Design of Wrap-Around Gusset Plates

BO DOWSWELL, FOUAD FOUAD, JAMES DAVIDSON and ROBERT WHYTE

ABSTRACT

This paper addresses the design of wrap-around gusset plates, which are commonly used where a horizontal brace connects at a beam-to-column intersection. Wrap-around gusset plates must be cut around the column, which can lead to high flexural stresses near the reentrant corner.

A rational design method is developed in this paper, based on the results of 15 experimental tests and the corresponding finite element models. The design method, which models the gusset legs as rectangular beams, considers the strength of each leg independently. Because the buckled shapes included both out-of-plane translation and twisting along the leg, the stability behavior is evaluated using a lateral-torsional buckling model according to AISC *Specification* Section F11. Based on the buckled shape of the specimens and finite element models, design buckling lengths are recommended. Information is also included for calculating the strength of wrap-around gusset plates with the interior corner cut on a diagonal. Three examples are provided to illustrate the proposed design procedure.

Keywords: wrap-around gusset plate, horizontal braced, design buckling length.

INTRODUCTION

In most commercial buildings, floor and roof diaphragms are used to distribute loads in the horizontal plane of the structure to the lateral load-resisting system. Due to the open nature of most industrial structures, diaphragms are not present, and horizontal bracing is often used to distribute the loads in the horizontal plane. Horizontal bracing is also used in commercial structures, where a diaphragm is not present or where the strength or stiffness of the diaphragm is not adequate. A simple horizontal bracing system is shown in Figure 1.

Connection Details

Figure 2a shows a typical horizontal bracing connection at a beam-to-beam intersection. Where a horizontal brace connects at a beam-to-column intersection, the gusset plate is typically cut around the column as shown in Figure 2b. At locations with large columns and beam clip angles, as shown in Figure 2c, a significant portion of the gusset plate must be cut out. The optional diagonal cut shown at the inside corner of the plate can increase the bending strength of the gusset plate. The optional cut at the outside corner is used to ensure compliance with the maximum edge distance requirements in AISC *Specification* (AISC, 2010) Section J3.5.

Objective

A consistent design method for wrap-around gusset plates has not been established. The objective of this paper is to develop a rational design method, based on the results of 15 experimental tests and the corresponding finite element models (Dowswell, 2005). The tests and finite element models were also documented by Dowswell and Fouad (2006), Dowswell et al. (2006) and Dowswell and Fouad (2007). This paper is focused on design information; therefore, only a brief discussion of the pertinent research data will be presented. Three examples are provided to illustrate the proposed design procedure.

LOAD DISTRIBUTION

Due to the geometry of wrap-around gusset plates, shear and flexural stresses develop in the plate that must be accounted for in design. Because the force distribution in horizontal brace connections is indeterminate, simplifying assumptions are required for connection design. The load distribution can be determined using a stiffness analysis, but modeling the entire connection using finite elements is not practical for design. Localized yielding in bracing connections begins in the early stages of loading and the load distribution changes

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Paper No. 2016-02

as the brace force increases. Further complicating the analysis are the effects of frame action, foundation settlements, thermal expansion, erection stresses, etc. Therefore, the loads used in connection design should be based on the ultimate strength condition.

Interface Loads

The lower-bound theorem of limit analysis states that a load calculated based on an assumed load distribution that satisfies equilibrium conditions with forces nowhere exceeding the capacity will be less than or equal to the true limit load. Using the lower-bound theorem for a given connection, the strength calculated using the load distribution that results in the highest strength is closest to the actual strength. In practical terms, this means that engineers can choose any load distribution that is convenient if the following conditions are satisfied:

1. All framing members and connection elements must be in equilibrium.
2. All elements in the load path must have adequate strength to resist the assumed loads.
3. All elements in the joint must have adequate ductility to allow the loads to redistribute so the assumed distribution can be achieved.

The assumed load distribution must be consistent throughout the connection and member design. If the aforementioned conditions are satisfied, the loads can be distributed in any convenient manner. Because this is critical to the economy of the connection, selecting a reasonable load distribution often requires significant experience and judgment.

For the connection in Figure 2b, Figure 3 shows two potential methods to distribute the axial load in the brace to the gusset-to-beam interfaces. Using a deformation compatibility approach, the interface forces shown in Figure 3a are incorrect. This is because the gusset-to-beam interfaces are typically much stiffer when loaded in the plane of the web relative to out-of-plane loading. The out-of-plane flexibility can be caused by weak-axis bending of the beams, local bending of the web, and bending (prying) of the clip angles.

If the forces perpendicular to the web at both interfaces are deemed negligible, the resulting interface forces are shown in Figure 3b. The load distribution in Figure 3b uses the simplest and most direct path to get the components of the brace force directly into the beams, and it is considered standard practice for these connections.

Even if deformation compatibility were not an issue, there are several negative consequences for selecting the load distribution in Figure 3a:

1. The connection angles and beam webs must be designed for the forces perpendicular to the webs, $F_E/2$ or $F_N/2$. This will require thicker connection angles and will likely require stiffeners in the beam webs.
2. The beams may be overstressed because they are not typically designed for weak-axis bending and weak-axis shear.
3. In addition to any transfer force and beam end reaction, the beam-to-column connections would be subjected to a horizontal shear force.

Internal Loads

The load distribution in wrap-around gusset plate connections is shown in Figure 4. The plate legs are subject to flexure, and each leg is modeled independently as a rectangular beam with a linearly varying moment diagram. The

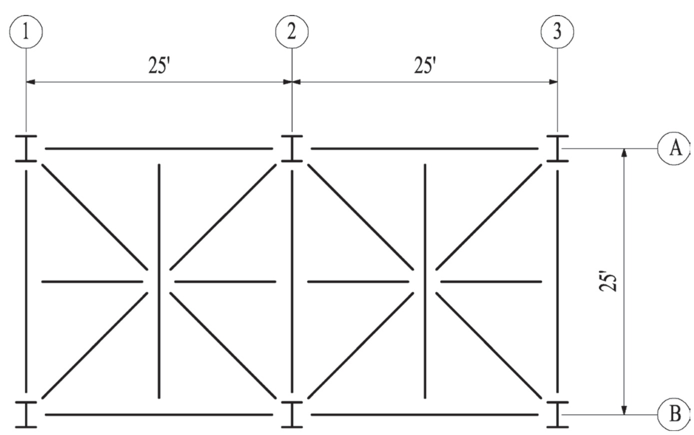


Fig. 1. Plan view of a horizontal bracing system.

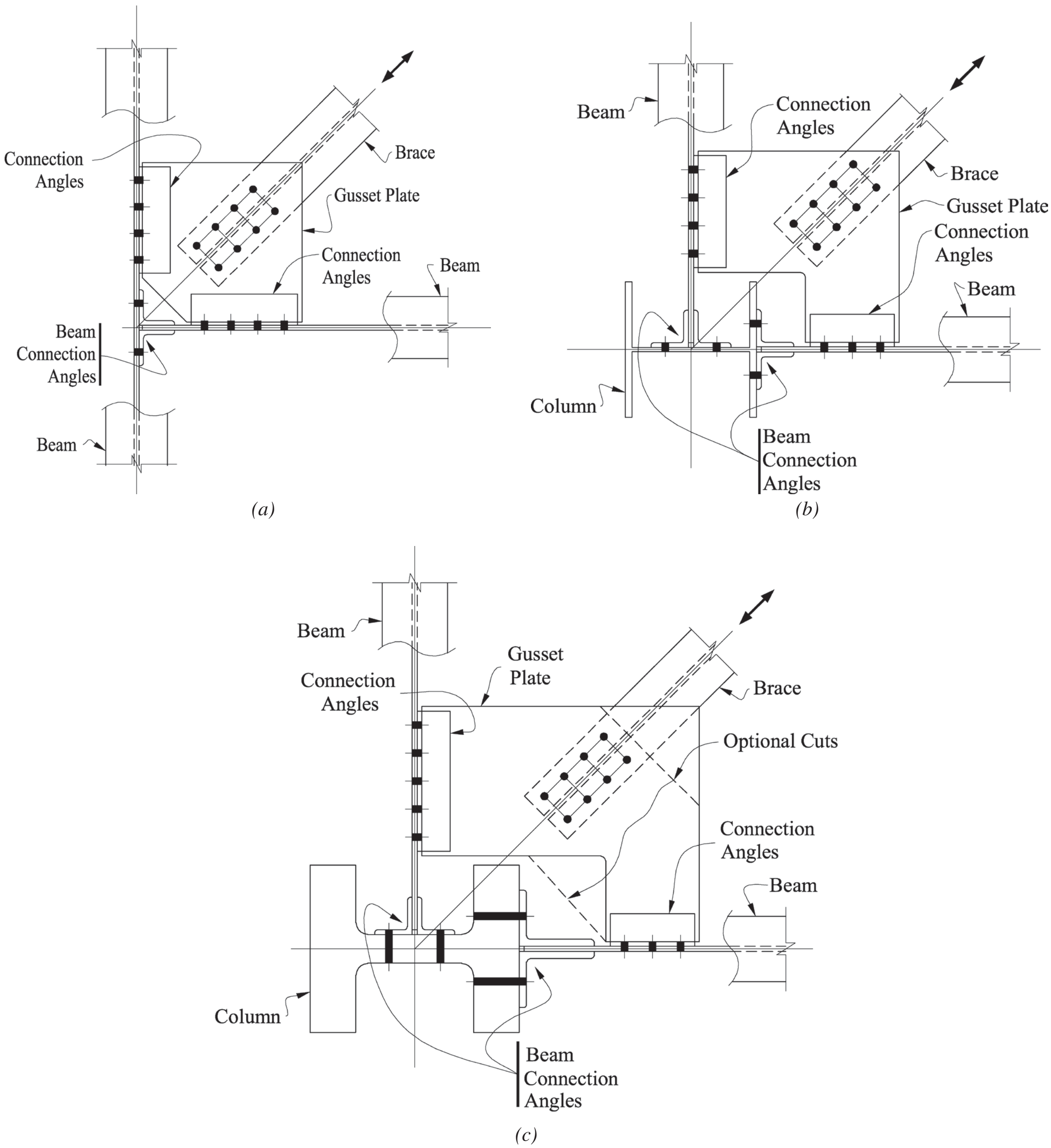


Fig. 2. Horizontal brace gusset plates: (a) beam-to-beam interface; (b) beam-to-column interface with wrap-around gusset plate; (c) wrap-around gusset plate with a large cutout and optional diagonal cuts.

