1	7.826	0.053	47.949	47.105	70.438	36.644	136.364	55.8	0.8	37.8	✓
	+	✓	A3	LE1	0.000	0.055	-	47.105	-	18.200	136.364	0.0	1.2	0.1	✓
	+	✓	A4	LE1	0.000	0.055	-	47.105	-	18.200	136.364	0.0	1.2	0.1	✓

Design data

	Grade	φNsa [kip]	φVsa [kip]
>	3/4 A307 - 1	14.038	6.257

Figure 20: Check of anchors

Check of contact stress in concrete for extreme load effect

		Status	Item	Loads	A1 [in2]	A2 [in2]	σ [ksi]	Ut [%]
>	+	✓	CB 1	LE1	182.758	1932.961	0.9	19.6

Figure 21: Check of concrete in bearing

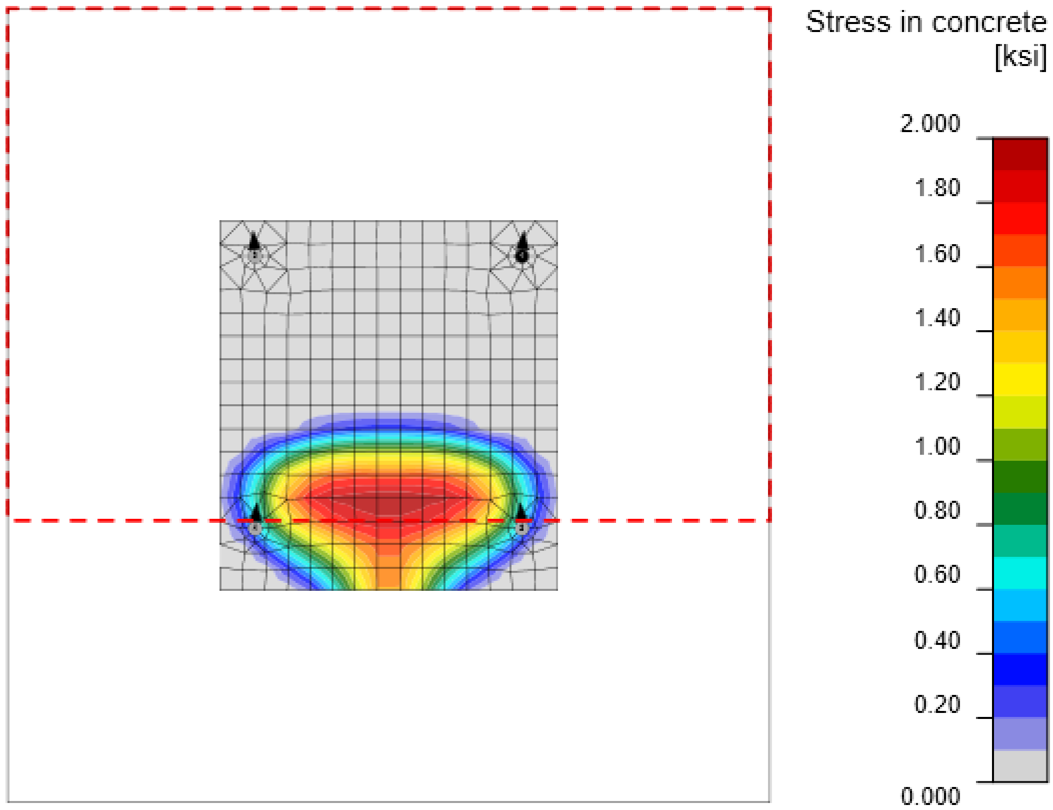

Figure 22: Stress in concrete under the base plate and area of concrete cone breakout

Shear in contact plane for extreme load effect

		Status	Item	Loads	V [kip]	φVr [kip]	φPbr [kip]	φVcb [kip]	Ut [%]
>	+		BP1	LE1	42.833	-	310.357	220.989	19.4

Figure 23: Check of shear lug – bearing capacity and concrete breakout strength

2.4 Comparison

It is clear that the finite element analysis shows different distribution of internal forces than simple assumptions. The gusset plate also helps transferring the bending moment and thus gusset plate and its welds are much more loaded than in standard design assumptions. The forces in anchors are slightly lower in IDEA because the stress below base plate is not exactly under the column flange. The most heavily utilized element in manual assessment is the web of the shear lug. In IDEA StatiCa, the equivalent stress on the shear lug web is at 30.1 kip which is close to yielding.

The check in design software IDEA StatiCa Connection is in agreement with the manual assessment according to AISC 360, ACI 318, and ACI 341. The small differences are caused mainly by simplifications in hand calculations.

3 Extended moment end-plate connection – ASD

Two beams with cross-section W10×26 are connected to each other by extended four-bolt stiffened moment end-plate connection. The end plates have the thickness of 1/2" and are connected by 3 bolt rows. All steel is grade A572 Gr. 50 ($f_y = 50\,\text{ksi}$, $f_u = 65\,\text{ksi}$) and bolts are grade 3/4" grade A325 ($f_{yb} = 92\,\text{ksi}$, $f_{ub} = 119.7\,\text{ksi}$). The connection is loaded by maximum bending moment determined from manual assessment using Design guide 16 and AISC 360-16.

