WRAP-AROUND GUSSET PLATES

Where a horizontal brace is located at a beam-to-column intersection, the gusset plate must be cut out around the column as shown in Figure 1. These are called wrap-around gusset plates. At locations with large columns and heavy beam connection angles, a large area of the gusset plate is cut out as shown in Figure 2. In addition to the limit states presented in this chapter, the designer should also investigate the limit states that would normally be checked for standard gusset plates, such as bolt strength, weld strength, shear fracture and block shear.

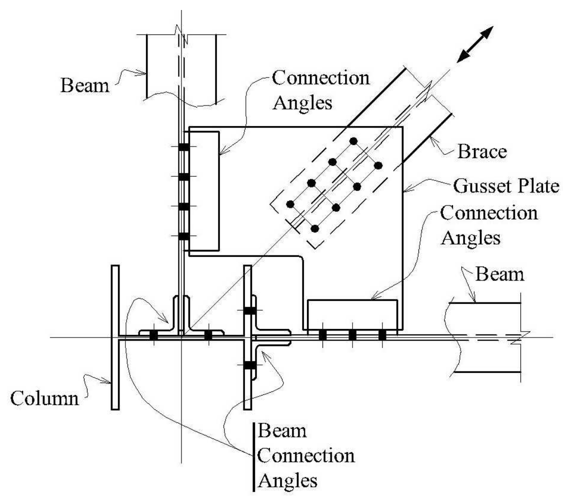


Figure 1. Horizontal brace connection at beam-to-column intersection.

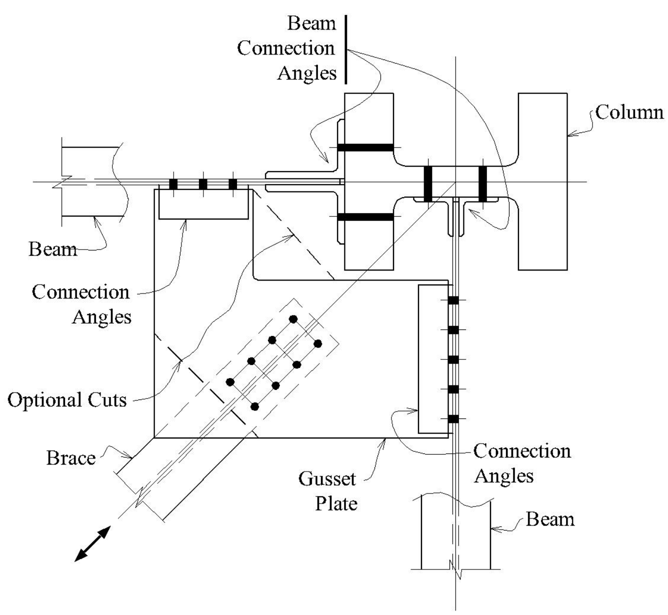


Figure 2. Horizontal brace connection at beam-to-column intersection.

FORCE DISTRIBUTION

The assumed force distribution in wrap-around gusset plate connections is shown in Figure 3. The legs of wrap-around gusset plates are subject to limit states common to flexural members; therefore, each leg of the gusset plate is modeled as a cantilever beam. For design purposes, the legs are assumed independent of one another.

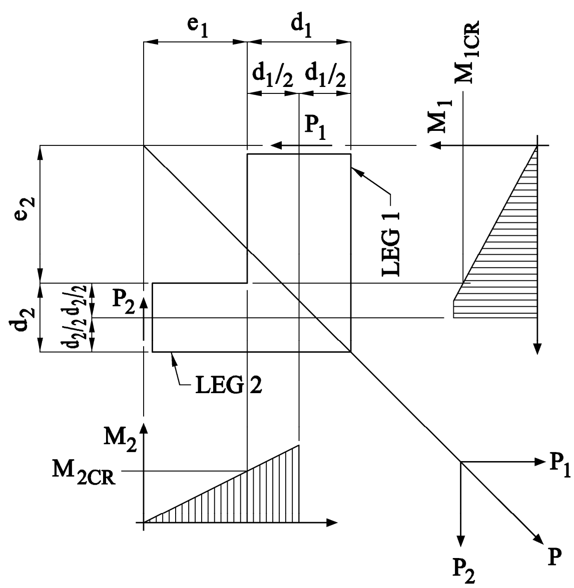


Figure 3. Force system for wrap-around gusset plates.

SHEAR STRENGTH

The nominal shear strength of each leg is,

$$V_{n1} = 0.6 F_y d_1 t \tag{1a}$$

$$V_{n2} = 0.6 F_y d_2 t \tag{1b}$$

where t is the gusset plate thickness, d_1 and d_2 are the depths of the gusset plate legs, and F_y is the yield strength. For the design to be adequate, the following must be satisfied,

$$\phi V_{n1} \geq P_1 \tag{2a}$$

$$\phi V_{n2} \geq P_2 \tag{2b}$$

where P_1 and P_2 are the factored components of P. For shear yielding, $\phi = 1.0$.

FLEXURAL STRENGTH

Each leg of the gusset plate must resist the flexural stresses generated by the force system in Figure 3. This force system results in maximum bending moments at the reentrant corner in each leg as shown in Figure 4. Figure 5 shows the stress contour plots for a finite element model loaded in tension. In Figure 5a, it can be seen that the highest von Mises stresses are concentrated at the reentrant corner where the two legs meet. Figure 5b shows the normal stresses in the x-direction and Figure 5c shows the normal stresses in the y-direction. The stresses are largest at the edges of the gusset plate legs. The stresses in the y-direction are higher than the stresses in the x-direction because of the larger cutout dimension in the y-direction. These stress patterns verify the accuracy of the proposed design model in determining the location of the maximum flexural stresses.

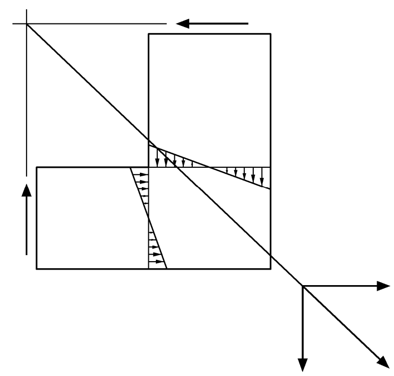


Figure 4. Bending stresses in each leg.

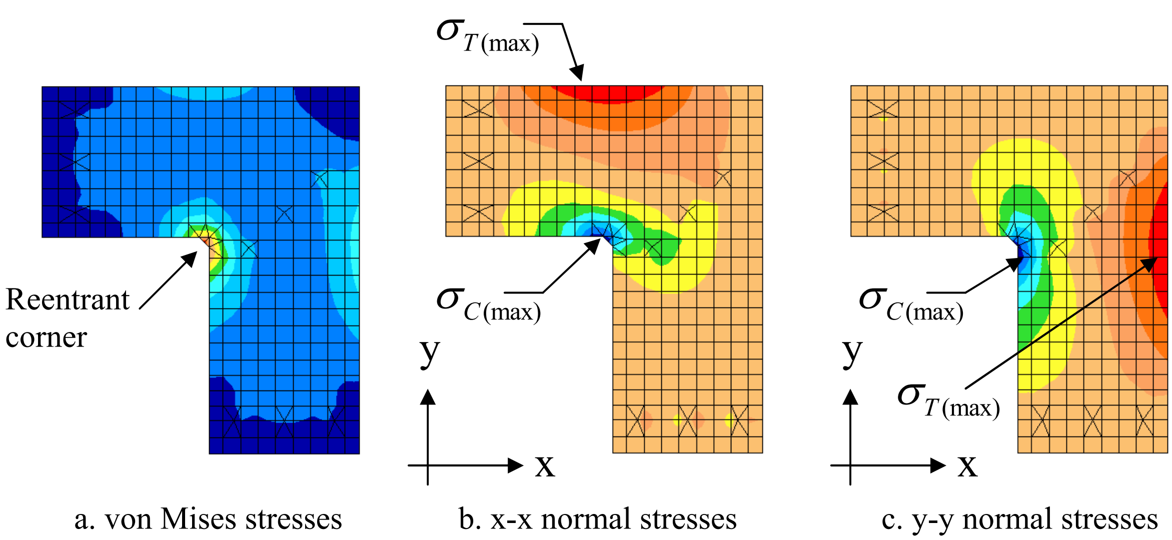


Figure 5. Elastic Stress contours for a typical model loaded in tension.