maximum moments are located at the interior face of the adjacent leg. Equations 1a and 1b, which may be conservative for some gusset plate geometries, can be used to calculate the required moments at the critical section of each leg.

For leg 1

$$M_{r1} = P_1 e_2 \tag{1a}$$

For leg 2

$$M_{r2} = P_2 e_1 \tag{1b}$$

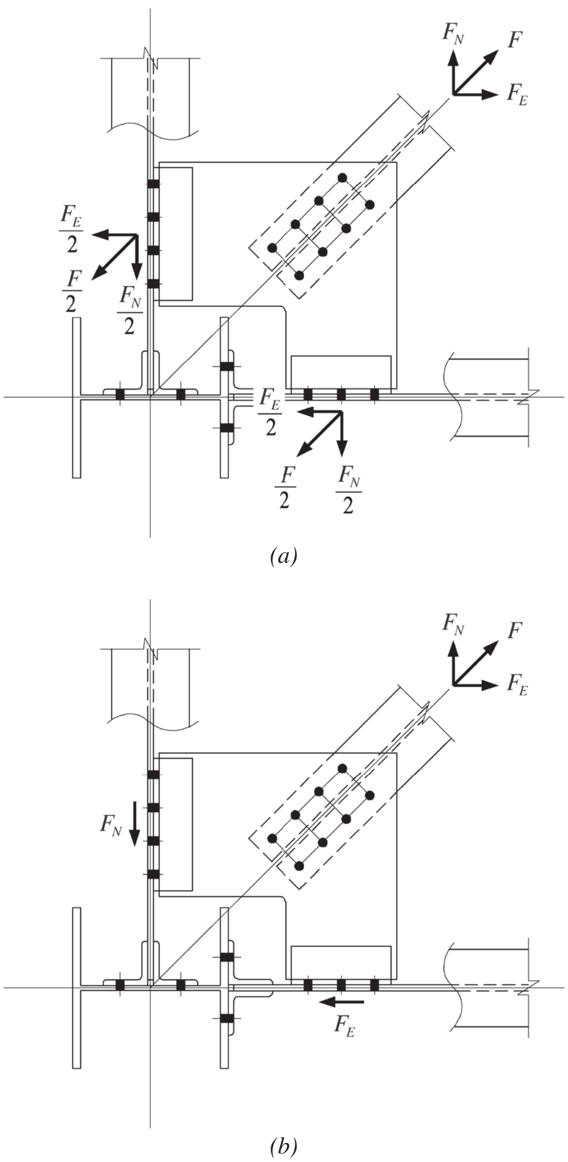


Fig. 3. Interface loads at a horizontal brace connection: (a) inefficient; (b) efficient.

where

P_1 = component of the brace load, P, at leg 1, kips
P_2 = components of the brace load, P, at leg 2, kips
e_1 = cutout dimension perpendicular to leg 1, in.
e_2 = cutout dimension perpendicular to leg 2, in.

FLEXURAL STRENGTH

Each leg of the plate is subject to limit states common to flexural members; therefore, the legs are modeled independently, as separate beams. In addition to the flexural strength, including lateral-torsional buckling, the shear strength of each leg should be considered in the design.

Stress

Each leg of the gusset plate must resist the moments generated by the load system in Figure 4. This results in maximum bending stresses at the reentrant corner where the two legs meet. Figure 5 shows the stress contour plots for a finite element model loaded in tension. Figure 5a shows maximum von Mises stresses at the reentrant corner. The normal stresses in the *x*- and *y*-directions are shown in Figures 5b and 5c, respectively. As expected, the maximum stresses are at the edges of each leg and at the intersection of the two legs—where the maximum moments occur. Because the brace load was in tension, the flexural stresses are compressive at the inner edges and tensile at the outer edges. 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.

The stress patterns from 15 finite element models and the strain gage data from the corresponding experimental specimens verified the accuracy of the load system in

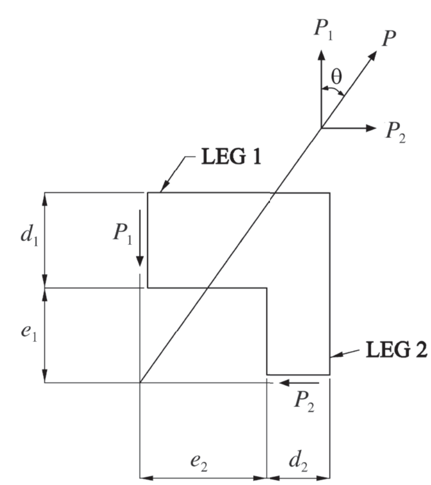


Fig. 4. Internal loads for wrap-around gusset plates.

Figure 4. At the failure load, the strain gage data and finite element models indicated some yielding. Although most of the plates had a substantial amount of material above the proportional limit, none of the plates reached full plasticity before buckling.

Lateral-Torsional Buckling

Although five of the 15 specimens were tested with tensile brace loads, the gusset leg flexural loads caused buckling in all of the experimental specimens and finite element models as shown in Figures 6 and 7, respectively. Figure 6a shows the permanent deformation after testing for specimen 2T, which was tested in tension. Figure 6b shows the permanent deformation after testing for specimen 7C, which was tested in compression. For both specimens, the maximum out-of-plane translation corresponds to the location of maximum compressive stress. The buckled shapes included both out-of-plane translation and twisting along the leg.

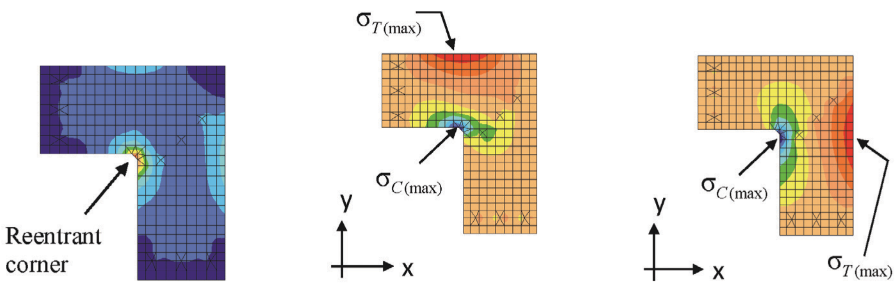


Fig. 5. Elastic stress contours for a typical model loaded in tension: (a) von Mises stresses; (b) x-x normal stresses; (c) y-y normal stresses.

(a)

(b)

Fig. 6. Specimens 2T and 7C after testing: (a) specimen 2T; (b) specimen 7C.

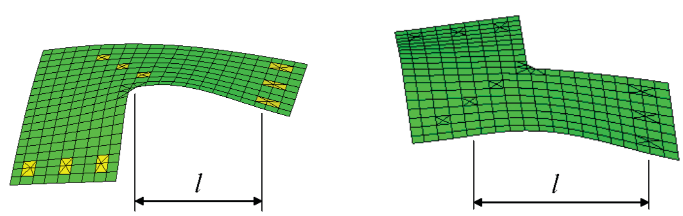
Effect of Diagonal Cut at the Interior Corner

Wrap-around gusset plates can have the interior corner cut on a diagonal as shown in Figure 2c. To determine the effect of the diagonal cut, test specimens 8 and 10, shown in Figure 8, were fabricated with identical geometry, except for the diagonal cut on specimen 10. The tests and finite element models showed a strength increase of about 20% due to the diagonal cut.

For each leg, the critical section could be located at either end of the diagonal cut; therefore, the flexural strength at both sections should be considered. Figures 9a and 9b show the normal stress contours for specimen 10 in the *x*- and *y*-directions, respectively. Because the gusset plate is loaded in tension, the interior edges of the cutout are subjected to compression stresses. The maximum *x*-direction stresses are located at the reentrant corner farthest from the beam center at leg 1. In this case, the critical section is subjected to a moment equal to P_1e_2. The maximum *y*-direction stresses are located at the reentrant corner closest to the beam center at leg 2. In this case, the critical section is subjected to a moment equal to P_2e_{r1}. These results show the importance of calculating the flexural strength for each leg at both cross-sections located at the reentrant corners of the diagonal cut.