3.1 Geometry

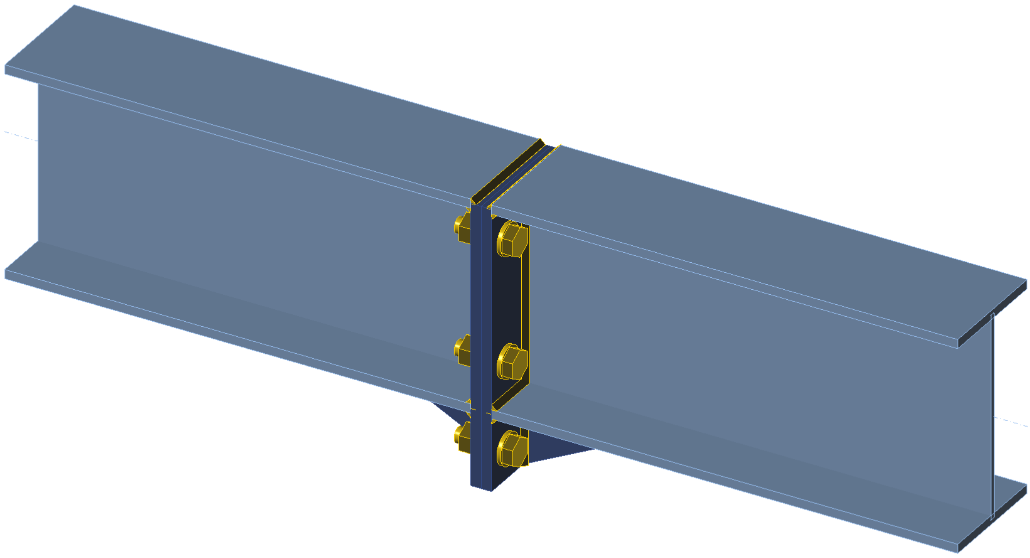

Figure 24: Investigated connection

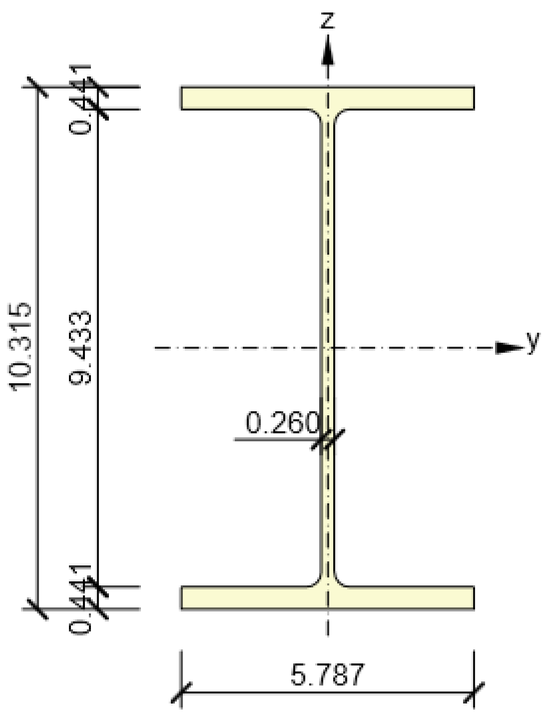

Figure 25: Beam cross-section

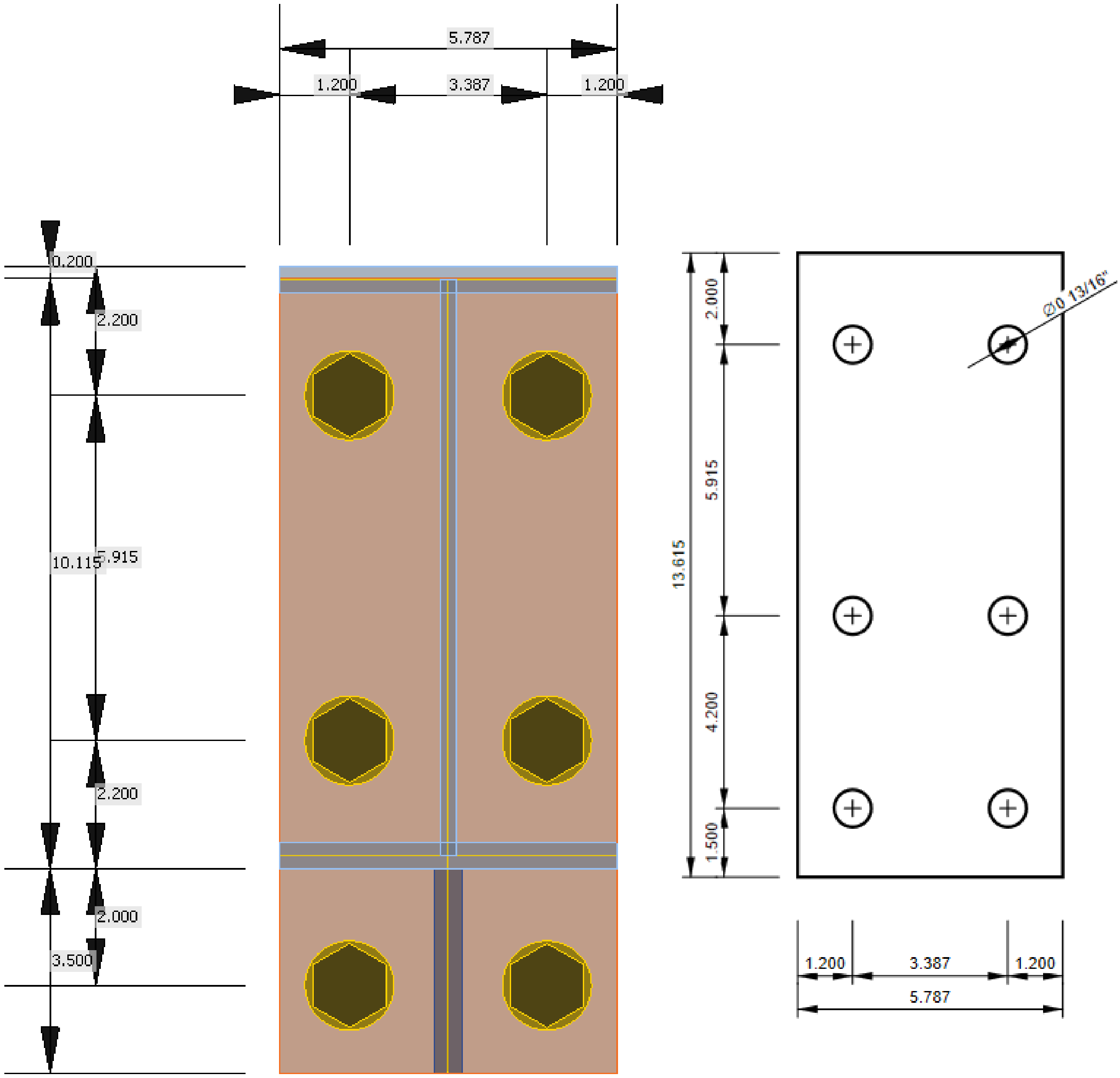

Figure 26: Dimensions of end-plate connection

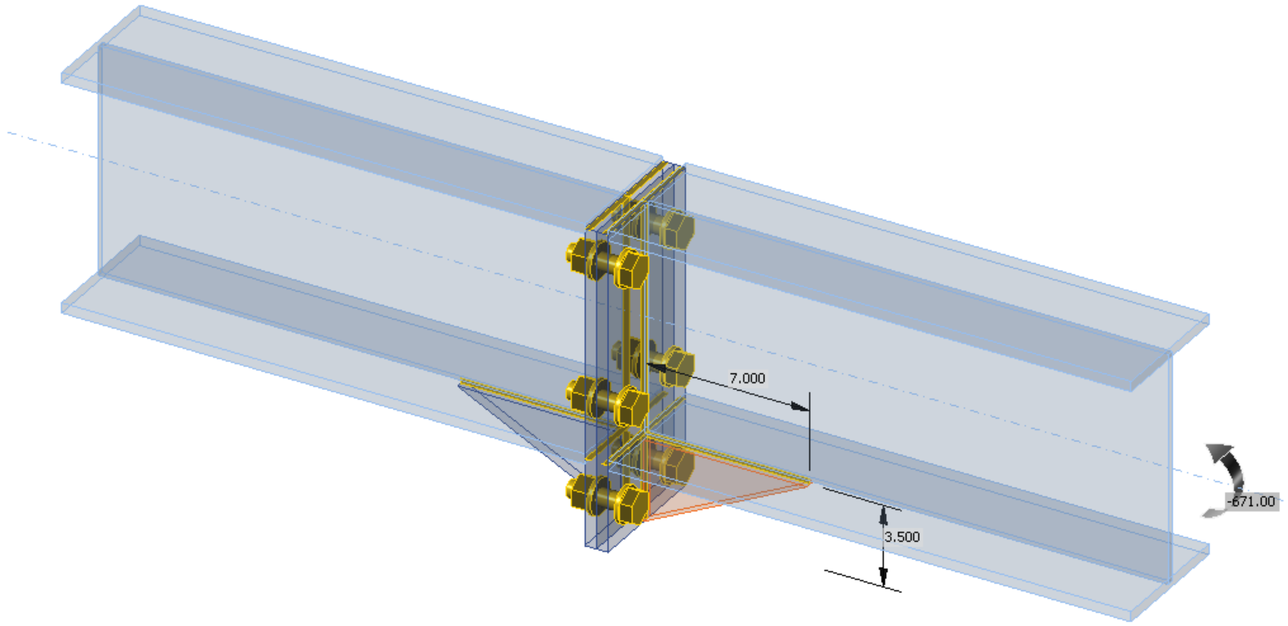

Figure 27: Transparent model with dimensions of widener and applied load

3.2 Manual assessment

Manual assessment is performed according to Design guide 16: Flush and Extended Multiple-Row Moment End-Plate Connections – Chapter 4: Extended End-Plate Design and AISC 360-16 – Chapter J. The following checks are required:

- Bolt strength in tension – AISC 360-16 – J3.6
- End-plate yielding – Design guide 16
- Weld strength – AISC 360-16 – J2.4

The design of beams is assumed to be checked elsewhere.

3.2.1 Bolt and end-plate yielding strength

Bolt tensile strength

$$A_b = \frac{\pi d_b^2}{4} = \frac{\pi \cdot 0.75^2}{4} = 0.442\,\texttt{in}^2 \tag{56}$$