From strain gage data and finite element models, it was determined that the flexural stresses in the gusset plates exceeded the yield stress throughout much of the gusset plate. Although most of the plates had a substantial amount of the material above the proportional limit, none of the plates reached full plasticity before buckling. A typical plot of the elastic and inelastic stresses is shown in Figure 6 for a finite element model loaded to its failure load. The theoretical stresses, which were calculated using simple beam theory, are also shown in the figure. The proposed design method is based on an elastic bending stress distribution. The in-plane flexural deformation of Specimen 2T is shown in Figure 7.

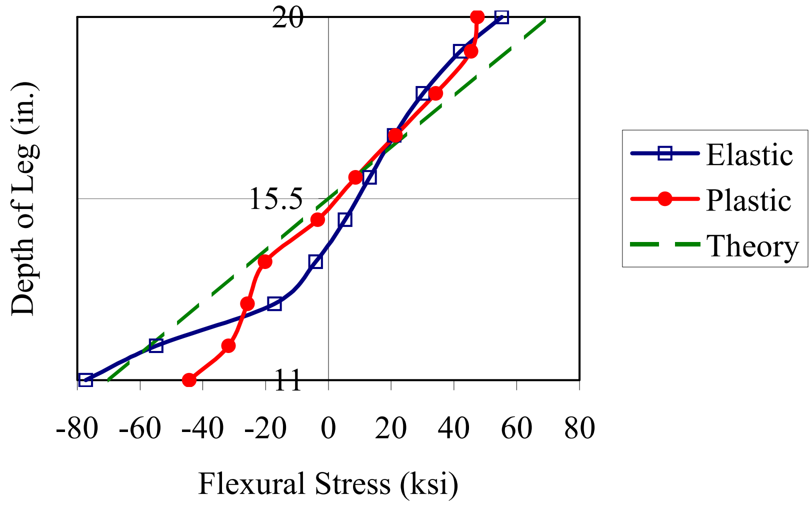


Figure 6. Typical flexural stresses in gusset plate legs.

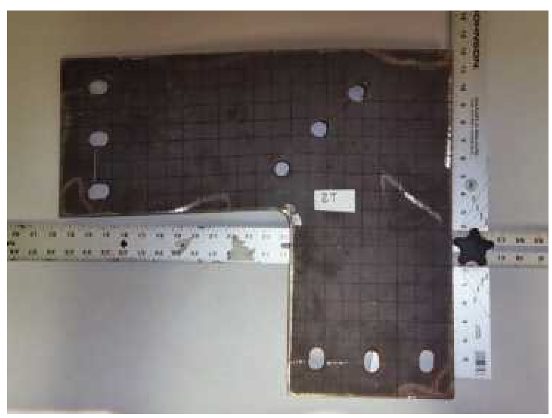


Figure 7. In-plane deformation of Specimen 2T.

The bending moments at the critical sections of the plate are

$$M_{u1} = P_1 e_2 \tag{3a}$$

$$M_{u2} = P_2 e_1 \tag{3b}$$

where P_1 and P_2 are the components of the factored brace load, P. e_1 and e_2 are the cutout dimensions at each leg, as shown in Figure 2. The nominal moment capacity of each leg is

$$M_{n1} = F_y \frac{t d_1^2}{6} \qquad (4a)$$

$$M_{n2} = F_y \frac{t d_2^2}{6} \qquad (4b)$$

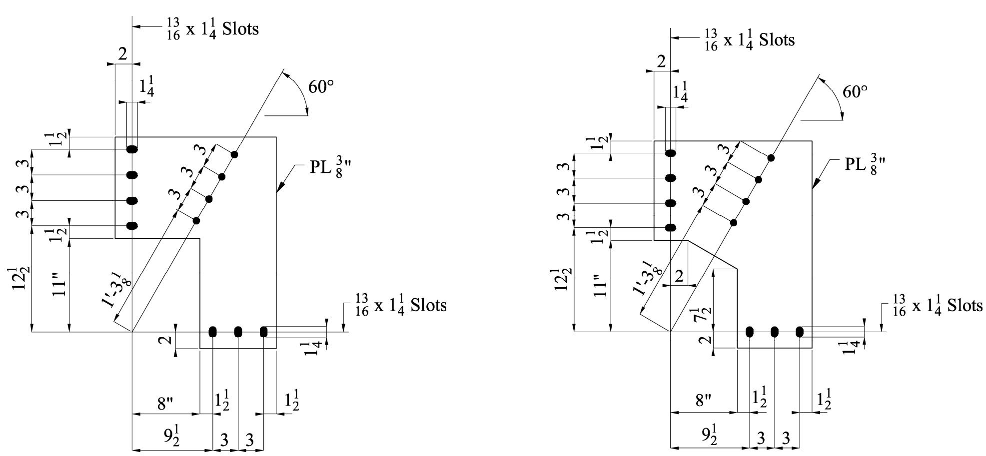
For the design to be adequate, the following must be satisfied:

$$\phi M_{n1} \geq M_{u1} \qquad (5a)$$

$$\phi M_{n2} \geq M_{u2} \qquad (5b)$$

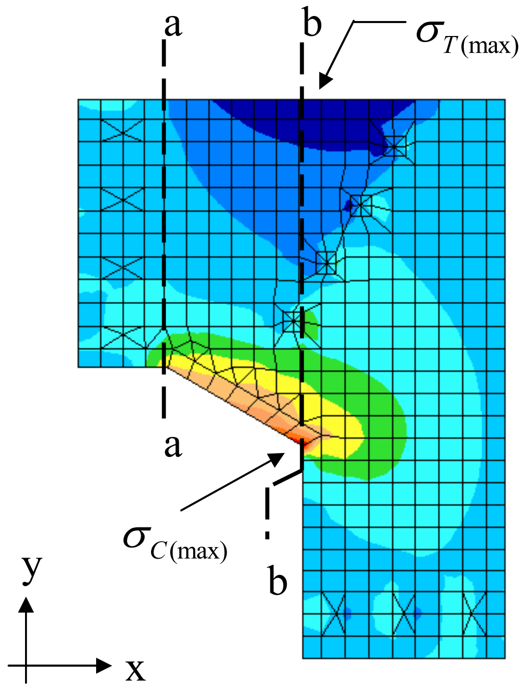
where $\phi = 0.9$ for flexural yielding.

Sometimes wrap-around gusset plates have the interior corner cut on a diagonal as shown in Figure 8b in an effort to increase their capacity. The test Specimens 8 and 10, shown in Figure 8, were identical except for the diagonal cut on Specimen 10. The tests and finite element models showed that the average capacity for Specimen 10 was 22 percent higher than the average capacity for Specimen 8. Figure 9 shows the stress contour plots for Specimen 10. Figure 9a shows the normal stresses in the x-direction and Figure 9b shows the normal stresses in the y-direction. The moment capacity at cross sections *a-a* and *b-b* should be checked at each leg using Equations 3, 4, and 5. Calculations show that the flexural stresses in the x-direction at Section *b-b* are 2.40 times the stresses at Section *a-a*. The finite element stresses in Figure 8a confirm this. Similarly, Figure 8b confirms that Section a-a controls the design for the stresses in the y-direction. The calculated flexural stresses in the y-direction are 89% higher at Section a-a than they are at Section *b-b*.