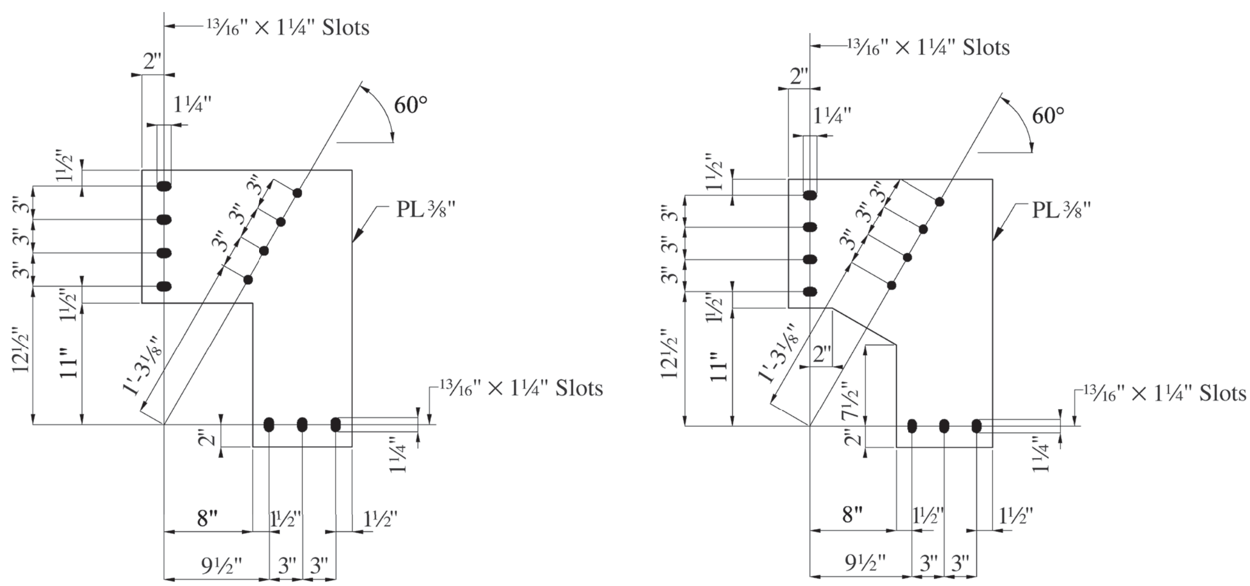
DESIGN

The experimental and finite element results validate the practice of treating each leg of the gusset plate as a rectangular beam. Because the buckled shapes included both out-of-plane translation and twisting along the leg, the stability behavior can be evaluated using a lateral-torsional buckling model. Dowswell (2016) verified that AISC *Specification* Section F11 can be used for designing connection elements subjected to lateral-torsional buckling. Dowswell and Whyte (2014) suggested this approach for calculating the local flexural strength of double-coped beams, which had buckled shapes that are similar to those of wrap-around gusset plate



(a) (b)

Fig. 7. Buckled shapes of finite element models: (a) tension load; (b) compression load.



(a) (b)

Fig. 8. Details of specimens 8 and 10: (a) specimen 8; (b) specimen 10.

legs. The shear strength of each leg is calculated according to AISC *Specification* Section J4.2.

AISC *Specification* Section F11

AISC *Specification* (AISC, 2010) Section F11 defines the flexural strength of rectangular members. The nominal strength is the lower value obtained according to the limit states of yielding and lateral-torsional buckling.

For $\frac{L_b d}{t^2} \le \frac{0.08E}{F_y}$, yielding is the controlling limit state.

The nominal strength is:

$$M_n = M_p = F_y Z \le 1.6 M_y \qquad (Spec.\ \text{Eq. F11-1})$$

For $\frac{0.08E}{F_y} < \frac{L_b d}{t^2} \le \frac{1.9E}{F_y}$, inelastic lateral-torsional buckling is the controlling limit state.

$$M_n = C_b\left[1.52 - 0.274\left(\frac{L_b d}{t^2}\right)\frac{F_y}{E}\right]M_y \le M_p \qquad (Spec.\ \text{Eq. F11-2})$$

For $\frac{L_b d}{t^2} > \frac{1.9E}{F_y}$, elastic lateral-torsional buckling is the controlling limit state.

$$M_n = F_{cr} S_x \le M_p \qquad (Spec.\ \text{Eq. F11-3})$$

where

$$F_{cr} = \frac{1.9EC_b}{\frac{L_b d}{t^2}} \qquad (Spec.\ \text{Eq. F11-4})$$

C_b = lateral-torsional buckling modification factor for nonuniform moment diagrams

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)

F_y = specified minimum yield stress, ksi (MPa)

L_b = distance between braces, in. (mm)

M_p = plastic bending moment, kip-in. (N-mm)

M_y = moment at yielding of the extreme fiber, kip-in. (N-mm)

S_x = elastic section modulus taken about the x-axis, in.^{3} (mm^3)

Z = plastic section modulus about the axis of bending, in.^{3} (mm^3)

d = depth of the gusset plate leg, in. (mm)

t = thickness of the gusset plate, in. (mm)

General Design Method

Because the buckling strength in AISC *Specification* Section F11 is dependent on both L_b and C_b, appropriate values for these variables were developed to provide the best estimate of the true strength and behavior of the specimens. To determine the buckling length, L_b, the buckled shape of the specimens and finite element models was observed. A general trend was observed, where the buckling length of the tension specimens was limited to the cutout dimension and the buckling length of the compression specimens extended approximately to the mid-depth of the adjacent leg. The following buckling lengths are applicable when the plate is loaded in tension: $L_{b1} = e_2$ for leg 1, and $L_{b2} = e_1$ for leg 2. For plates loaded in compression, the following buckling lengths

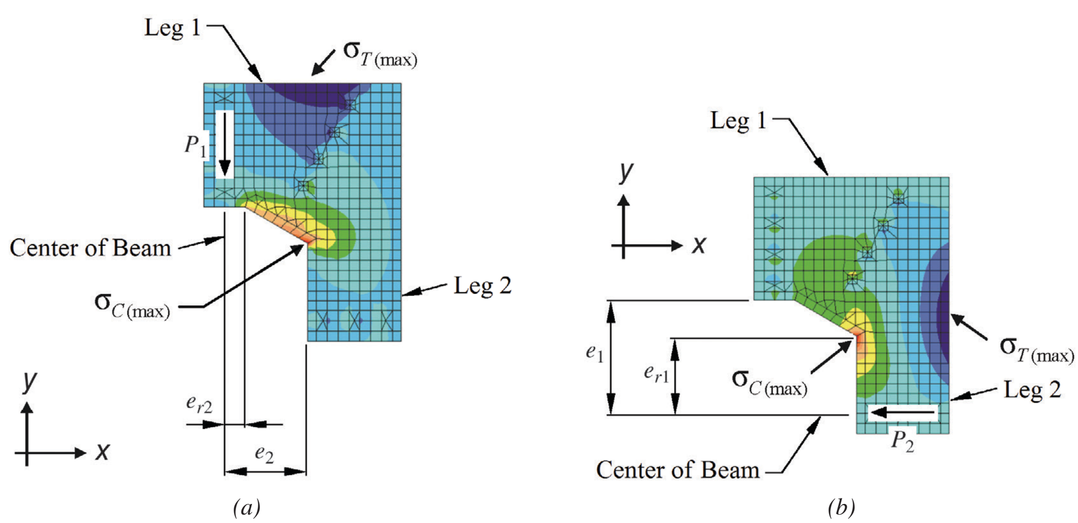


Fig. 9. Elastic stress contours for specimen 10, which was loaded in tension: (a) x-direction normal stresses; (b) y-direction normal stresses.

are applicable: $L_{b1} = e_2 + d_2/2$ for leg 1, and $L_{b2} = e_1 + d_1/2$ for leg 2. As shown in Figure 4, d_1 and d_2 are the depths of legs 1 and 2, respectively.

According to equations developed by Dowswell (2004), $C_b = 1.84$ for a rectangular cantilever beam loaded at the shear center with bracing at each end and a concentrated load at the tip. However, the experimental and finite element results show that the legs can be assumed fully braced at both ends only under certain conditions. For gusset plates carrying tensile brace loads, $C_b = 1.84$.

For many gusset plates carrying compressive brace loads, both legs will not reach their critical load simultaneously. Therefore, the noncritical leg can provide restraint to the critical leg, and $C_b = 1.84$ provides accurate results. Conversely, when the critical load ratio of both legs are similar, full bracing cannot be assumed, and $C_b = 1.00$ is more accurate. For gusset plates carrying compressive brace loads, C_b, can be determined based on the critical load ratio, α, which was developed by Dowswell (2012).