$$P_t = R_n = F_n A_b = 90 \cdot 0.442 = 39.8\,\texttt{kip} \tag{57}$$

Snug-tightened bolt pretension:

$$T_b = 0.5 \cdot 28 = 14\,\texttt{kip} \tag{58}$$

Prying forces The prying forces are determined according to Design guide 16 – Table 4-1:

Inside bolt row:

$$a_i = 3.682\left(\frac{t_p}{d_b}\right)^3 - 0.085 = 3.682\left(\frac{0.5}{0.75}\right)^3 - 0.085 = 1.006 \tag{59}$$

$$w' = b_p/2 - (d_b + 1/16) = 5.787/2 - (0.75 + 1/16) = 2.081\,\texttt{in} \tag{60}$$

$$F_i' = \frac{t_p^2 F_{py}\left(0.85\frac{b_p}{2} + 0.80w'\right) + \frac{\pi d_b^3 F_t}{8}}{4p_{f,i}} \tag{61}$$

$$F_i' = \frac{0.5^2 \cdot 50\left(0.85 \cdot \frac{5.787}{2} + 0.80 \cdot 2.081\right) + \frac{\pi \cdot 0.75^3 \cdot 90}{8}}{4 \cdot 1.759} = 9.446 \tag{62}$$

$$Q_{max,i} = \frac{w' t_p^2}{4a_i}\sqrt{F_{py}^2 - 3\left(\frac{F_i'}{w' t_p}\right)^2} \tag{63}$$

$$Q_{max,i} = \frac{2.081 \cdot 0.5^2}{4 \cdot 1.006}\sqrt{50^2 - 3 \cdot \left(\frac{9.446}{2.081 \cdot 0.5}\right)^2} = 6.137\,\texttt{kip} \tag{64}$$

