Effective Length Factors for Gusset Plate Buckling

BO DOWSWELL

Gusset plates are commonly used in steel buildings to connect bracing members to other structural members in the lateral force resisting system. Figure 1 shows a typical vertical bracing connection at a beam-to-column intersection.

Problem Statement

Design procedures have previously been developed from the research on gusset plates in compression. The design procedures to determine the buckling capacity of gusset plates are well documented (AISC, 2001), but the accuracy of these procedures is not well established. Uncertainties exist in the selection of the effective length factor for each gusset plate configuration.

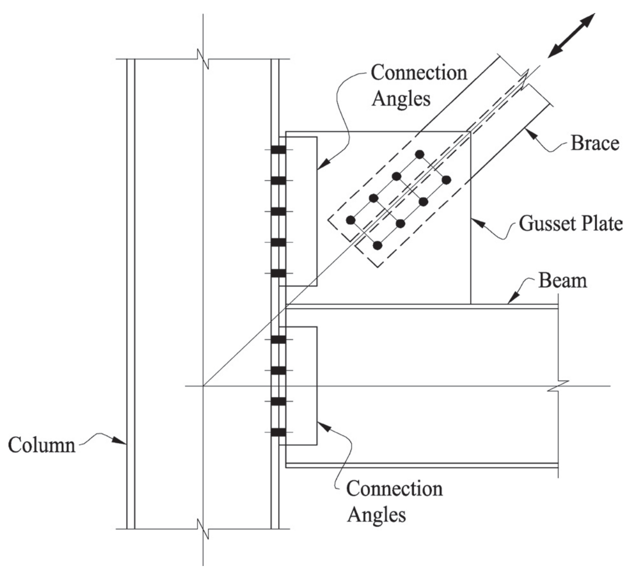

Fig. 1. Vertical brace connection.

Bo Dowswell is principal of Structural Design Solutions, LLC, Birmingham, AL.

Objective

The objectives of this research were to determine the accuracy of the current design method using a statistical analysis of data from the available research, and to propose appropriate effective length factors for use in the current design procedures.

Procedure

The available experimental and finite element data on the buckling capacity of gusset plates was collected and reviewed. The experimental and finite element capacities were compared with the calculated nominal loads for each specimen, and the most appropriate effective length factor was selected for each gusset plate configuration. The accuracy of the design method was determined using the selected effective length factors.

Gusset Plate Configurations

Gusset plates are fabricated in many different configurations. The most common configurations are shown in Figure 2.

The corner-brace configuration has a single brace framing to the gusset plate at the intersection of two other structural members. The gusset plate is connected to both members. There are three types of corner-brace configurations that are considered in this paper: compact, noncompact, and extended. For compact gusset plates, the free edges of the gusset plate are parallel to the connected edges and the brace member is pulled in close to the other framing members as shown in Figure 2a. For noncompact gusset plates, the free edges of the gusset plate are parallel to the connected edges and the brace member is not pulled in close to the other framing members. This configuration is shown in Figure 2b. For the extended corner-brace configuration, the gusset plate is shaped so the free edges of the gusset plate are cut at an angle to the connected edges as shown in Figure 2c. The extended corner-brace configuration is mainly used where high seismic loads are expected. The additional setback is tended to ensure ductile behavior in extreme seismic events by allowing a plastic hinge to develop in the free length between the end of the brace and the restrained edges of the gusset plate.

The gusset plate in the single-brace configuration is connected at only one edge of the plate as shown in Figure 2d.

The chevron-brace configuration has two braces framing to the gusset plate as shown in Figure 2e. The gusset plate is connected at only one edge of the plate.

CURRENT DESIGN PROCEDURES

Effective Width

In design, gusset plates are treated as rectangular, axially loaded members with a cross section $L_w \times t$, where L_w is the effective width, and t is the gusset plate thickness. The effective width is calculated by assuming the stress spreads through the gusset plate at an angle of 30°. The effective width is shown in Figure 3 for various connection configurations and is defined as the distance perpendicular to the load, where 30° lines, which project from the first row of bolts or the start of the weld, intersect at the last row of bolts or the end of the weld. The effective cross section is commonly referred to as the "Whitmore Section."