For gusset plates with $\alpha > 1.6$ or $\alpha < 1/1.6$, $C_b = 1.84$

For gusset plates with $1/1.6 \le \alpha \le 1.6$, $C_b = 1.00$

where

$$\alpha = \frac{(M_{cr}/M_r)_2}{(M_{cr}/M_r)_1} = \frac{d_1 L_{b2} P_2 e_1}{d_2 L_{b1} P_1 e_2} = \frac{d_1 L_{b2} e_1}{d_2 L_{b1} e_2} \tan\theta \tag{2}$$

$(M_{cr}/M_r)_1$ = critical moment to required moment ratio at leg 1

$(M_{cr}/M_r)_2$ = critical moment to required moment ratio at leg 2

M_{cr} = $F_{cr}S_x$

θ = angle between the brace and the beam connected to leg 1 (Figure 4)

For plates with a diagonal cut, L_b and C_b are calculated the same as plates with square cutouts. Plate leg depths, d_1 and d_2, are the dimensions between the parallel edges, without consideration of the diagonal cut. For buckling calculations, d_1 and d_2 are used to calculate the sectional properties. The required moments for buckling at the legs can be calculated using Equations 1a and 1b with e_{m1} and e_{m2} substituted for e_1 and e_2, respectively. The variables e_{m1} and e_{m2} are the distances along the adjacent leg, from the work point to the midpoint of the diagonal cut, as shown in Figure 10. The flexural yielding strength, using the available plastic moment, should also be checked at the reentrant corners of the diagonal cut.

The strength of each specimen was calculated using the general design method, with the results summarized in the second column of Table 1. All of the loads are expressed as the brace load based on the minimum strength of the two legs. All loads were calculated using the actual plate thickness and yield strength measured before testing. The predicted failure modes are listed in the third column. The specimen numbers are suffixed with "T" if the plate was loaded in tension and "C" if it was loaded in compression.

The experimental loads are listed in the fourth and fifth columns of Table 1. P_{ep} is the experimental load at the proportional limit, determined using a line offset ¹⁄₃₂ in. from the linear portion of the experimental curve. P_{eu} is the maximum experimental load. The experimental failure modes are listed in the sixth column. The experimental-to-calculated strength ratios, P_{ep}/P_c and P_{eu}/P_c, are listed in the seventh and eighth columns of Table 1, respectively. P_{ep}/P_c varies from 0.786 to 1.89, with an average of 1.15 and a standard deviation of 0.302. P_{eu}/P_c varies from 0.880 to 2.01, with an average of 1.40 and a standard deviation of 0.312.

Simplified Design Method

To simplify the design process, $C_b = 1.00$ can be used for all gusset plates with compressive brace loads. Using $C_b = 1.84$ for gusset plates with tensile brace loads, four of the five experimental specimens were predicted to fail at the plastic strength. These four specimens were ⅜-in. thick. Specimen 6T, which had a predicted failure mode of elastic buckling, was only ¼-in. thick. Because gusset plates in steel structures are usually at least ⅜-in. thick, these gusset plates can be designed assuming the legs are fully braced against lateral-torsional buckling.

The strength of each specimen was calculated using the simplified design method, with the results summarized in the second column of Table 2. All of the loads are expressed as the brace load based on the minimum strength of the

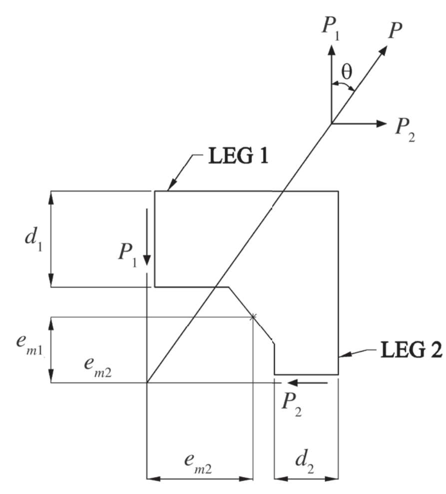


Fig. 10. Dimensions for wrap-around gusset plates with a diagonal cut.

Table 1. Calculated and Experimental Loads (General Design Method)							
Specimen No.	**P_c, kips**	**Predicted Failure Mode**	**P_{ep}, kips**	**P_{eu}, kips**	**Experimental Failure Mode**	**$\frac{P_{ep}}{P_c}$**	**$\frac{P_{eu}}{P_c}$**
2T	49.7	P	69.0	89.9	I	1.39	1.81
6T	45.1	E	42.3	53.6	I	0.938	1.19
8T	69.0	P	85.3	91.2	I	1.24	1.32
9T	53.6	P	51.5	63.6	I	0.961	1.19
10T	82.7	P	96.2	110.0	I	1.16	1.33
1C	39.5	I	33.3	45.8	I	0.843	1.16
2C	49.3	I	47.3	63.9	I	0.960	1.30
3C	41.9	I	46.6	64.2	I	1.11	1.53
4C	16.9	E	32.0	32.0	E	1.89	1.89
5C	23.3	E	28.7	46.8	I	1.23	2.01
6C	15.7	E	25.3	25.3	E	1.61	1.61
7C	52.9	I	46.4	46.5	I	0.877	0.880
8C	48.9	I	38.4	60.8	I	0.786	1.24
9C	37.3	I	44.4	51.5	I	1.19	1.38
10C	57.0	I	57.0	66.5	I	1.00	1.17

P_c = calculated strength, kips
P_{ep} = experimental load at proportional limit (1⁄32 in. offset), kips
P_{eu} = maximum experimental load, kips
E: Elastic buckling
I: Inelastic buckling
P: Plastic flexural strength

Table 2. Calculated and Experimental Loads (Simplified Design Method)							
Specimen No.	**P_c, kips**	**Predicted Failure Mode**	**P_{ep}, kips**	**P_{eu}, kips**	**Experimental Failure Mode**	**$\frac{P_{ep}}{P_c}$**	**$\frac{P_{eu}}{P_c}$**
2T	49.7	P	69.0	89.9	I	1.39	1.81
6T	47.7	P	42.3	53.6	I	0.887	1.12
8T	69.0	P	85.3	91.2	I	1.24	1.32
9T	53.6	P	51.5	63.6	I	0.960	1.19
10T	82.7	P	96.2	110.0	I	1.16	1.33
1C	28.4	I	33.3	45.8	I	1.17	1.61
2C	35.9	I	47.3	63.9	I	1.32	1.78
3C	41.9	I	46.6	64.2	I	1.11	1.53
4C	9.2	E	32.0	32.0	E	3.48	3.48
5C	12.6	E	28.7	46.8	I	2.28	3.71
6C	15.7	E	25.3	25.3	E	1.61	1.61
7C	52.9	I	46.4	46.5	I	0.877	0.880
8C	48.9	I	38.4	60.8	I	0.785	1.24
9C	37.3	I	44.4	51.5	I	1.19	1.38
10C	57.0	I	57.0	66.5	I	1.00	1.17

P_c = calculated strength, kips
P_{ep} = experimental load at proportional limit (1⁄32 in. offset), kips
P_{eu} = maximum experimental load, kips
E: Elastic buckling
I: Inelastic buckling
P: Plastic flexural strength

two legs. All loads were calculated using the actual plate thickness and yield strength measured before testing. The predicted failure modes are listed in the third column. The experimental loads are listed in the fourth and fifth columns, and the experimental failure modes are listed in the sixth column. The P_{ep}/P_c and P_{eu}/P_c ratios are listed in the seventh and eighth columns of Table 2, respectively. P_{ep}/P_c varies from 0.786 to 3.48, and P_{eu}/P_c varies from 0.880 to 3.70. The highest experimental-to-nominal load ratios are for cases where the general design method could be used to increase C_b from 1.00 to 1.84.

SUMMARY

A rational design method for wrap-around gusset plates was developed in this paper, based on the results of 15 experimental tests and the corresponding finite element models. The design method, which models the gusset legs as rectangular beams, considers the strength of each leg independently.

Because the buckled shapes included both out-of-plane translation and twisting along the leg, the buckling strength of each leg is evaluated using a lateral-torsional buckling model according to AISC *Specification* Section F11. Appropriate values for L_b and C_b were developed to provide the best estimate of the true strength and behavior of the specimens. Three examples are provided to illustrate the proposed design procedure.

For many practical geometries, the noncritical leg can provide restraint against lateral-torsional buckling of the critical leg. In these cases, including gusset plates carrying tensile brace loads, C_b = 1.84 is recommended for design. Otherwise, C_b = 1.00 is applicable. At the maximum experimental load, the mean experimental-to-calculated strength ratio is 1.40 with a standard deviation of 0.312.

A simplified design method was also discussed, where C_b = 1.00 for all gusset plates carrying compressive brace loads. For gusset plates with tensile brace loads, the limit state of lateral-torsional buckling is neglected, and the flexural strength is calculated using the plastic moment capacity. At the maximum experimental load, the experimental-to-calculated strength ratios varied from 0.880 to 3.70. However, the highest experimental-to-nominal load ratios are for cases where the general design method could be used to increase C_b from 1.00 to 1.84.

EXAMPLE 1—SIMPLIFIED DESIGN METHOD

In this example, the strength the gusset plate shown in Figure 11 is calculated using the simplified design method. The gusset plate is ⅜-in.-thick ASTM A572 Grade 50 material. The LRFD and ASD loads are 50.0 kips tension/30 kips compression and 33.3 kips tension/20 kips compression, respectively.

From AISC *Manual* Table 2-4, the yield strength is F_y = 50 ksi.