Outside bolt row:

$$a_o = 3.682\left(\frac{t_p}{d_b}\right)^3 - 0.085 = 3.682\left(\frac{0.5}{0.75}\right)^3 - 0.085 = 1.006 \tag{65}$$

$$w' = b_p/2 - (d_b + 1/16) = 5.787/2 - (0.75 + 1/16) = 2.081\,\texttt{in} \tag{66}$$

$$F_o' = \frac{t_p^2 F_{py}\left(0.85\frac{b_p}{2} + 0.80w'\right) + \frac{\pi d_b^3 F_t}{8}}{4p_{f,o}} \tag{67}$$

$$F_o' = \frac{0.5^2 \cdot 50\left(0.85 \cdot \frac{5.787}{2} + 0.80 \cdot 2.081\right) + \frac{\pi \cdot 0.75^3 \cdot 90}{8}}{4 \cdot 2} = 8.308 \tag{68}$$

$$Q_{max,i} = \frac{w' t_p^2}{4a_o}\sqrt{F_{py}^2 - 3\left(\frac{F_o'}{w' t_p}\right)^2} \tag{69}$$

$$Q_{max,i} = \frac{2.081 \cdot 0.5^2}{4 \cdot 1.006}\sqrt{50^2 - 3 \cdot \left(\frac{8.308}{2.081 \cdot 0.5}\right)^2} = 6.212\,\texttt{kip} \tag{70}$$

End-plate yielding

$$s = \frac{1}{2}\sqrt{b_p g} = \frac{1}{2}\sqrt{5.787 \cdot 3.387} = 2.214\,\texttt{in} \tag{71}$$

Dimension s is larger than dimension d_e, therefore case 2 applies.

$$Y = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{f,i}} + \frac{1}{s}\right) + h_o\left(\frac{1}{p_{f,o}} + \frac{1}{2s}\right)\right] + \frac{2}{g}[h_1(p_{f,i} + s) + h_o(d_e + p_{f,o})] \tag{72}$$

$$\begin{aligned} Y = {} & \frac{5.787}{2}\left[8.115\left(\frac{1}{1.759} + \frac{1}{2.214}\right) + 12.315\left(\frac{1}{2} + \frac{1}{2 \cdot 2.214}\right)\right] + \\ & + \frac{2}{3.387}[8.115(1.759 + 2.214) + 12.315(1.5 + 2)] = 94.310\,\texttt{in} \end{aligned} \tag{73}$$

$$\frac{M_n}{\Omega} = \frac{M_{pl}}{\Omega} = \frac{F_{py} t_p^2 Y}{\Omega} = \frac{500.5^2 \cdot 94.310}{1.67} = 705.911\,\texttt{kip-in} \tag{74}$$

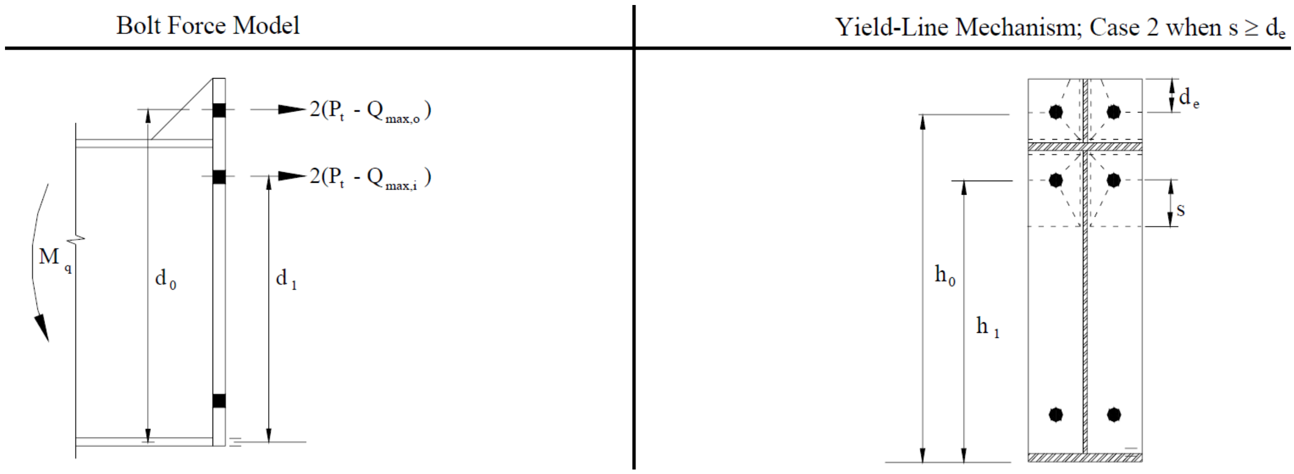

Figure 28: End-plate yield mechanism (Design guide 16)

Bolt rupture with prying action

$$M_n = \max \left| \begin{array}{l} 2[(P_t - Q_{max,o})d_o + 2(P_t - Q_{max,i})d_1] \\ 2[(P_t - Q_{max,o})d_o + 2(T_b)d_1] \\ 2[(P_t - Q_{max,o)}d_o + 2(T_b)d_1] \\ 2[(T_b)d_o + 2(T_b)d_1] \end{array} \right. \tag{75}$$

$$M_n = \max \left| \begin{array}{l} 2[(39.8 - 6.212) \cdot 12.094 + 2(39.8 - 6.137)7.895] = 1342.40\,\texttt{kip} \\ 2[(39.8 - 6.212)12.094 + 2(14)7.895] = 1032.55\,\texttt{kip} \\ 2[(39.8 - 6.212)12.094 + 2(14)7.895] = 869.54\,\texttt{kip} \\ 2[(14)12.094 + 2(14)7.895] = 559.69\,\texttt{kip} \end{array} \right. \tag{76}$$

$$\frac{M_n}{\Omega} = \frac{1342.4}{2} = 671.198\,\texttt{kip} \tag{77}$$

Bolt rupture without prying action

$$\frac{M_n}{\Omega} = \frac{2P_t(d_o + d_1)}{\Omega} \frac{2 \cdot 39.8 \cdot (12.095 + 7.895)}{2} = 795.602\,\texttt{kip} \tag{78}$$

The decisive failure mode is the one with smallest strength, i.e. bolt rupture with prying action, $\frac{M_n}{\Omega} = 671.198\,\texttt{kip}$.