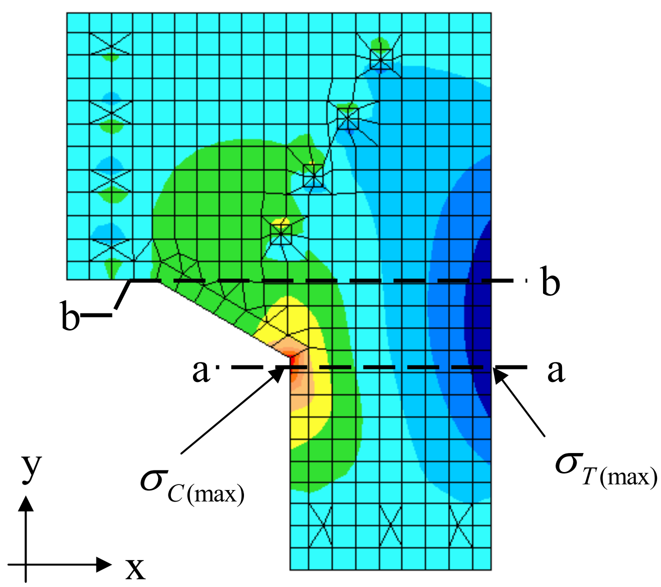


a. Specimen 8 b. Specimen 10

Figure 8. Details of Specimens 8 and 10.



a. x-direction normal stresses



b. y-direction normal stresses

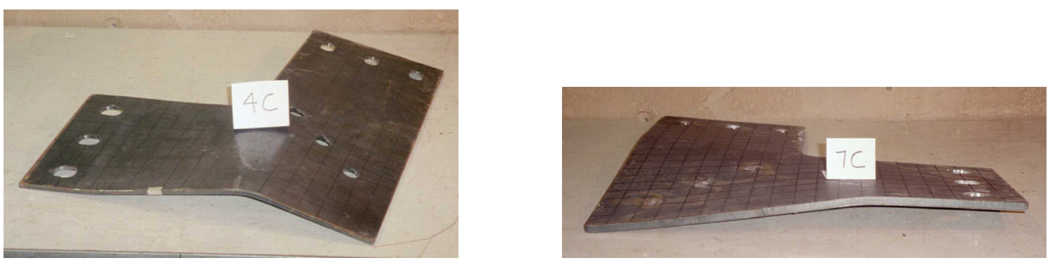
Figure 9. Elastic Stress contours for a typical model loaded in tension.

LATERAL-TORSIONAL BUCKLING

Due to the flexural stresses in the gusset plate legs, they are subject to lateral-torsional buckling. Tests showed that the flexural stresses in the legs can cause lateral-torsional buckling, even if the brace is loaded in tension. The permanent deformation in tension and compression specimens can be seen in Figures 10 and 11 respectively. All of the specimens had a permanent out-of-plane deformation at the plate edges with flexural compression stresses. The out-of-plane deformation was accompanied by twisting of the gusset plate legs, indicating a lateral-torsional buckling failure. The finite element models are shown in Figures 12 and 13 for tension and compression loads respectively.

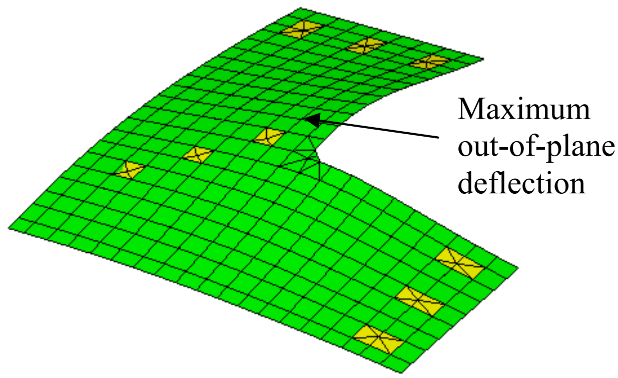
a. top view. b. side view of longer edge.

Figure 10. Tension Specimen 2T after test.

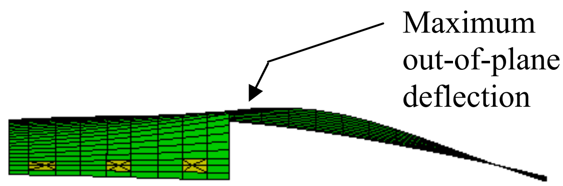


a. Specimen 4C b. Specimen 7C

Figure 11. Compression Specimens 4C and 7C after test.



a. Top view.



b. Side View.

Figure 12. Typical buckled shape for the models loaded in tension.

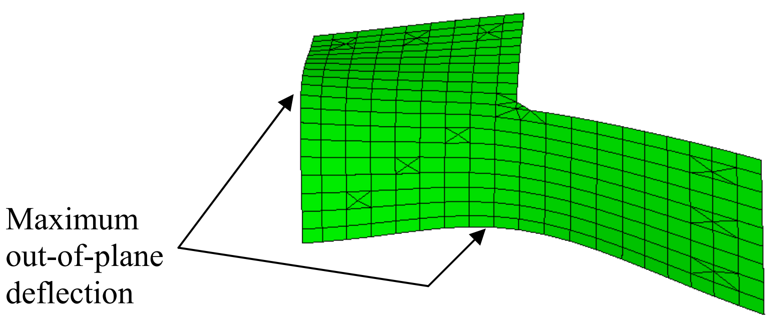


Figure 13. Typical buckled shape for the models loaded in compression.

Each leg of the gusset plate can be modeled as a cantilever beam to determine the buckling load. Using the equations developed by Dowswell (2004) for wide flange cantilever beams, the critical moment is,

$$M_{cr} = 0.94\sqrt{EG}\,\frac{d_i t^3}{L_i} \tag{9}$$

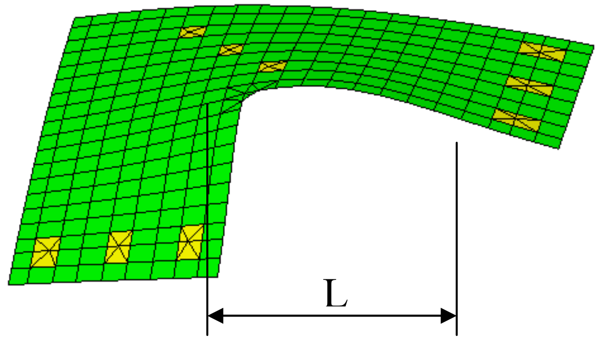
Where E is the modulus of elasticity and G is the shear modulus. The design buckling capacity must be greater than the internal moment at each gusset plate leg.

$$\phi M_{cr1} \geq M_{u1} \tag{10a}$$

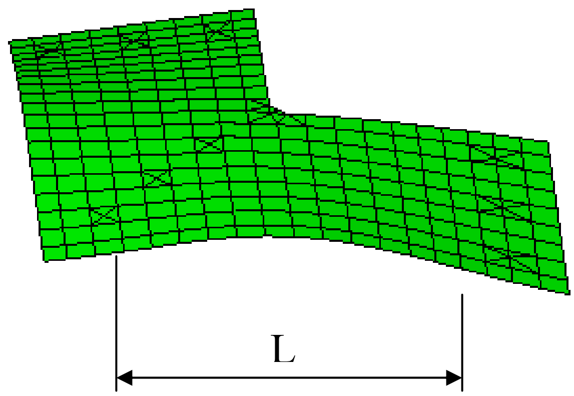
$$\phi M_{cr2} \geq M_{u2} \tag{10b}$$

where M_{u1} and M_{u2} are determined with Equations 3a and 3b respectively, and ϕ = 0.9 for lateral-torsional buckling.