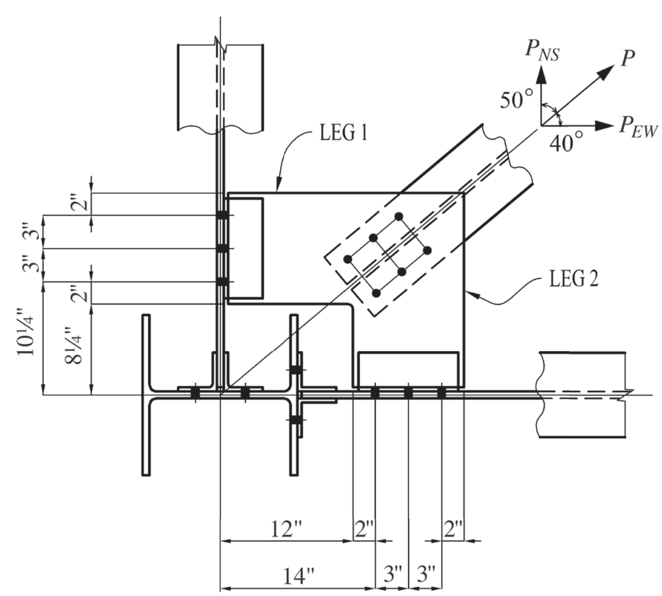


Fig. 11. Wrap-around gusset plate for Example 1.

Tension Brace Load

The brace axial load is:

LRFD	ASD
$P_u = 50.0$ kips (tension)	$P_a = 33.3$ kips (tension)

The north–south and east–west brace components are:

LRFD	ASD
$P_{NS} = (50\text{ kips})\sin 40° = 32.1\text{ kips}$ $P_{EW} = (50\text{ kips})\cos 40° = 38.3\text{ kips}$	$P_{NS} = (33.3\text{ kips})\sin 40° = 21.4\text{ kips}$ $P_{EW} = (33.3\text{ kips})\cos 40° = 25.5\text{ kips}$

Flexural Strength of Leg 1

The required moment at the interior face of the adjacent leg is:

LRFD	ASD
$M_u = P_{NS}e_2$ (from Eq. 1a) $= (32.1\text{ kips})(12.0\text{ in.})$ $= 385\text{ kip-in.}$	$M_a = P_{NS}e_2$ (from Eq. 1a) $= (21.4\text{ kips})(12.0\text{ in.})$ $= 257\text{ kip-in.}$

When the brace is in tension, the legs are assumed to be fully braced; therefore, the nominal flexural strength according to AISC *Specification* Section F11 is:

$$M_n = M_p = F_yZ \qquad (\textit{Spec}.\text{ Eq. F11-1})$$
$$= (50\text{ ksi})\left[\frac{(\tfrac{3}{8}\text{ in.})(10.0\text{ in.})^2}{4}\right]$$
$$= 469\text{ kip-in.}$$

The available flexural strength is:

LRFD	ASD
$\phi M_n = (0.90)(469\text{ kip-in.})$ $= 422\text{ kip-in.} > 385\text{ kip-in.}$ **o.k.**	$\dfrac{M_n}{\Omega} = \dfrac{469\text{ kip-in.}}{1.67}$ $= 281\text{ kip-in.} > 257\text{ kip-in.}$ **o.k.**

Shear Yielding of Leg 1

$$V_n = 0.60F_yA_{gv} \qquad (\text{from } \textit{Spec}.\text{ Eq. J4-3})$$
$$= (0.60)(50\text{ ksi})(10.0\text{ in.})(\tfrac{3}{8}\text{ in.})$$
$$= 113\text{ kips}$$

The available shear strength is:

LRFD	ASD
$\phi V_n = (1.00)(113\text{ kips})$ $= 113\text{ kips} > 32.1\text{ kips}$ **o.k.**	$\dfrac{V_n}{\Omega} = \dfrac{113\text{ kips}}{1.50}$ $= 75.3\text{ kips} > 21.4\text{ kips}$ **o.k.**

Flexural Strength of Leg 2

The required moment at the interior face of the adjacent leg is:

LRFD	ASD
$M_u = P_{EW}e_1$ (from Eq. 1b) $= (38.3 \text{ kips})(8\tfrac{1}{4} \text{ in.})$ $= 316 \text{ kip-in.}$	$M_a = P_{EW}e_1$ (from Eq. 1b) $= (25.5 \text{ kips})(8\tfrac{1}{4} \text{ in.})$ $= 210 \text{ kip-in.}$

When the brace is in tension, the legs are assumed to be fully braced; therefore, the nominal flexural strength according to AISC *Specification* Section F11 is:

$$M_n = M_p = F_yZ \qquad (\textit{Spec}. \text{ Eq. F11-1})$$

$$= (50 \text{ ksi})\left[\frac{(\tfrac{3}{8} \text{ in.})(10.0 \text{ in.})^2}{4}\right]$$

$$= 469 \text{ kip-in.}$$

The available flexural strength is:

LRFD	ASD
$\phi M_n = (0.90)(469 \text{ kip-in.})$ $= 422 \text{ kip-in.} > 316 \text{ kip-in.}$ **o.k.**	$\dfrac{M_n}{\Omega} = \dfrac{469 \text{ kip-in.}}{1.67}$ $= 281 \text{ kip-in.} > 210 \text{ kip-in.}$ **o.k.**

Shear Yielding of Leg 2

$$V_n = 0.60F_yA_{gv} \qquad (\text{from } \textit{Spec}. \text{ Eq. J4-3})$$

$$= (0.60)(50 \text{ ksi})(10.0 \text{ in.})(\tfrac{3}{8} \text{ in.})$$

$$= 113 \text{ kips}$$

The available shear strength is:

LRFD	ASD
$\phi V_n = (1.00)(113 \text{ kips})$ $= 113 \text{ kips} > 38.3 \text{ kips}$ **o.k.**	$\dfrac{V_n}{\Omega} = \dfrac{113 \text{ kips}}{1.50}$ $= 75.3 \text{ kips} > 25.5 \text{ kips}$ **o.k.**

Compression Brace Load

The brace axial load is:

LRFD	ASD
$P_u = 30.0$ kips (compression)	$P_a = 20.0$ kips (compression)

The north–south and east–west brace components are:

LRFD	ASD
$P_{NS} = (30.0 \text{ kips})\sin 40° = 19.3 \text{ kips}$ $P_{EW} = (30.0 \text{ kips})\cos 40° = 23.0 \text{ kips}$	$P_{NS} = (20.0 \text{ kips})\sin 40° = 12.9 \text{ kips}$ $P_{EW} = (20.0 \text{ kips})\cos 40° = 15.3 \text{ kips}$

Flexural Strength of Leg 1

$$S_x = \frac{(3/8 \text{ in.})(10.0 \text{ in.})^2}{6}$$
$$= 6.25 \text{ in.}^3$$

$$M_p = F_y Z \qquad \text{(from \textit{Spec.} Eq. F11-1)}$$
$$= (50 \text{ ksi})\left[\frac{(3/8 \text{ in.})(10.0 \text{ in.})^2}{4}\right]$$
$$= 469 \text{ kip-in.}$$

The required moment at the interior face of the adjacent leg is:

LRFD	ASD
$M_u = P_{NS}e_2$ (from Eq. 1b) $= (19.3 \text{ kips})(12.0 \text{ in.})$ $= 232 \text{ kip-in.}$	$M_a = P_{NS}e_2$ (from Eq. 1b) $= (12.9 \text{ kips})(12.0 \text{ in.})$ $= 155 \text{ kip-in.}$

Using $C_b = 1.00$ and $L_{b1} = e_2 + d_2/2 = 12 \text{ in.} + 5 \text{ in.} = 17.0 \text{ in.}$:

$$\frac{L_{b1}d_1}{t^2} = \frac{(17.0 \text{ in.})(10.0 \text{ in.})}{(3/8 \text{ in.})^2}$$
$$= 1{,}210$$

$$\frac{1.9E}{F_y} = \frac{(1.9)(29{,}000 \text{ ksi})}{(50 \text{ ksi})}$$
$$= 1{,}100$$

Because 1,210 > 1,100, elastic lateral-torsional buckling is the controlling limit state, and the critical stress is:

$$F_{cr} = \frac{1.9EC_b}{\frac{L_{b1}d_1}{t^2}} \qquad \text{(\textit{Spec.} Eq. F11-4)}$$
$$= \frac{(1.9)(29{,}000 \text{ ksi})(1.00)}{1{,}210}$$
$$= 45.5 \text{ ksi}$$

And thus the nominal flexural strength is:

$$M_n = F_{cr}S_x \le M_p \qquad \text{(\textit{Spec.} Eq. F11-3)}$$
$$= (45.5 \text{ ksi})(6.25 \text{ in.}^3) \le 469 \text{ kip-in.}$$
$$= 284 \text{ kip-in.}$$

The available flexural strength is:

LRFD	ASD
$\phi M_n = (0.90)(284 \text{ kip-in.})$ $= 256 \text{ kip-in.} > 232 \text{ kip-in.}$ **o.k.**	$\frac{M_n}{\Omega} = \frac{284 \text{ kip-in.}}{1.67}$ $= 170 \text{ kip-in.} > 155 \text{ kip-in.}$ **o.k.**

Shear Yielding of Leg 1

$$V_n = 0.60F_yA_{gv} \quad \text{(from } \textit{Spec}\text{. Eq. J4-3)}$$
$$= (0.60)(50 \text{ ksi})(10.0 \text{ in.})(3/8 \text{ in.})$$
$$= 113 \text{ kips}$$