3.2.2 Weld strength

In manual assessment, it is assumed that effective weld transferring bending moment is a cruciform consisting of the weld of the stiffener to the end-plate extension ($l = 3.5$ in, $w = 1/4''$), weld of the flange to the end-plate ($l = 5.787$ in, $w = 1/4''$), and weld of the estimated effective part of the web to the end-plate ($l = 3.5$ in, $w = 1/4''$). The center of gravity of such cruciform is conveniently at the beam flange, thus the lever arm is 9.874 in. The weld cruciform needs to transfer force $M_u/9.874 = 671/9.874 = 68$ kip.

$$A_{we} = 1/4 \cdot 2 \cdot (3.5 + 5.787 + 3.5)/\sqrt{(2)} = 4.52\,\texttt{in}^2 \tag{79}$$

$$F_{nw} = 0.6F_{EXX}(1 + 0.5\sin^{1.5}\theta) = 0.6 \cdot 70 \cdot (1 + 0.5\sin^{1.5} 40^\circ) = 53\,\texttt{ksi} \tag{80}$$

$$R_n/\Omega = F_{nw}A_{we}/\Omega = 53 \cdot 4.52/2 = 119.78\,\texttt{kip} \tag{81}$$

The weld strength is sufficient.

The weld strength of compressed welds is not checked here, because it is expected that the loads are transferred by direct contact.

3.3 Check in IDEA StatiCa

In IDEA StatiCa Connection, all prying forces and yield lines are determined automatically by finite element analysis. The bolt forces are shown with included prying forces. The point of rotation is also calculated automatically and requires no educated guess. All welds are checked and no force transfer by contact is assumed. The workaround would be setting of contact or butt weld instead of fillet weld.

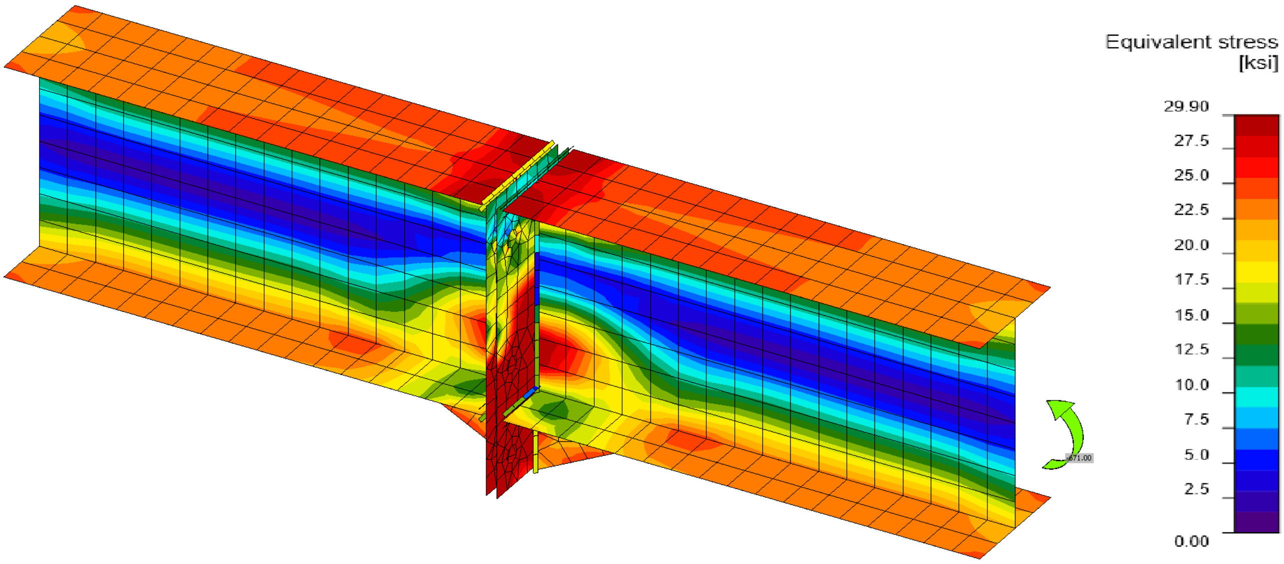

Figure 29: Von Mises stress

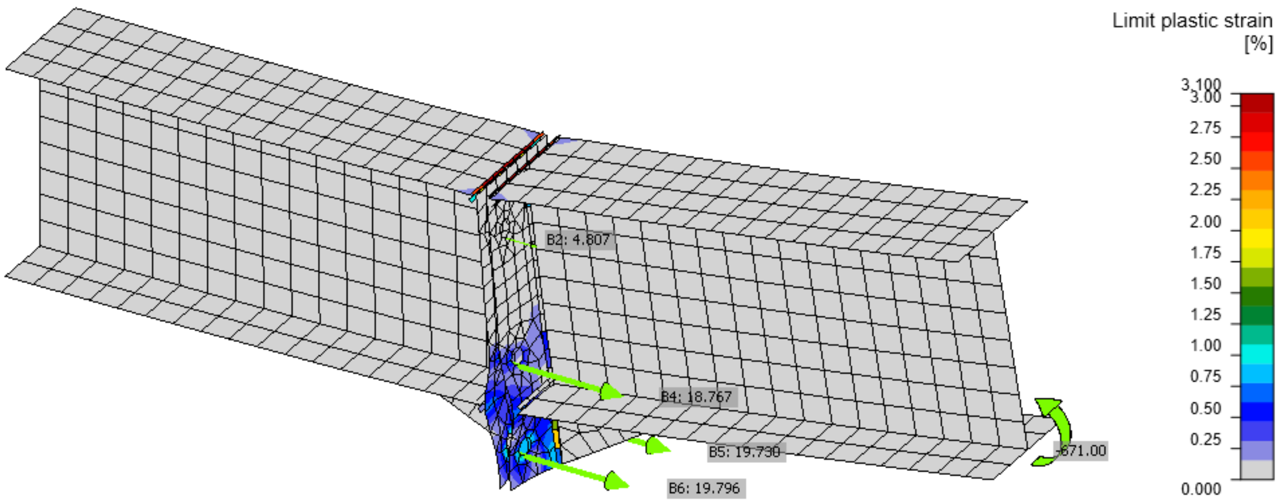

Figure 30: Plastic strain, applied load, and bolt forces on a deformed model (scale 10×)

The stiffness can also be easily evaluated in IDEA StatiCa Connection. This connection is close to the boundary of rigid and semi-rigid. The boundary depends on the length of connected beam.