Buckling Capacity

Thornton (1984) proposed a method to calculate the buckling capacity of gusset plates. He recommended that the gusset plate area between the brace end and the framing members be treated as a rectangular column with a cross section $L_w \times t$. For corner gusset plates, the column length, l_{avg} is calculated as the average of l_1, l_2 , and l_3 as shown in Figure 4. The buckling capacity is then calculated using the column curve in the AISC *Load and Resistance Factor Design Specification for Structural Steel Buildings* (AISC, 1999), hereafter referred to as the AISC Specification.

The AISC *Load and Resistance Factor Design Manual of Steel Construction, Volume II, Connections* (AISC, 1995) provides effective length factors for compact corner gusset plates, noncompact corner gusset plates, and single-brace gusset plates. The effective length factors and the suggested buckling lengths are summarized in Table 1. (Tables begin on page 99.) Table 1 also shows the average ratio of experimental buckling load to calculated nominal capacity based on the tests and finite element models in Tables 2, 3, and 5. The current design method is conservative by 47% for compact corner gusset plates and is conservative by 140% for single-brace gusset plates. The current design method for noncompact corner gusset plates appears to be accurate based on the test-to-predicted ratio of 0.98; however, the standard deviation is 0.46, and the test-to-predicted ratio was as low as 0.33 for one of the specimens. There appears to be a source of improvement in the design procedure for these three gusset plate configurations by simply selecting an effective length factor that gives predicted capacities closer to the test and finite element results.

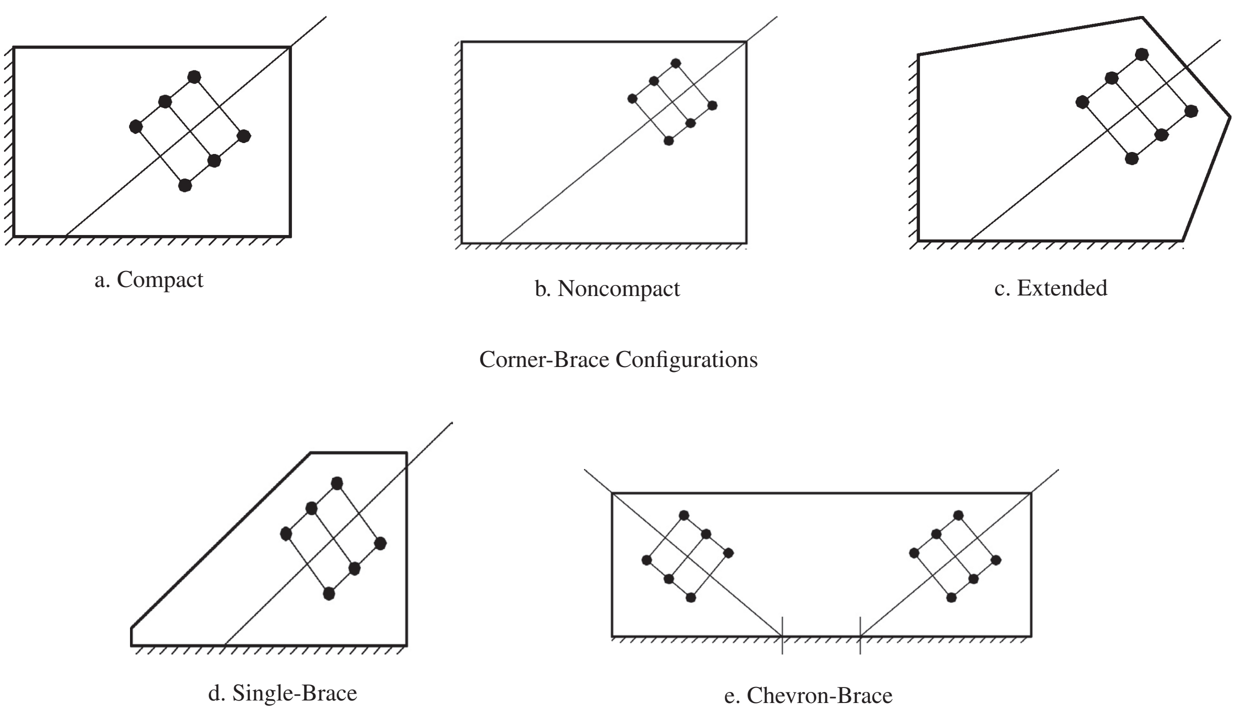

Fig. 2. Gusset plate configurations.