The available shear strength is:

LRFD	ASD
$\phi V_n = (1.00)(113 \text{ kips})$ $= 113 \text{ kips} > 19.3 \text{ kips}$ **o.k.**	$\dfrac{V_n}{\Omega} = \dfrac{113 \text{ kips}}{1.50}$ $= 75.3 \text{ kips} > 12.9 \text{ kips}$ **o.k.**

Flexural Strength of Leg 2

$$S_x = \frac{(3/8 \text{ in.})(10.0 \text{ in.})^2}{6}$$
$$= 6.25 \text{ in.}^3$$

$$M_y = F_yS_x$$
$$= (50 \text{ ksi})(6.25 \text{ in.}^3)$$
$$= 313 \text{ kip-in.}$$

$$M_p = F_yZ \quad \text{(from } \textit{Spec}\text{. Eq. F11-1)}$$
$$= (50 \text{ ksi})\left[\frac{(3/8 \text{ in.})(10.0 \text{ in.})^2}{4}\right]$$
$$= 469 \text{ kip-in.}$$

The required moment at the interior face of the adjacent leg is:

LRFD	ASD
$M_u = P_{EW}e_1$ (from Eq. 1b) $= (23.0 \text{ kips})(8\tfrac{1}{4} \text{ in.})$ $= 190 \text{ kip-in.}$	$M_a = P_{EW}e_1$ (from Eq. 1b) $= (15.3 \text{ kips})(8\tfrac{1}{4} \text{ in.})$ $= 126 \text{ kip-in.}$

Using $C_b = 1.00$ and $L_{b2} = e_1 + d_1/2 = 8\tfrac{1}{4}$ in. + 5 in. = $13\tfrac{1}{4}$ in.:

$$\frac{L_{b2}d_2}{t^2} = \frac{(13\tfrac{1}{4} \text{ in.})(10.0 \text{ in.})}{(3/8 \text{ in.})^2}$$
$$= 942$$

$$\frac{0.08E}{F_y} = \frac{(0.08)(29{,}000 \text{ ksi})}{(50 \text{ ksi})}$$
$$= 46.4$$

$$\frac{1.9E}{F_y} = \frac{(1.9)(29{,}000 \text{ ksi})}{(50 \text{ ksi})}$$
$$= 1{,}100$$

Because 46.4 < 942 < 1,100, inelastic lateral-torsional buckling is the controlling limit state, and the nominal flexural strength is:

$$M_n = C_b\left[1.52 - 0.274\left(\frac{L_{b2}d_2}{t^2}\right)\frac{F_y}{E}\right]M_y \le M_p \qquad \text{(from } \textit{Spec}\text{. Eq. F11-2)}$$

$$= (1.00)\left[1.52 - (0.274)(942)\left(\frac{50 \text{ ksi}}{29{,}000 \text{ ksi}}\right)\right](313 \text{ kip-in.}) \le 469 \text{ kip-in.}$$

$$= 336 \text{ kip-in.}$$

The available flexural strength is:

LRFD	ASD
$\phi M_n = (0.90)(336 \text{ kip-in.})$ $= 302 \text{ kip-in.} > 190 \text{ kip-in.}$ **o.k.**	$\dfrac{M_n}{\Omega} = \dfrac{336 \text{ kip-in.}}{1.67}$ $= 201 \text{ kip-in.} > 126 \text{ kip-in.}$ **o.k.**

Shear Yielding of Leg 2

$$V_n = 0.60F_yA_{gv} \qquad \text{(from } \textit{Spec}\text{. Eq. J4-3)}$$
$$= (0.60)(50 \text{ ksi})(10.0 \text{ in.})(\tfrac{3}{8} \text{ in.})$$
$$= 113 \text{ kips}$$

The available shear strength is:

LRFD	ASD
$\phi V_n = (1.00)(113 \text{ kips})$ $= 113 \text{ kips} > 23.0 \text{ kips}$ **o.k.**	$\dfrac{V_n}{\Omega} = \dfrac{113 \text{ kips}}{1.50}$ $= 75.3 \text{ kips} > 15.3 \text{ kips}$ **o.k.**

EXAMPLE 2—GENERAL DESIGN METHOD

In this example, the strength of the gusset plate in Example 1 is calculated using the general design method.

Tension Brace Load

Flexural Strength of Leg 1

$$S_x = \frac{(\tfrac{3}{8} \text{ in.})(10.0 \text{ in.})^2}{6}$$
$$= 6.25 \text{ in.}^3$$

$$M_y = F_yS_x$$
$$= (50 \text{ ksi})(6.25 \text{ in.}^3)$$
$$= 313 \text{ kip-in.}$$

$$M_p = F_yZ \qquad \text{(from } \textit{Spec}\text{. Eq. F11-1)}$$
$$= (50 \text{ ksi})\left[\frac{(\tfrac{3}{8} \text{ in.})(10.0 \text{ in.})^2}{4}\right]$$
$$= 469 \text{ kip-in.}$$

The required moment at the interior face of the adjacent leg is:

LRFD	ASD
$M_u = P_{NS}e_2$ (from Eq. 1a) $= (32.1\text{ kips})(12.0\text{ in.})$ $= 385\text{ kip-in.}$	$M_a = P_{NS}e_2$ (from Eq. 1a) $= (21.4\text{ kips})(12.0\text{ in.})$ $= 257\text{ kip-in.}$

Using $C_b = 1.84$ and $L_{b1} = e_2 = 12.0$ in.:

$$\frac{L_{b1}d_1}{t^2} = \frac{(12.0\text{ in.})(10.0\text{ in.})}{(3/8\text{ in.})^2} = 853$$

$$\frac{0.08E}{F_y} = \frac{(0.08)(29{,}000\text{ ksi})}{(50\text{ ksi})} = 46.4$$

$$\frac{1.9E}{F_y} = \frac{(1.9)(29{,}000\text{ ksi})}{(50\text{ ksi})} = 1{,}100$$

Because 46.4 < 853 < 1,100, inelastic lateral-torsional buckling is the controlling limit state, and the nominal flexural strength is:

$$M_n = C_b\left[1.52 - 0.274\left(\frac{L_{b1}d_1}{t^2}\right)\frac{F_y}{E}\right]M_y \le M_p \quad \text{(from } \textit{Spec}\text{. Eq. F11-2)}$$

$$= (1.84)\left[1.52 - (0.274)(853)\left(\frac{50\text{ ksi}}{29{,}000\text{ ksi}}\right)\right](313\text{ kip-in.}) \le 469\text{ kip-in.}$$

$$= 469\text{ kip-in.}$$

The available flexural strength is:

LRFD	ASD
$\phi M_n = (0.90)(469\text{ kip-in.})$ $= 422\text{ kip-in.} > 385\text{ kip-in.}$ **o.k.**	$\dfrac{M_n}{\Omega} = \dfrac{469\text{ kip-in.}}{1.67}$ $= 281\text{ kip-in.} > 257\text{ kip-in.}$ **o.k.**

Shear Yielding of Leg 1

$$V_n = 0.60F_yA_{gv} \quad \text{(from } \textit{Spec}\text{. Eq. J4-3)}$$

$$= (0.60)(50\text{ ksi})(10.0\text{ in.})(3/8\text{ in.})$$

$$= 113\text{ kips}$$

The available shear strength is:

LRFD	ASD
$\phi V_n = (1.00)(113\text{ kips})$ $= 113\text{ kips} > 32.1\text{ kips}$ **o.k.**	$\dfrac{V_n}{\Omega} = \dfrac{113\text{ kips}}{1.50}$ $= 75.3\text{ kips} > 21.4\text{ kips}$ **o.k.**

Flexural Strength of Leg 2

$$S_x = \frac{(\tfrac{3}{8}\text{ in.})(10.0\text{ in.})^2}{6}$$
$$= 6.25\text{ in.}^3$$

$$M_y = F_y S_x$$
$$= (50\text{ ksi})(6.25\text{ in.}^3)$$
$$= 313\text{ kip-in.}$$

$$M_p = F_y Z \qquad \text{(from } \textit{Spec}\text{. Eq. F11-1)}$$
$$= (50\text{ ksi})\left[\frac{(\tfrac{3}{8}\text{ in.})(10.0\text{ in.})^2}{4}\right]$$
$$= 469\text{ kip-in.}$$

The required moment at the interior face of the adjacent leg is:

LRFD	ASD
$M_u = P_{EW}e_1$ (from Eq. 1b) $= (38.3\text{ kips})(8\tfrac{1}{4}\text{ in.})$ $= 316\text{ kip-in.}$	$M_a = P_{EW}e_1$ (from Eq. 1b) $= (25.5\text{ kips})(8\tfrac{1}{4}\text{ in.})$ $= 210\text{ kip-in.}$

Using $C_b = 1.84$ and $L_{b2} = e_1 = 8\tfrac{1}{4}$ in.:

$$\frac{L_{b2}d_2}{t^2} = \frac{(8\tfrac{1}{4}\text{ in.})(10.0\text{ in.})}{(\tfrac{3}{8}\text{ in.})^2}$$
$$= 587$$

$$\frac{0.08E}{F_y} = \frac{(0.08)(29{,}000\text{ ksi})}{(50\text{ ksi})}$$
$$= 46.4$$

$$\frac{1.9E}{F_y} = \frac{(1.9)(29{,}000\text{ ksi})}{(50\text{ ksi})}$$
$$= 1{,}100$$