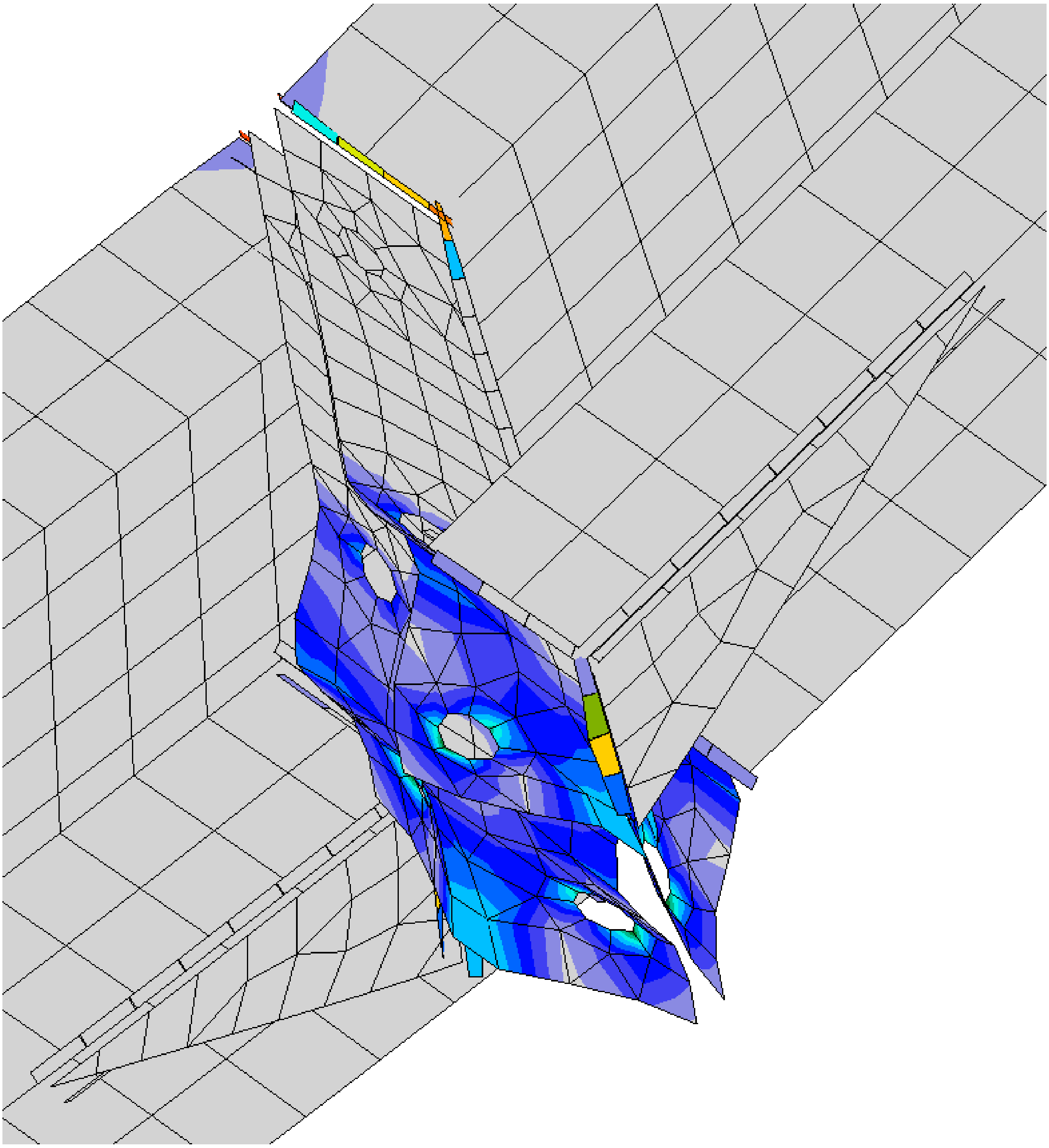

Figure 31: Detail of end plate deformation (scale 20×)

Check of members and steel plates for extreme load effect

	Status	Item	Th [in]	Loads	σ,Ed [ksi]	ε,Pl [%]
	✓	W10x26 (R)-bfl 1	7/16"	LE1	26.1	0.0
>	✓	W10x26 (R)-tfl 1	7/16"	LE1	30.0	0.2
	✓	W10x26 (R)-w 1	1/4"	LE1	29.7	0.1
	✓	W10x26 (L)-bfl 1	7/16"	LE1	26.1	0.0
	✓	W10x26 (L)-tfl 1	7/16"	LE1	30.0	0.2
	✓	W10x26 (L)-w 1	1/4"	LE1	29.7	0.1
	✓	PP1a	1/2"	LE1	30.5	1.8
	✓	PP1b	1/2"	LE1	30.5	1.8
	✓	WID1	1/2"	LE1	28.9	0.0
	✓	WID2	1/2"	LE1	28.9	0.0

Design data

	Grade	Fy [ksi]	ε,lim [%]
>	A572 Gr.50	50.0	5.0

Figure 32: Check of stress and strain in plates

Check of bolts for extreme load effect

		Status	Item	Loads	Ft [kip]	V [kip]	Bearing Rn/Ω [kip]	Utt [%]	Uts [%]	Utts [%]
>	+	✓	B1	LE1	4.810	0.000	29.250	24.2	0.0	-
	+	✓	B2	LE1	4.807	0.000	29.250	24.2	0.0	-
	+	✓	B3	LE1	18.759	0.000	29.250	94.4	0.0	-
	+	✓	B4	LE1	18.767	0.000	29.250	94.5	0.0	-
	+	✓	B5	LE1	19.730	0.000	29.250	99.3	0.0	-
	+	✓	B6	LE1	19.796	0.000	29.250	99.7	0.0	-