To determine the buckling length, L_i to be used in Equation 9, the buckled shape of the specimens and finite element models was observed. For the specimens loaded in tension, the inside edges buckled farther than the outside edges. This behavior was expected because the maximum compressive flexural stresses are on the inside edges, at the reentrant corner. The specimens loaded in compression buckled farther on the outside edges. Figure 14 shows how this behavior affects the buckling length of the legs. The plate in Figure 14a was loaded in tension. The buckling of each leg is restrained at the reentrant corner and the buckling length, L_i is the length of the cutout. The following buckling lengths can be used when the plate is loaded in tension: $L_1 = e_2$ for Leg 1, and $L_2 = e_1$ for Leg 2. For plates loaded in compression, as shown in Figure 14b, the buckling length ends approximately at the center of the adjacent leg. For design purposes, the following buckling lengths can be used when the plate is loaded in compression: $L_1 = e_2 + d_2/2$ for Leg 1, and $L_2 = e_1 + d_1/2$ for Leg 2.



a. Plates loaded in tension.



b. Plates loaded in compression.

Figure 14. Effective length of gusset legs.

For plates with a diagonal cut as shown in Figure 7b, which are loaded in tension, the buckling length can be taken as the portion of the leg with parallel edges, measured to the start of the diagonal cut. For plates with a diagonal cut loaded in compression, the buckling length is determined the same as for standard gusset plates.

VAILIDATION OF DESIGN METHOD

The nominal capacity of the plate, P_{min} is the minimum of the bending, shear, and buckling limit states. The capacities for each specimen were calculated using the proposed design method, and summarized in Table 1. All of the loads are expressed as the maximum nominal load parallel to the brace based on the minimum capacity of the two legs. P_e is the bending capacity, P_v is the shear capacity, P_b is the lateral-torsional buckling capacity, and P_{min} is the minimum of P_e, P_v and P_b. The specimen numbers are suffixed with "T" if the plate was loaded in tension, and "C" if it was loaded in compression.

Table 1. Calculated capacities.

Spec. No.	P_e	P_v	P_b	P_{min}	Pred. Failure Mode
2T	33.12	145.7	105.94	33.12	Y
6T	31.79	101.7	43.48	31.79	Y
8T	46.00	155.8	141.92	46.00	Y
9T	35.77	115.6	84.90	35.77	Y
10T	68.00	204.0	145.27	68.00	Y
1C	39.52	115.9	56.80	39.52	Y
2C	32.87	144.6	73.46	32.87	Y
3C	36.14	115.6	96.08	36.14	Y
4C	23.12	101.7	16.32	16.32	B
5C	25.84	82.70	22.43	22.43	B
6C	31.79	101.7	27.83	27.83	B
7C	45.07	144.2	124.29	45.07	Y
8C	46.48	157.4	94.73	46.48	Y
9C	36.71	118.6	69.46	36.71	Y
10C	67.47	202.4	91.83	67.47	Y

P_e calculated elastic bending capacity
P_v calculated shear capacity
P_b calculated lateral-torsional buckling capacity
P_{min} minimum of P_e, P_v and P_b (proposed nominal capacity)

Y: yielding
B: buckling

The experimental results are summarized in Table 2. P_{ey} is the experimental yield load, P_{fy} is the yield load from the finite element models, and P_{avg} is the average of P_{ey} and P_{fy}.

Table 2. Experimental and finite element loads.

Spec. No.	P_{ey}	P_{fy}	P_{avg}	$\frac{P_{avg}}{P_{min}}$	Exp. Failure Mode
2T	69.0	50.7	59.85	1.81	Y/B
6T	42.3	39.9	41.10	1.29	Y/B
8T	85.3	73.8	79.55	1.73	Y/B
9T	51.5	46.9	49.20	1.38	Y/B
10T	96.2	84.8	90.50	1.33	Y/B
1C	33.3	51.6	42.45	1.07	Y/B
2C	47.3	44.7	46.00	1.40	Y
3C	46.6	45.0	45.80	1.27	Y/B
4C	32.0	30.2	31.10	1.91	B
5C	28.7	36.3	32.50	1.45	Y/B
6C	25.3	34.5	29.90	1.07	B
7C	46.4	51.3	48.85	1.08	Y/B
8C	38.4	67.6	53.00	1.14	Y
9C	44.4	38.7	41.55	1.13	Y/B
10C	57.0	76.6	66.80	0.99	Y/B

P_{ey} experimental yield load determined using a 1/64 in. offset

P_{fy} finite element yield load determined using a 1/64 in. offset

P_{avg} average of P_{ey} and P_{fy}

The P_{avg} to P_{min} ratios are in the fifth column of Table 2. P_{avg}/P_{min} varied from 0.99 to 1.91 with an average of 1.34 and a standard deviation of 0.28. Dowswell and Barber (2004) summarized the test results for compact corner gusset plates in compression, and found the current design equations to be conservative by an average of 47 percent with a standard deviation of 0.23. Based on this data, the accuracy of the proposed design procedure for wrap-around gusset plates is similar to the accuracy of the current design procedure for standard gusset plates.

The experimental failure modes are summarized in the sixth column of Table 2. All of the specimens failed by a combination of buckling and yielding; therefore it was difficult to determine whether the correct failure mode was predicted with the proposed design method. Two of the specimens, Specimens 2C and 8C, were almost fully yielded before buckling occurred due to a loss of stiffness. Both of these specimens also had a predicted failure mode of yielding as shown in Table 1. Specimens 4C and 6C buckled while most of the plate material was in the elastic range. The predicted failure mode for both of these plates was also buckling. For these four tests with definite failure modes, the design model predicted the correct failure mode.

EXAMPLES

Example 1

The strength of the wrap around gusset plate connection in Figure 15 will be checked using LRFD. The bottom of the brace is down 14¼ in. from the top of steel.

Axial force in brace: 35 kips tension or compression
Gusset plate thickness: ⅜ in.
Gusset plate material: A572 Grade 50
Clip angle material: A36
W and WT material: A992
Bolts: ¾-in. diameter A325N
Weld: 70 ksi AWS
Edge Distance: 1¼ in. unless shown otherwise
Holes: Standard 13⁄16-in. diameter