Because 46.4 < 587 < 1,100, inelastic lateral-torsional buckling is the controlling limit state, and the nominal flexural strength is:

$$M_n = C_b\left[1.52 - 0.274\left(\frac{L_{b2}d_2}{t^2}\right)\frac{F_y}{E}\right]M_y \le M_p \qquad \text{(from } \textit{Spec}\text{. Eq. F11-2)}$$
$$= (1.84)\left[1.52 - (0.274)(587)\left(\frac{50\text{ ksi}}{29{,}000\text{ ksi}}\right)\right](313\text{ kip-in.}) \le 469\text{ kip-in.}$$
$$= 469\text{ kip-in.}$$

The available flexural strength is:

LRFD	ASD
$\phi M_n = (0.90)(469 \text{ kip-in.})$ $= 422 \text{ kip-in.} > 316 \text{ kip-in.}$ **o.k.**	$\dfrac{M_n}{\Omega} = \dfrac{469 \text{ kip-in.}}{1.67}$ $= 281 \text{ kip-in.} > 210 \text{ kip-in.}$ **o.k.**

Shear Yielding of Leg 2

$$V_n = 0.60F_yA_{gv} \qquad \text{(from } \textit{Spec.} \text{ Eq. J4-3)}$$
$$= (0.60)(50 \text{ ksi})(10.0 \text{ in.})(3/8 \text{ in.})$$
$$= 113 \text{ kips}$$

The available shear strength is:

LRFD	ASD
$\phi V_n = (1.00)(113 \text{ kips})$ $= 113 \text{ kips} > 38.3 \text{ kips}$ **o.k.**	$\dfrac{V_n}{\Omega} = \dfrac{113 \text{ kips}}{1.50}$ $= 75.3 \text{ kips} > 25.5 \text{ kips}$ **o.k.**

Compression Brace Load

Critical Load Ratio

$$\theta = 50^\circ$$

$$\alpha = \frac{d_1L_{b2}e_1}{d_2L_{b1}e_2}\tan\theta \qquad (2)$$
$$= \frac{(10.0 \text{ in.})(13\tfrac{1}{4} \text{ in.})(8\tfrac{1}{4} \text{ in.})}{(10.0 \text{ in.})(17.0 \text{ in.})(12.0 \text{ in.})}\tan(50^\circ)$$
$$= 0.639$$

Because $1/1.6 < 0.639 < 1.6$, $C_b = 1.00$ and the remaining calculations are identical to Example 1.

Both the tension and compression strength is the same as for Example 1, which used the simplified design method.

EXAMPLE 3—GUSSET WITH DIAGONAL CUT

In this example, a diagonal cut is added to the gusset plate in Figure 11 as shown in Figure 12. The simplified design method is used to calculate the strength. The gusset plate is ⅜-in.-thick ASTM A572 Grade 50 material. The LRFD and ASD loads are 60.0 kips tension/35.0 kips compression and 40.0 kips tension/23.3 kips compression, respectively.

From AISC *Manual* Table 2-4, $F_y = 50$ ksi.

Tension Brace Load

The brace axial load is:

LRFD	ASD
$P_u = 60.0$ kips (Tension)	$P_a = 40.0$ kips (Tension)

The north–south and east–west brace components are:

LRFD	ASD
$P_{NS} = (60 \text{ kips})\sin 40° = 38.6 \text{ kips}$ $P_{EW} = (60 \text{ kips})\cos 40° = 46.0 \text{ kips}$	$P_{NS} = (40.0 \text{ kips})\sin 40° = 25.7 \text{ kips}$ $P_{EW} = (40.0 \text{ kips})\cos 40° = 30.6 \text{ kips}$

Flexural Strength of Leg 1

The required moment at the reentrant corner of the diagonal cut is:

LRFD	ASD
$M_u = P_{NS}e_2$ (from Eq. 1a) $= (38.6 \text{ kips})(8.00 \text{ in.})$ $= 309 \text{ kip-in.}$	$M_a = P_{NS}e_2$ (from Eq. 1a) $= (25.7 \text{ kips})(8.00 \text{ in.})$ $= 206 \text{ kip-in.}$

When the brace is in tension, the legs are assumed to be fully braced; therefore, the nominal flexural strength according to AISC *Specification* Section F11 is:

$$M_p = F_y Z \qquad \text{(from } Spec. \text{ Eq. F11-1)}$$

$$= (50 \text{ ksi})\left[\frac{(3/8 \text{ in.})(10.0 \text{ in.})^2}{4}\right]$$

$$= 469 \text{ kip-in.}$$

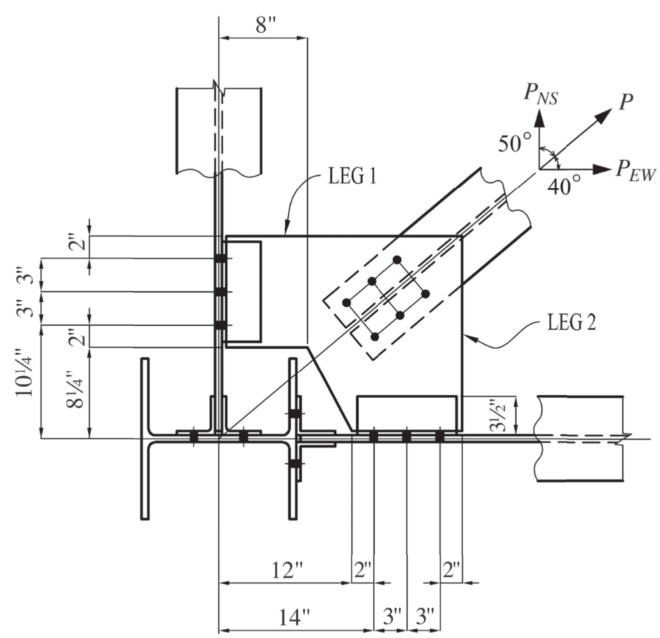


Fig. 12. Wrap-around gusset plate for Example 3.

The available flexural strength is:

LRFD	ASD
$\phi M_n = (0.90)(469\text{ kip-in.})$ $= 422\text{ kip-in.} > 309\text{ kip-in.}$ **o.k.**	$\dfrac{M_n}{\Omega} = \dfrac{469\text{ kip-in.}}{1.67}$ $= 281\text{ kip-in.} > 206\text{ kip-in.}$ **o.k.**

Shear Yielding of Leg 1

$$V_n = 0.60F_yA_{gv} \qquad \text{(from } \textit{Spec}\text{. Eq. J4-3)}$$
$$= (0.60)(50\text{ ksi})(10.0\text{ in.})(3/8\text{ in.})$$
$$= 113\text{ kips}$$

The available shear strength is:

LRFD	ASD
$\phi V_n = (1.00)(113\text{ kips})$ $= 113\text{ kips} > 38.6\text{ kips}$ **o.k.**	$\dfrac{V_n}{\Omega} = \dfrac{113\text{ kips}}{1.50}$ $= 75.3\text{ kips} > 25.7\text{ kips}$ **o.k.**

Flexural Strength of Leg 2

The required moment at the interior face of the adjacent leg is:

LRFD	ASD
$M_u = P_{EW}e_1$ (from Eq. 1b) $= (46.0\text{ kips})(8\tfrac{1}{4}\text{ in.})$ $= 380\text{ kip-in.}$	$M_a = P_{EW}e_1$ (from Eq. 1b) $= (30.6\text{ kips})(8\tfrac{1}{4}\text{ in.})$ $= 252\text{ kip-in.}$

The depth of leg 2 at the interior face of the adjacent leg is 14 in. When the brace is in tension, the legs are assumed to be fully braced; therefore, the nominal flexural strength according to AISC *Specification* Section F11 is:

$$M_p = F_yZ \qquad \text{(from } \textit{Spec}\text{. Eq. F11-1)}$$
$$= (50\text{ ksi})\left[\frac{(3/8\text{ in.})(14.0\text{ in.})^2}{4}\right]$$
$$= 919\text{ kip-in.}$$

The available flexural strength is:

LRFD	ASD
$\phi M_n = (0.90)(919\text{ kip-in.})$ $= 827\text{ kip-in.} > 380\text{ kip-in.}$ **o.k.**	$\dfrac{M_n}{\Omega} = \dfrac{919\text{ kip-in.}}{1.67}$ $= 550\text{ kip-in.} > 252\text{ kip-in.}$ **o.k.**

Shear Yielding of Leg 2

The shear strength will be analyzed at a plane immediately beyond the clip angle leg. The gusset plate leg width at this location is:

d_{2V} = 10.0 in + (4.00 in.)(3.50 in./8.25 in.)
= 11.7 in.