Design data

	Grade	Tension Rn/Ω [kip]	Shear Rn/Ω [kip]
>	3/4 A325 - 1	19.863	11.918

Figure 33: Check of bolts

Check of welds for extreme load effect (Plastic redistribution)

Status	Item	Edge	Xu	Th [in]	Ls [in]	L [in]	Lc [in]	Loads	Fn [kip]	Rn/Ω [kip]	Ut [%]
	PP1a	W10x26 (R)-bfl 1	E70xx	◢3/16"◣	◢1/4"◣	5.760	0.823	LE1	3.445	4.581	75.2
			E70xx	◢3/16"◣	◢1/4"◣	5.774	0.825	LE1	3.441	4.513	76.2
	PP1a	W10x26 (R)-tfl 1	E70xx	◢3/16"◣	◢1/4"◣	5.760	0.823	LE1	4.148	4.582	90.5
			E70xx	◢3/16"◣	◢1/4"◣	5.760	0.823	LE1	3.980	4.582	86.9
	PP1a	W10x26 (R)-w 1	E70xx	◢3/16"◣	◢1/4"◣	9.850	0.821	LE1	3.934	4.560	86.3
			E70xx	◢3/16"◣	◢1/4"◣	9.850	0.821	LE1	3.933	4.560	86.3
	PP1b	W10x26 (L)-bfl 1	E70xx	◢3/16"◣	◢1/4"◣	5.760	0.823	LE1	3.445	4.581	75.2
			E70xx	◢3/16"◣	◢1/4"◣	5.774	0.825	LE1	3.441	4.513	76.2
	PP1b	W10x26 (L)-tfl 1	E70xx	◢3/16"◣	◢1/4"◣	5.760	0.823	LE1	4.148	4.582	90.5
			E70xx	◢3/16"◣	◢1/4"◣	5.760	0.823	LE1	3.980	4.582	86.9
	PP1b	W10x26 (L)-w 1	E70xx	◢3/16"◣	◢1/4"◣	9.850	0.821	LE1	3.933	4.560	86.2
			E70xx	◢3/16"◣	◢1/4"◣	9.850	0.821	LE1	3.934	4.560	86.3
	PP1a	WID1	E70xx	◢3/16"◣	◢1/4"◣	3.484	0.871	LE1	4.135	4.850	85.3
			E70xx	◢3/16"◣	◢1/4"◣	3.484	0.871	LE1	4.237	4.850	87.4
	W10x26 (R)-bfl 1	WID1	E70xx	◢3/16"◣	◢1/4"◣	6.984	0.873	LE1	2.641	3.508	75.3
			E70xx	◢3/16"◣	◢1/4"◣	6.984	0.873	LE1	2.640	3.508	75.3
	PP1b	WID2	E70xx	◢3/16"◣	◢1/4"◣	3.484	0.871	LE1	4.238	4.850	87.4
			E70xx	◢3/16"◣	◢1/4"◣	3.484	0.871	LE1	4.134	4.850	85.2
	W10x26 (L)-bfl 1	WID2	E70xx	◢3/16"◣	◢1/4"◣	6.984	0.873	LE1	2.641	3.510	75.3
			E70xx	◢3/16"◣	◢1/4"◣	6.984	0.873	LE1	2.641	3.509	75.3

Figure 34: Check of welds

Rotational stiffness of joint component

	Item	Comp.	Loads	MEd [kip.in]	Mj,Rd [kip.in]	Sj,ini [kip-in/deg]	Sjs [kip-in/deg]	Φ [mrad]	Φc [mrad]	L [in]	Sj,R [kip-in/deg]	Sj,P [kip-in/deg]	Class
>	W10x26 (R)	My	LE1	-671.00	-648.78	6043.4	756.8	-15.5	-23.5	250.000	5838.4	583.8	Rigid