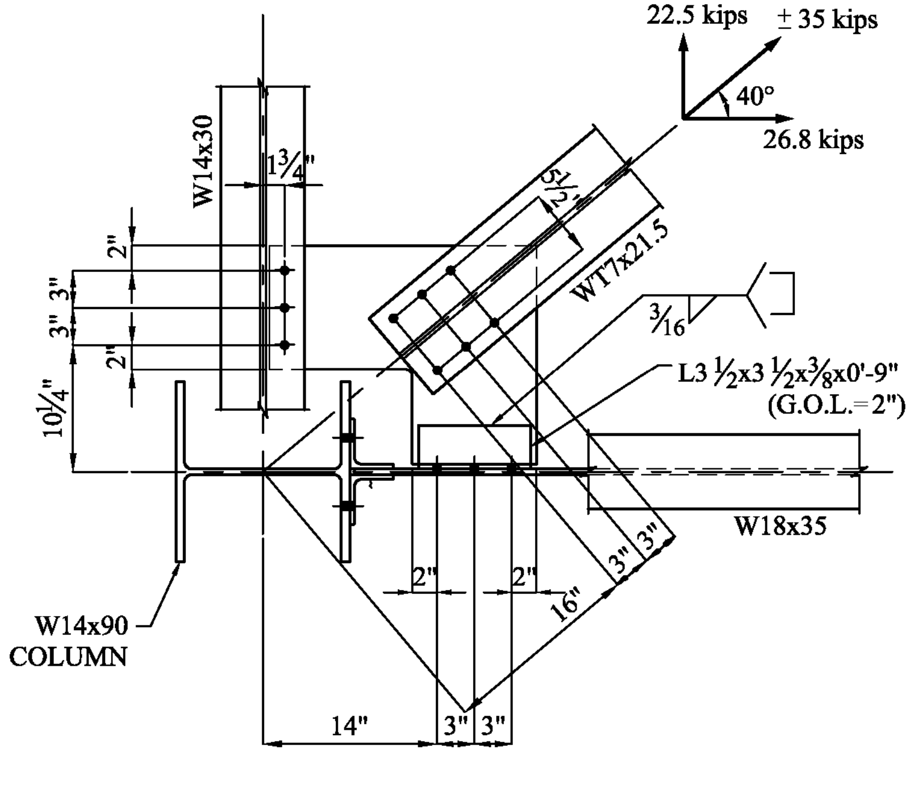


Figure. E1-1. Connection for Example 1.

Brace-to-Gusset Plate

Bolt shear fracture

From Manual Table 7-1, the design shear strength for each bolt, $\phi r_n = 15.9 \text{ kips/bolt}$.

$\phi R_{nv} = (6\text{ bolts})(15.9\text{ kips/bolt}) = 95.4\text{ kips} > 35\text{ kips}$ **o.k.**

Bolt bearing at gusset plate

The smallest edge distance is for the bolt closest to the W18. The distance in the direction of load, from the center of the bolt to the plate edge, is 2.64 in.

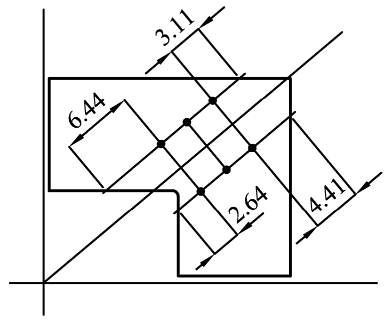


Figure E1-2.

$L_c = 2.64\text{ in.} - (1/2)(13/16\text{ in.}) = 2.23\text{ in.}$

$\phi R_n = (0.75)(1.2)(2.23\text{ in.})(3/8\text{ in.})(65\text{ ksi}) \le (0.75)(2.4)(3/4\text{ in.})(3/8\text{ in.})(65\text{ ksi})$

$\phi R_n = 48.9\text{ kips / bolt} > 32.9\text{ kips / bolt}$. Use $\phi R_n = 32.9\text{ kips / bolt}$

Between the bolts,

$L_c = 3\text{ in.} - 13/16\text{ in.} = 2.19\text{ in.}$

$\phi R_n = (0.75)(1.2)(2.19\text{ in.})(3/8\text{ in.})(65\text{ ksi}) \le (0.75)(2.4)(3/4\text{ in.})(3/8\text{ in.})(65\text{ ksi})$

$\phi R_n = 48.0\text{ kips / bolt} > 32.9\text{ kips / bolt}$. Use $\phi R_n = 32.9\text{ kips / bolt}$

$\phi R_n = (32.9\text{ kips / bolt})(6\text{ bolts}) = 197\text{ kips} > 35\text{ kips}$ **o.k.**

Bolt bearing at brace flange

Holes at end of brace:

$L_c = 1\tfrac{1}{4}\text{ in.} - (1/2)(13/16\text{ in.}) = 0.844\text{ in.}$

$\phi R_n = (0.75)(1.2)(0.844\text{ in.})(0.530\text{ in.})(65\text{ ksi}) \le (0.75)(2.4)(3/4\text{in.})(0.530\text{ in.})(65\text{ ksi})$

$\phi R_n = 26.2$ kips / bolt $\leq$ 46.5 kips / bolt . Use $\phi R_n = 26.2$ kips / bolt

Remaining holes:

$$L_c = 3 - 13/16 = 2.19 \text{ in.}$$

$$\phi R_n = (0.75)(1.2)(2.19 \text{ in.})(0.530 \text{ in.})(65 \text{ ksi}) \leq (0.75)(2.4)(3/4 \text{ in.})(0.530 \text{ in.})(65 \text{ ksi})$$
$\phi R_n = 67.9$ kips / bolt > 46.5 kips / bolt . Use $\phi R_n = 46.5$ kips / bolt

$$\phi R_n = (26.2 \text{ kips / bolt})(2 \text{ bolts}) + (46.5 \text{ kips / bolt})(4 \text{ bolts}) = 238 \text{ kips} > 35 \text{ kips}$$ **o.k.**

Whitmore yielding

$$L_w = 5\tfrac{1}{2} \text{ in.} + (2)(6 \text{ in.}) \cdot \tan(30°) = 12.4 \text{ in.}$$

$$A_w = (3/8 \text{ in.})(12.4 \text{ in.}) = 4.65 \text{ in.}$$

$$P_n = (4.65 \text{in})(50 \text{ ksi}) = 232 \text{ kips}$$

$$\phi P_n = (0.9)(232 \text{ kips}) = 209 \text{ kips} > 35 \text{ kips}$$ **o.k.**

Block shear of brace flange

$$A_{gv} = (7.25 \text{ in.})(0.530 \text{ in.}) = 3.84 \text{ in}^2$$

$$A_{nv} = [7.25 \text{ in.} - (2.5 \text{ holes})(7/8 \text{ in.})](0.530 \text{ in.}) = 2.68 \text{ in}^2$$

$$A_{gt} = \frac{8.00 \text{ in.-}5\tfrac{1}{2} \text{ in.}}{2} = 1.25 \text{ in}^2$$

$$A_{nt} = [1\tfrac{1}{4} \text{ in.} - (1/2 \text{ hole})(7/8 \text{ in.})](0.530 \text{ in.}) = 0.431 \text{ in}^2$$

$U_{bs} = 1.0$ $\qquad \phi = 0.75$

$$R_n = (2)\left(0.6F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6F_y A_{gv} + U_{bs} F_u A_{nt}\right)$$

$$0.6F_u A_{nv} + U_{bs} F_u A_{nt} = (0.6)(65 \text{ ksi })(2.68 \text{ in.}^2) + (1.0)(65 \text{ ksi})(0.431 \text{ in.}^2) = 132 \text{ kips}$$

$$0.6F_y A_{gv} + U_{bs} F_u A_{nt} = (0.6)(50 \text{ ksi})(3.84 \text{ in.}^2) + (1.0)(65 \text{ ksi})(0.431 \text{ in.}^2) = 143 \text{ kips}$$

$\phi R_n = (0.75)(2)(132 \text{ kips}) = 198 \text{ kips} > 35 \text{ kips}$ **o.k.**

Block shear of gusset plate

Due to the geometry of the gusset plate, block shear can occur in compression, where the shear planes extend into the cutout. For compression loads, the edge distances in the direction of load are 6.44 in. and 2.64 in., and for tension loads, the edge distances are 3.11 in. and 4.41 in. The sum of edge distances is smaller for tension loads; therefore, this will result in the lowest block shear capacity.