$V_n = 0.60 F_y A_{gv}$ (from *Spec*. Eq. J4-3)
= (0.60)(50 ksi)(11.7 in.)(0.375 in.)
= 132 kips

The available shear strength is:

LRFD	ASD
$\phi V_n = (1.00)(132 \text{ kips})$ = 132 kips > 46.0 kips **o.k.**	$\dfrac{V_n}{\Omega} = \dfrac{132 \text{ kips}}{1.50}$ = 88.0 kips > 30.6 kips **o.k.**

Compression Brace Load

The brace axial load is:

LRFD	ASD
P_u = 35.0 kips (compression)	P_a = 23.3 kips (compression)

The north–south and east–west brace components are:

LRFD	ASD
P_{NS} = (35.0 kips)sin40° = 22.5 kips P_{EW} = (35.0 kips)cos40° = 26.8 kips	P_{NS} = (23.3 kips)sin40° = 15.0 kips P_{EW} = (23.3 kips)cos40° = 17.8 kips

Flexural Strength of Leg 1

$$S_x = \frac{(\tfrac{3}{8} \text{ in.})(10.0 \text{ in.})^2}{6}$$
$$= 6.25 \text{ in.}^3$$

$M_p = F_y Z$ (from *Spec*. Eq. F11-1)

$$= (50 \text{ ksi})\left[\frac{(\tfrac{3}{8} \text{ in.})(10.0 \text{ in.})^2}{4}\right]$$
$$= 469 \text{ kip-in.}$$

The required moment at the mid-point of the diagonal cut is:

LRFD	ASD
$M_u = P_{NS} e_{m2}$ (from Eq. 1a) = (22.5 kips)(10.0 in.) = 225 kip-in.	$M_a = P_{NS} e_{m2}$ (from Eq. 1a) = (15.0 kips)(10.0 in.) = 150 kip-in.

Using C_b = 1.00 and $L_{b1} = e_2 + d_2/2$ = 12 in. + 5 in. = 17.0 in.:

$$\frac{L_{b1} d_1}{t^2} = \frac{(17.0 \text{ in.})(10.0 \text{ in.})}{(\tfrac{3}{8} \text{ in.})^2}$$
$$= 1{,}210$$

$$\frac{1.9E}{F_y} = \frac{(1.9)(29,000 \text{ ksi})}{(50 \text{ ksi})}$$
$$= 1,100$$

Because 1,210 > 1,100, elastic lateral-torsional buckling is the controlling limit state, and the critical stress is:

$$F_{cr} = \frac{1.9EC_b}{\frac{L_{b1}d_1}{t^2}} \qquad \text{(from } \textit{Spec}\text{. Eq. F11-4)}$$
$$= \frac{(1.9)(29,000 \text{ ksi})(1.00)}{1,210}$$
$$= 45.5 \text{ ksi}$$

The nominal flexural strength is:

$$M_n = F_{cr}S_x \le M_p \qquad (\textit{Spec}\text{. Eq. F11-3})$$
$$= (45.5 \text{ ksi})(6.25 \text{ in.}^3) \le 469 \text{ kip-in.}$$
$$= 284 \text{ kip-in.}$$

The available flexural strength is:

LRFD	ASD
$\phi M_n = (0.90)(284 \text{ kip-in.})$ $= 256 \text{ kip-in.} > 225 \text{ kip-in.}$ **o.k.**	$\frac{M_n}{\Omega} = \frac{284 \text{ kip-in.}}{1.67}$ $= 170 \text{ kip-in.} > 150 \text{ kip-in.}$ **o.k.**

Shear Yielding of Leg 1

$$V_n = 0.60F_yA_{gv} \qquad \text{(from } \textit{Spec}\text{. Eq. J4-3)}$$
$$= (0.60)(50 \text{ ksi})(10.0 \text{ in.})(\tfrac{3}{8} \text{ in.})$$
$$= 113 \text{ kips}$$

The available shear strength is:

LRFD	ASD
$\phi V_n = (1.00)(113 \text{ kips})$ $= 113 \text{ kips} > 22.5 \text{ kips}$ **o.k.**	$\frac{V_n}{\Omega} = \frac{113 \text{ kips}}{1.50}$ $= 75.3 \text{ kips} > 15.0 \text{ kips}$ **o.k.**

Flexural Strength of Leg 2

$$S_x = \frac{(\tfrac{3}{8} \text{ in.})(10.0 \text{ in.})^2}{6}$$
$$= 6.25 \text{ in.}^3$$

$$M_y = F_yS_x$$
$$= (50 \text{ ksi})(6.25 \text{ in.}^3)$$
$$= 313 \text{ kip-in.}$$

$$M_p = F_y Z \quad \text{(from } \textit{Spec}\text{. Eq. F11-1)}$$

$$= (50 \text{ ksi})\left[\frac{(\tfrac{3}{8}\text{ in.})(10.0\text{ in.})^2}{4}\right]$$

$$= 469 \text{ kip-in.}$$

The required moment at the mid-point of the diagonal cut is:

LRFD	ASD
$M_u = P_{EW}e_{m1}$ (from Eq. 1b) $= (26.8\text{ kips})(4\tfrac{3}{8}\text{ in.})$ $= 117\text{ kip-in.}$	$M_a = P_{EW}e_1$ (from Eq. 1b) $= (17.8\text{ kips})(4\tfrac{3}{8}\text{ in.})$ $= 77.9\text{ kip-in.}$

Using $C_b = 1.00$ and $L_{b2} = e_1 + d_1/2 = 8\tfrac{1}{4}$ in. + 5 in. = $13\tfrac{1}{4}$ in.:

$$\frac{L_{b2}d_2}{t^2} = \frac{(13\tfrac{1}{4}\text{ in.})(10.0\text{ in.})}{(\tfrac{3}{8}\text{ in.})^2}$$

$$= 942$$

$$\frac{0.08E}{F_y} = \frac{(0.08)(29{,}000\text{ ksi})}{(50\text{ ksi})}$$

$$= 46.4$$

$$\frac{1.9E}{F_y} = \frac{(1.9)(29{,}000\text{ ksi})}{(50\text{ ksi})}$$

$$= 1{,}100$$

Because 46.4 < 942 < 1,100, inelastic lateral-torsional buckling is the controlling limit state, and the nominal flexural strength is:

$$M_n = C_b\left[1.52 - 0.274\left(\frac{L_{b2}d_2}{t^2}\right)\frac{F_y}{E}\right]M_y \le M_p \quad \text{(from } \textit{Spec}\text{. Eq. F11-2)}$$

$$= (1.00)\left[1.52 - (0.274)(942)\left(\frac{50\text{ ksi}}{29{,}000\text{ ksi}}\right)\right](313\text{ kip-in.}) \le 469\text{ kip-in.}$$

$$= 336\text{ kip-in.}$$

The available flexural strength is:

LRFD	ASD
$\phi M_n = (0.90)(336\text{ kip-in.})$ $= 302\text{ kip-in.} > 117\text{ kip-in.}$ **o.k.**	$\dfrac{M_n}{\Omega} = \dfrac{336\text{ kip-in.}}{1.67}$ $= 201\text{ kip-in.} > 77.9\text{ kip-in.}$ **o.k.**

The required moment at the interior face of the adjacent leg is:

LRFD	ASD
$M_u = P_{EW}e_1$ (from Eq. 1b) $= (26.8\text{ kips})(8\tfrac{1}{4}\text{ in.})$ $= 221\text{ kip-in.}$	$M_a = P_{EW}e_1$ (from Eq. 1b) $= (17.8\text{ kips})(8\tfrac{1}{4}\text{ in.})$ $= 147\text{ kip-in.}$

The depth of leg 2 at the reentrant corner is 14 in. For yielding at the reentrant corner, the legs are assumed to be fully braced; therefore, the nominal flexural strength according to AISC *Specification* Section F11 is:

$$M_p = F_y Z \qquad \text{(from } \textit{Spec.} \text{ Eq. F11-1)}$$

$$= (50 \text{ ksi})\left[\frac{(\tfrac{3}{8} \text{ in.})(14.0 \text{ in.})^2}{4}\right]$$

$$= 919 \text{ kip-in.}$$

The available flexural strength is:

LRFD	ASD
$\phi M_n = (0.90)(919 \text{ kip-in.})$ $= 827 \text{ kip-in.} > 221 \text{ kip-in.}$ **o.k.**	$\dfrac{M_n}{\Omega} = \dfrac{919 \text{ kip-in.}}{1.67}$ $= 550 \text{ kip-in.} > 147 \text{ kip-in.}$ **o.k.**

Shear Yielding of Leg 2

The shear strength will be analyzed at a plane immediately beyond the clip angle leg. The gusset plate leg width at this location is:

$$d_{2V} = 10.0 \text{ in} + (4.00 \text{ in.})(3.50 \text{ in.}/8.25 \text{ in.})$$
$$= 11.7 \text{ in.}$$

$$V_n = 0.60 F_y A_{gv} \qquad \text{(from } \textit{Spec.} \text{ Eq. J4-3)}$$
$$= (0.60)(50 \text{ ksi})(11.7 \text{ in.})(0.375 \text{ in.})$$
$$= 132 \text{ kips}$$

The available shear strength is:

LRFD	ASD
$\phi V_n = (1.00)(132 \text{ kips})$ $= 132 \text{ kips} > 26.8 \text{ kips}$ **o.k.**	$\dfrac{V_n}{\Omega} = \dfrac{132 \text{ kips}}{1.50}$ $= 88.0 \text{ kips} > 17.8 \text{ kips}$ **o.k.**

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