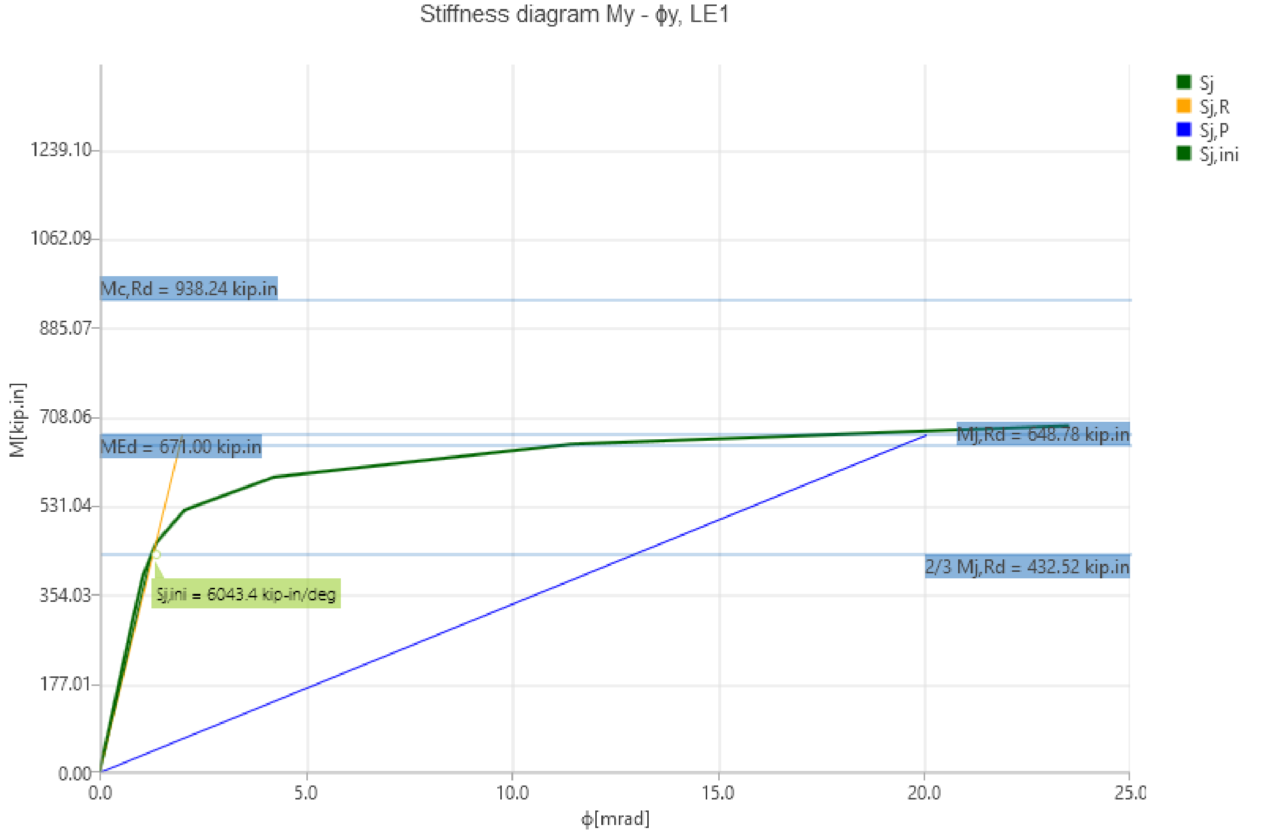

Figure 35: Stiffness of the joint

3.4 Comparison

IDEA StatiCa Connection provides the same results as the manual assessment. Bolts are utilized at 99.7 %, the plates of the end-plates are yielding, the plastic strain is 1.8 %, which means that the failure mode of end-plate yielding is close. The deformed shape coincides with the assumed deformation in Design Guide 16. The utilization at 100 % is at bending moment 673 kip-in (0.3 % difference).

4 Conclusion

Relatively simple connections were shown here for feasible manual assessment. In all presented cases, IDEA StatiCa Connection provided good agreement with manual assessment. Not less important is the presentation of results that supplies deep insight into the behavior of the joint and allows for better utilization of all elements.

However, the strength of the software lies in complex connections with complex loading where manual assessment is extremely difficult and most of the work-flow cannot be found in design guides and manuals.

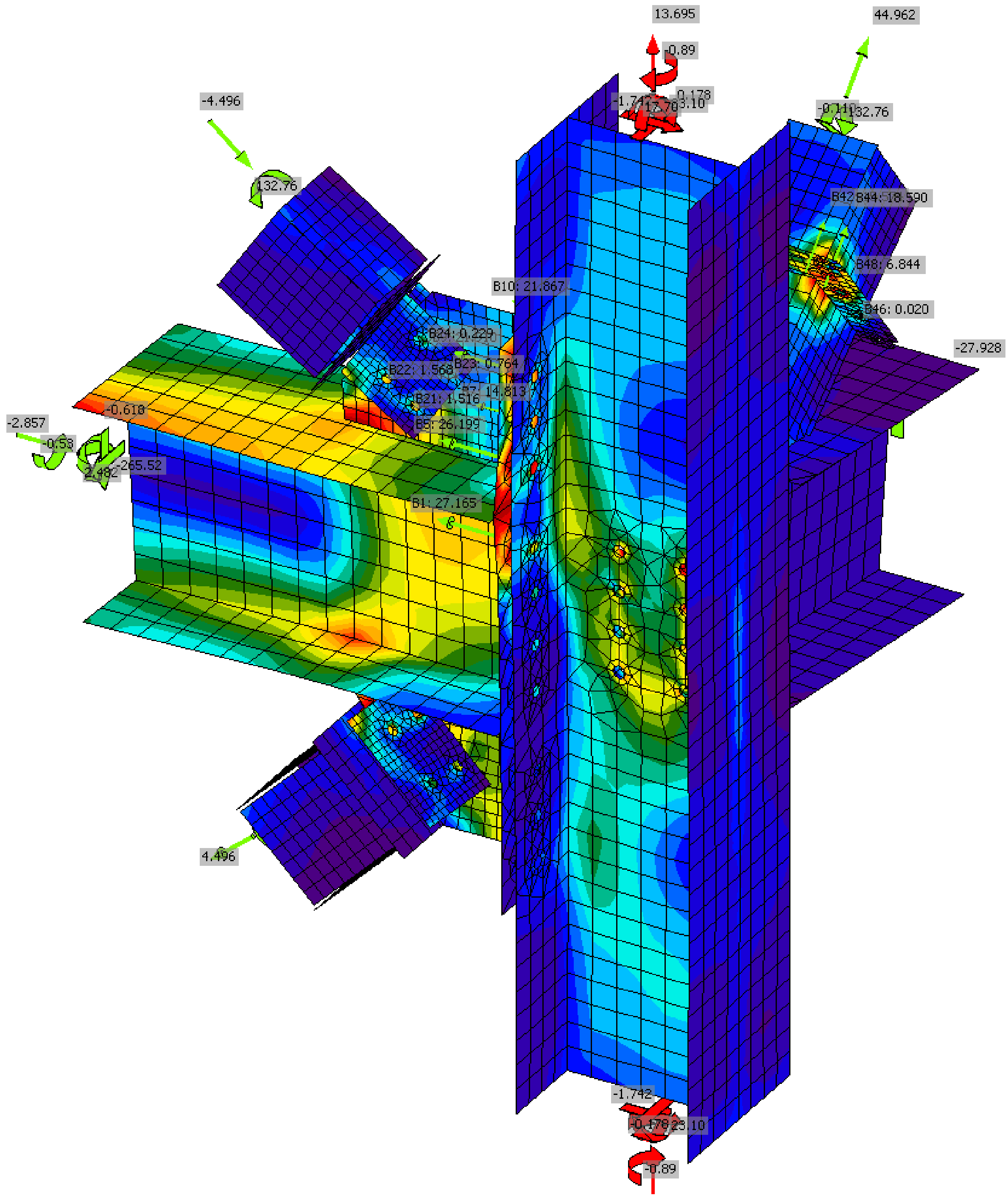

Figure 36: Complex joint analyzed in IDEA StatiCa Connection