$$A_{gv} = \left[(2)(6 \text{ in.}) + 3.11 \text{ in.} + 4.41 \text{ in.}\right](3/8 \text{ in.}) = 7.32 \text{ in}^2$$

$$A_{nv} = 7.32 \text{ in.}^2 - (2)(2.5 \text{ holes})(7/8 \text{ in.})(3/8 \text{ in.}) = 5.68 \text{ in}^2$$

$$A_{nt} = \left[5.5 \text{ in.} - (2)(1/2 \text{ hole})(7/8 \text{ in.})\right](3/8 \text{ in.}) = 1.73 \text{ in}^2$$

$$U_{bs} = 1.0 \qquad \phi = 0.75$$

$$R_n = 0.6F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.6F_yA_{gv} + U_{bs}F_uA_{nt}$$

$$0.6F_uA_{nv} + U_{bs}F_uA_{nt} = (0.6)(65 \text{ ksi})(5.68 \text{ in.}^2) + (1.0)(65 \text{ ksi})(1.73 \text{ in.}^2) = 334 \text{ kips}$$

$$0.6F_yA_{gv} + U_{bs}F_uA_{nt} = (0.6)(50 \text{ ksi})(7.32 \text{ in.}^2) + (1.0)(65 \text{ ksi})(1.73 \text{ in.}^2) = 332 \text{ kips}$$

$\phi R_n = (0.75)(332 \text{ kips}) = 249 \text{ kips} > 35 \text{ kips}$ **o.k.**

Leg Connecting to W14

Shear yielding of gusset plate

$$\phi V_{n1} = (1.0)(0.6)F_yd_1t = (1.0)(0.6)(50 \text{ ksi})(10 \text{ in.})(3/8 \text{ in.}) = 112 \text{ kips}$$

$112 \text{ kips} > 22.5 \text{ kips}$ **o.k.**

Flexural yielding of gusset plate

$$\phi M_{n1} = (0.9)F_y\frac{td_1^2}{6} = (0.9)(50 \text{ ksi})\frac{(3/8 \text{ in.})(10 \text{ in.})^2}{6} = 281 \text{ kip}\cdot\text{in.}$$

$$M_{u1} = P_1e_2 = (22.5 \text{ kips})(12 \text{ in.}) = 270 \text{ kip}\cdot\text{in.}$$

$281 \text{ kip-in.} > 270 \text{ kip-in.}$ **o.k.**

Lateral-torsional buckling of gusset plate leg

L=17 in. for compression loads and 12 in. for tension loads. Use L=17 in.

$$\phi M_{cr1} = (0.9)(16{,}848)\frac{d_1 t^3}{L_1} = (0.9)(16{,}848)\frac{(10 \text{ in.})(3/8 \text{ in.})^3}{17 \text{ in.}} = 470 \text{ k}\cdot\text{in.}$$

$470 \text{ k-in.} > 270 \text{ k-in.}$ **o.k.**

Bolt shear fracture

The work point is located at the center of the W14; therefore, the bolts are subject to an eccentricity of 1¾ in. From Manual Table 7-1, the available shear strength for each bolt, $\phi r_n = 15.9 \text{ kips/bolt}$.

Interpolating from Manual Table 7-7 with Angle = 0°, S = 3 in., n = 3, and e_x = 1¾ in.: C = 2.33.

$\phi R_{nv} = (2.33 \text{ bolts})(15.9 \text{ kips/bolt}) = 37.0 \text{ kips} > 22.5 \text{ kips}$ **o.k.**

Bolt bearing at gusset plate

For bolt bearing, the effect of eccentricity must be accounted for; therefore, the effective number of bolts, C = 2.33, will be used from Manual Table 7-7. The smallest edge distance is 1¼ in., which is at the edge parallel to the beam axis. Although the load is parallel to this edge, the bolt group is loaded eccentrically, and each bolt has a skewed resultant load, with at least a small component perpendicular to the smallest edge. It is conservative to assume the 1¼ in. edge distance applies to all bolts in the bearing calculation.

$$L_c = 1\tfrac{1}{4} \text{ in.} - (1/2)(13/16 \text{ in.}) = 0.844 \text{ in.}$$

Between the bolts,

$$L_c = 3 \text{ in.} - 13/16 \text{ in.} = 2.19 \text{ in.}$$

$0.844 \text{ in.} < 2.19 \text{ in.}$; use $L_c = 0.844 \text{ in.}$

$$\phi R_n = (0.75)(1.2)(0.844 \text{ in.})(3/8 \text{ in.})(58 \text{ ksi}) \le (0.75)(2.4)(3/4 \text{ in.})(3/8 \text{ in.})(58 \text{ ksi})$$

$\phi R_n = 16.5 \text{ kips / bolt} \le 29.4 \text{ kips / bolt}$. Use $\phi R_n = 16.5 \text{ kips / bolt}$

$\phi R_n = (16.5 \text{ kips / bolt})(2.33 \text{ bolts}) = 38.4 \text{ kips} > 22.5 \text{ kips}$ **o.k.**

Bolt bearing at beam flange

For bolt bearing, the effect of eccentricity must be accounted for; therefore, the effective number of bolts, C = 2.33, will be used from Manual Table 7-7. The edge distance perpendicular to the beam axis is

$$L = (1/2)(6.73 \text{ in.}) - 1\tfrac{3}{4}\text{ in.} = 1.62 \text{ in.}$$

$$L_c = 1.62 \text{ in.} - (1/2)(13/16 \text{ in.}) = 1.21 \text{ in.}$$

Between the bolts,

$$L_c = 3 \text{ in.} - 13/16 \text{ in.} = 2.19 \text{ in.}$$

$1.21 \text{ in.} < 2.19 \text{ in.}$; use $L_c = 1.21 \text{ in.}$

$$\phi R_n = (0.75)(1.2)(1.21 \text{ in.})(0.385 \text{ in.})(65 \text{ ksi}) \le (0.75)(2.4)(3/4 \text{ in.})(0.385 \text{ in.})(65 \text{ ksi})$$

$\phi R_n = 27.2 \text{ kips / bolt} \le 33.8 \text{ kips / bolt}$. Use $\phi R_n = 27.2 \text{ kips / bolt}$

$\phi R_n = (27.2 \text{ kips / bolt})(2.33 \text{ bolts}) = 63.4 \text{ kips} > 22.5 \text{ kips}$ **o.k.**

Shear fracture of gusset plate

$\phi R_n = (0.75)(0.6)(3/8 \text{ in.})[10 \text{ in.} - (3)(7/8 \text{ in.})](65 \text{ ksi}) = 80.9 \text{ kips} > 22.5 \text{ kips}$ **o.k.**

Flexural fracture of gusset plate at W14 bolts

$$Z_{net} = (2)\left[(1.50 \text{ in.})(2.12 \text{ in.})(3/8 \text{ in.}) + (4.22 \text{ in.})(1.56 \text{ in.})(3/8 \text{ in.})\right] = 7.32 \text{ in.}^3$$

$$\phi M_n = F_u Z_{net} = (65 \text{ ksi})(7.32 \text{ in.}^3) = 476 \text{ kip-in.}$$

$M_u = (22.5 \text{ kips})(1\tfrac{3}{4} \text{ in.}) = 39.4 \text{ kip-in.} < 476 \text{ kip-in.}$ **o.k.**

Block shear of gusset plate

$A_{gv} = (8 \text{ in.})(3/8 \text{ in.}) = 3.00 \text{ in}^2$

$A_{nv} = [8 \text{ in.} - (2.5 \text{ holes})(7/8 \text{ in.})](3/8 \text{ in.}) = 2.18 \text{ in}^2$

$A_{nt} = [1\tfrac{1}{4} \text{ in.} - (1/2 \text{ hole})(7/8 \text{ in.})](3/8 \text{ in.}) = 0.305 \text{ in}^2$

$U_{bs} = 1.0 \qquad \phi = 0.75$

$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6F_y A_{gv} + U_{bs} F_u A_{nt}$

$0.6F_u A_{nv} + U_{bs} F_u A_{nt} = (0.6)(65 \text{ ksi })(2.18 \text{ in.}^2) + (1.0)(65 \text{ ksi})(0.305 \text{ in.}^2) = 105 \text{ kips}$

$0.6F_y A_{gv} + U_{bs} F_u A_{nt} = (0.6)(50 \text{ ksi})(3.00 \text{ in.}^2) + (1.0)(65 \text{ ksi})(0.305 \text{ in.}^2) = 110 \text{ kips}$

$\phi R_n = (0.75)(105 \text{ kips}) = 78.8 \text{ kips} > 22.5 \text{ kips}$ **o.k.**

Leg Connecting to W18

Shear yielding of gusset plate

$\phi V_{n2} = (1.0)(0.6) F_y d_2 t = (1.0)(0.6)(50 \text{ ksi})(10 \text{ in.})(3/8 \text{ in.}) = 112 \text{ kips}$

$112 \text{ kips} > 26.8 \text{ kips}$ **o.k.**

Flexural yielding of gusset plate

$$\phi M_{n2} = (0.9) F_y \frac{t d_2^2}{6} = (0.9)(50 \text{ ksi}) \frac{(3/8 \text{ in.})(10 \text{ in.})^2}{6} = 281 \text{ kip} \cdot \text{in.}$$

$M_{u2} = P_2 e_1 = (26.8 \text{ kips})(8.25 \text{ in.}) = 221 \text{ kip} \cdot \text{in.}$

$281 \text{ kip-in.} > 221 \text{ kip-in.}$ **o.k.**

Lateral-torsional buckling of gusset plate leg

L=13.25 in. for compression loads and 8.25 in. for tension loads. Use L=13.25 in.

$$\phi M_{cr2} = (0.9)(16{,}848) \frac{d_2 t^3}{L_2} = (0.9)(16{,}848) \frac{(10 \text{ in.})(3/8 \text{ in.})^3}{13.25 \text{ in.}} = 603 \text{ kip} \cdot \text{in.}$$

$603 \text{ kip-in.} > 221 \text{ kip-in.}$ **o.k.**

Bolt shear fracture

The gage in the outstanding leg of the clip angle is 2 in.; therefore, the bolts are subject to an eccentricity of 2 in. From Manual Table 7-1, the available shear strength for each bolt, $\phi r_n = 15.9 \text{ kips/bolt}$.

From Manual Table 7-7 with Angle = 0°, S = 3 in., n = 3, and e_x = 2 in.: C = 2.23.

$$\phi R_{nv} = (2.23 \text{ bolts})(15.9 \text{ kips/bolt}) = 35.5 \text{ kips} > 26.8 \text{ kips} \quad \textbf{o.k.}$$

Shear yielding at clip angle

$$\phi R_n = (1.0)(0.6)(3/8 \text{ in.})(9 \text{ in.})(36 \text{ ksi}) = 72.9 \text{ kips} > 26.8 \text{ kips} \quad \textbf{o.k.}$$

Shear fracture at clip angle

$$\phi R_n = (0.75)(0.6)(3/8 \text{ in.})[9 \text{ in.} - (3)(7/8 \text{ in.})](58 \text{ ksi}) = 62.4 \text{ kips} > 26.8 \text{ kips} \quad \textbf{o.k.}$$

Weld clip angle to gusset plate

Manual Table 8-8 with the load angle equal zero

$$kl = 3\tfrac{1}{2} \text{ in.} - 1/2 \text{ in.} = 3.00 \text{ in.}$$

$$l = 9 \text{ in.}$$

$$k = \frac{kl}{l} = \frac{3.00 \text{ in.}}{9 \text{ in.}} = 0.333$$

x = 0.0670 Interpolated from Table 8-8

$$xl = (0.0670)(9 \text{ in.}) = 0.603 \text{ in.}$$

$$al = 3\tfrac{1}{2} \text{ in.} - 0.603 \text{ in.} = 2.90 \text{ in.}$$

$$a = \frac{al}{l} = \frac{2.90 \text{ in.}}{9 \text{ in.}} = 0.322$$

Interpolating from Table 8-8

	k	0.3	0.333	0.4
a				
0.300		2.79	2.94	3.23
0.322			2.86	
0.400		2.45	2.58	2.84

$$D_{req,d} = \frac{P_u}{\phi C C_1 l} = \frac{26.8 \text{ kips}}{(0.75)(2.86)(1.0)(9 \text{ in.})} = 1.39 \text{ sixteenths} < 3 \quad \textbf{o.k.}$$

Block shear of gusset plate around perimeter of weld

$$t_{req,d} = 3.09\frac{D_{req,d}}{F_u} = (3.09)\left(\frac{1.39}{65}\right) = 0.0661 < 3/8 \text{ in.} \quad \textbf{o.k.}$$

Bolt bearing at clip angle

For bolt bearing, the effect of eccentricity must be accounted for; therefore, the effective number of bolts, C = 2.23, will be used from Manual Table 7-7. The edge distance is 1½ in. The bolt group is loaded eccentrically, and each bolt has a skewed resultant load, with at least a small component perpendicular to the edge. It is conservative to assume the 1½ in. edge distance applies to all bolts in the bearing calculation.

$$L_c = 1\tfrac{1}{2} \text{ in.} - (1/2)(13/16 \text{ in.}) = 1.09 \text{ in.}$$

Between the bolts,

$$L_c = 3 \text{ in.} - 13/16 \text{ in.} = 2.19 \text{ in.}$$

$1.09 \text{ in.} < 2.19 \text{ in.}$; use $L_c = 1.09 \text{ in.}$

$$\phi R_n = (0.75)(1.2)(1.09 \text{ in.})(3/8 \text{ in.})(58 \text{ ksi}) \le (0.75)(2.4)(3/4 \text{ in.})(3/8 \text{ in.})(58 \text{ ksi})$$

$\phi R_n = 21.3 \text{ kips / bolt} \le 29.4 \text{ kips / bolt}$. Use $\phi R_n = 21.4 \text{ kips / bolt}$

$$\phi R_n = (21.4 \text{ kips / bolt})(2.23 \text{ bolts}) = 47.7 \text{ kips} > 26.8 \text{ kips} \quad \textbf{o.k.}$$

Bolt bearing at beam web

For bolt bearing, the effect of eccentricity must be accounted for; therefore, the effective number of bolts, C = 2.23, will be used from Manual Table 7-7.

$$\phi R_n = (0.75)(2.4)(3/4\,\text{in.})(0.300\text{ in.})(65\text{ ksi}) = 26.3\text{ kips}$$

$$\phi R_n = (26.3\text{ kips / bolt})(2.23\text{ bolts}) = 58.6\text{ kips} > 26.8\text{ kips} \qquad \textbf{o.k.}$$

Flexural yielding of clip angle at outstanding leg

$$Z = \frac{(3/8\text{ in.})(9\text{ in.})^2}{4} = 7.59\text{ in.}^3$$

$$\phi M_n = F_y Z = (50\text{ ksi})(7.59\text{ in.}^3) = 380\text{ kip-in.}$$

$$M_u = (26.8\text{ kips})(2\text{ in.}) = 53.6\text{ kip-in.} < 380\text{ kip-in.} \qquad \textbf{o.k.}$$

Flexural fracture of clip angle at outstanding leg

$$Z_{net} = (2)\left[(1.50\text{ in.})(2.12\text{ in.})(3/8\text{ in.}) + (3.97\text{ in.})(1.06\text{ in.})(3/8\text{ in.})\right] = 5.54\text{ in.}^3$$

$$\phi M_n = F_u Z_{net} = (65\text{ ksi})(5.54\text{ in.}^3) = 360\text{ kip-in.}$$

$$M_u = (26.8\text{ kips})(2\text{ in.}) = 53.6\text{ kip-in.} < 360\text{ kip-in.} \qquad \textbf{o.k.}$$

Block shear of clip angle

$$A_{gv} = (7\tfrac{1}{2}\text{ in.})(3/8\text{ in.}) = 2.81\text{ in}^2$$

$$A_{nv} = \left[7\tfrac{1}{2}\text{ in.} - (2.5\text{ holes})(7/8\text{ in.})\right](3/8\text{ in.}) = 1.99\text{ in}^2$$

$$A_{nt} = \left[1\tfrac{1}{2}\text{ in.} - (1/2\text{ hole})(7/8\text{ in.})\right](3/8\text{ in.}) = 0.398\text{ in}^2$$

$$U_{bs} = 1.0 \qquad \phi = 0.75$$

$$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt} \le 0.6F_y A_{gv} + U_{bs} F_u A_{nt}$$

$$0.6F_u A_{nv} + U_{bs} F_u A_{nt} = (0.6)(58\text{ ksi})(1.99\text{ in.}^2) + (1.0)(58\text{ ksi})(0.398\text{ in.}^2) = 92.3\text{ kips}$$

$$0.6F_y A_{gv} + U_{bs} F_u A_{nt} = (0.6)(36\text{ ksi})(2.81\text{ in.}^2) + (1.0)(58\text{ ksi})(0.398\text{ in.}^2) = 83.8\text{ kips}$$

$$\phi R_n = (0.75)(83.8 \text{ kips}) = 62.8 \text{ kips} > 26.8 \text{ kips} \quad \textbf{o.k.}$$

Discussion

The method of sections was used to calculate the stresses at critical locations in the gusset plate. The selection of the critical sections is at the discretion of the engineer and is based on judgment and experience, and in many cases, the critical cross section will not be at the same location as for this example. One example of this occurs when the outside corner of the gusset plate is cut on a diagonal. In this case, the bending capacity of the gusset must be checked along the diagonal. The inside corner can be cut as required to provide adequate bending resistance.

Example 2

The strength of the gusset plate in Figure 16 will be checked. The connection is identical to the one in Example 2, except the cutout has a diagonal cut and the load has been increased to 50 kips. Only the limit states associated with the strength of the gusset plate legs will be checked.

Axial force in brace: 50 kips tension or compression
Gusset plate thickness: ⅜ in.
Plate material: A572 Grade 50

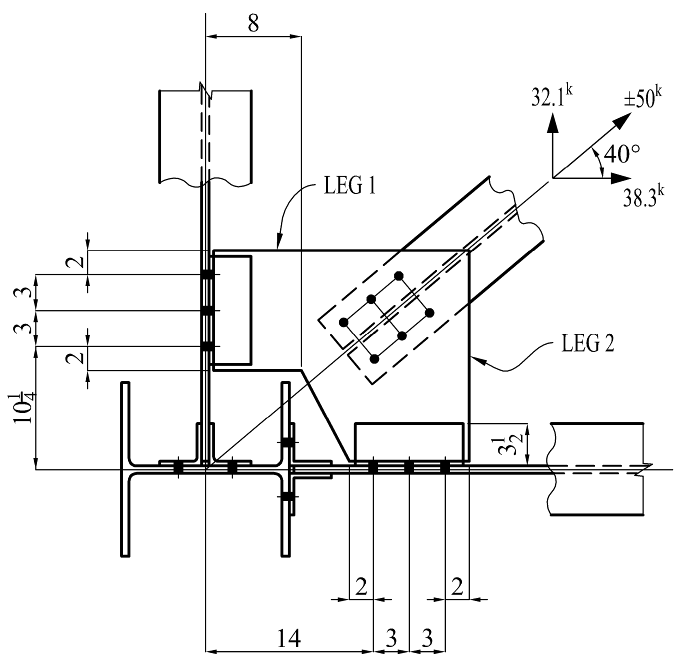


Figure. E2-1. Connection for Example 2.

<u>Shear in Leg 1</u>

$$\phi V_{n1} = (1.0)(0.6)F_y d_1 t = (1.0)(0.6)(50\text{ksi})(10\text{in.})(0.375\text{in.}) = 113 \text{ kips}$$

$113 \text{ kips} > 32.1 \text{ kips}$ **o.k.**

Shear in Leg 2

Shear capacity will be analyzed at a plane immediately beyond the clip angle leg. The leg width at this location is,

$$d_2 = 10\text{in.} + (4\text{in.})(3.5\text{in.}/8.25\text{in.}) = 11.7\text{in.}$$

$$\phi V_{n2} = (1.0)(0.6)F_y d_2 t = (1.0)(0.6)(50\text{ksi})(11.7\text{in.})(0.375\text{in.}) = 132 \text{ kips}$$

$132 \text{ kips} > 38.3 \text{ kips}$ **o.k.**

Bending in Leg 1

$$\phi M_{n1} = (0.9)F_y \frac{td_1^2}{6} = (0.9)(50\text{ksi})\frac{(0.375\text{in.})(10\text{in.})^2}{6} = 281 \text{ kip}\cdot\text{in.}$$

$$M_{u1} = P_1 e_2 = (32.1\text{kips})(8 \text{ in.}) = 257 \text{ kip}\cdot\text{in.}$$

$281 \text{ kip}\cdot\text{in.} > 257 \text{ kip}\cdot\text{in.}$ **o.k.**

Bending in Leg 2 (Case 1: Immediately beyond the clip angle leg)

$$d_2 = 10\text{in.} + (4\text{in.})(3.5\text{in.}/8.25\text{in.}) = 11.7\text{in.}$$

$$\phi M_{n2} = (0.9)F_y \frac{td_2^2}{6} = (0.9)(50\text{ksi})\frac{(0.375\text{in.})(11.7\text{in.})^2}{6} = 385 \text{ kip}\cdot\text{in.}$$

$$M_{u2} = P_2 e_1 = (38.3\text{kips})(3.5\text{in.}) = 134 \text{ kip}\cdot\text{in.}$$

$385 \text{ kip}\cdot\text{in.} > 134 \text{ kip}\cdot\text{in.}$ **o.k.**

Bending in Leg 2 (Case 2: End of diagonal cut)

$$\phi M_{n2} = (0.9)F_y \frac{td_2^2}{6} = (0.9)(50\text{ksi})\frac{(0.375\text{in.})(14\text{in.})^2}{6} = 551 \text{ kip}\cdot\text{in.}$$

$$M_{u2} = P_2 e_1 = (38.3\text{kips})(8.25\text{in.}) = 316 \text{ kip}\cdot\text{in.}$$

$551 \text{ kip}\cdot\text{in.} > 316 \text{ kip}\cdot\text{in.}$ **o.k.**

Buckling at Leg 1

L=17 in. for compression loads and 8 *in.* for tension loads. Use L=17 in.

$$\phi M_{cr1} = (0.9)(16{,}848)\frac{d_1 t^3}{L_1} = (0.9)(16{,}848)\frac{(10\text{ in.})(0.375\text{ in.})^3}{17\text{ in.}} = 470\text{ kip}\cdot\text{in.}$$

$470 \text{ kip}\cdot\text{in.} > 257 \text{ kip}\cdot\text{in.}$ **o.k.**

Buckling at Leg 2

L=13.25 in. for compression loads and 0 in. for tension loads. Use L=13.25 in.

$$\phi M_{cr2} = (0.9)(16{,}848)\frac{d_2 t^3}{L_2} = (0.9)(16{,}848)\frac{(10\text{ in.})(0.375\text{ in.})^3}{13.25\text{ in.}} = 604\text{ kip}\cdot\text{in.}$$

$604 \text{ kip}\cdot\text{in.} > 134 \text{ kip}\cdot\text{in.}$ **o.k.**

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