Sources of Inaccuracy

Some potential sources of inaccuracy in the design model are as follows:

- The out-of-plane restraint provided by the portions of the plate outside the boundaries of the effective width is neglected.
- The Poisson effect is neglected. The out-of-plane bending stiffness of the gusset plate is about 10% greater if the Poisson effect is included in the calculation (Timoshenko and Woinowsky-Krieger, 1959).
- The secondary stresses in the gusset plate due to frame action are neglected. When the brace at a corner gusset plate is in compression, the angle between the beam and column will increase, causing tension stresses to develop perpendicular to the brace load.

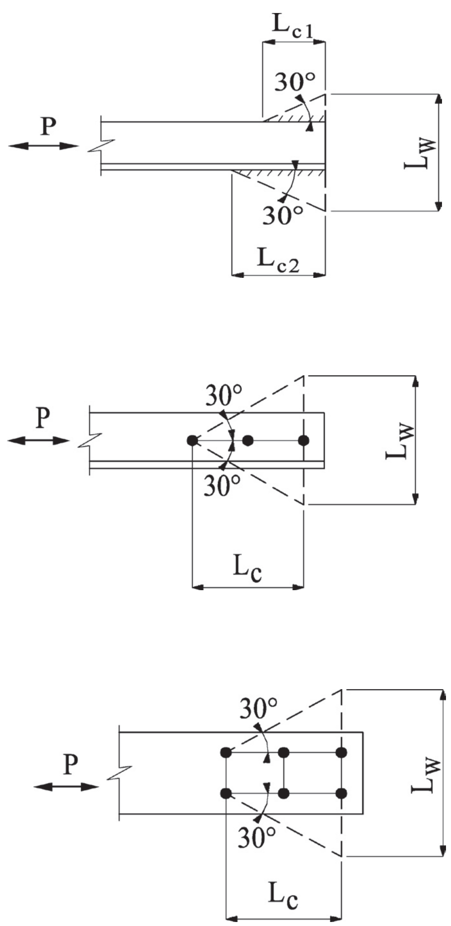

Fig. 3. Effective width for various connection configurations.

- The column curve in the AISC Specification (AISC, 1999) was not developed for plates. It was developed for column shapes with a specific residual stress pattern and an initial out-of-straightness of L /1,500. The expected out-of-flatness of gusset plates will almost always exceed the out-of-straightness of a typical column when expressed as the ratio of the out-of-flatness/buckling length.

ASTM A6 (2004) specifies a permissible camber of 0.025 in./ft and a permissible variation from flat of 0.25 in. for carbon steel plates less than 36 in. long. This gives a maximum permissible out-of-flatness of L/480, which is twice the permissible out-of-straightness of columns less than 30 ft long. The ASTM Standard specifies manufacturing tolerances and does not address the tolerances for plates after fabrication is complete. Some deformations can be expected from the shop operations. Fouad, Davidson, Delatte, Calvert, Chen, Nunez, and Abdalla (2003) surveyed state departments of transportation, manufacturers, and engineers to determine the current state of practice regarding flatness tolerances for connection plates and base plates. They recommended using the flatness requirements of ASTM A6 after fabrication is complete. Dowswell (2005) measured the out-of-flatness of gusset plates fabricated by a shop experienced in steel structures and certified by AISC for complex steel buildings and found the maximum out-of-plane imperfection to be 0.028 in., which gives a ratio of l_1/535. Additionally, gusset plates are usually welded and are sometimes galvanized. These operations will typically increase the out-of-flatness. The finite element models by Walbridge, Grondin, and Cheng (1998) indicated that the magnitude of the initial out-of-flatness significantly affects the compressive capacity of gusset plates.

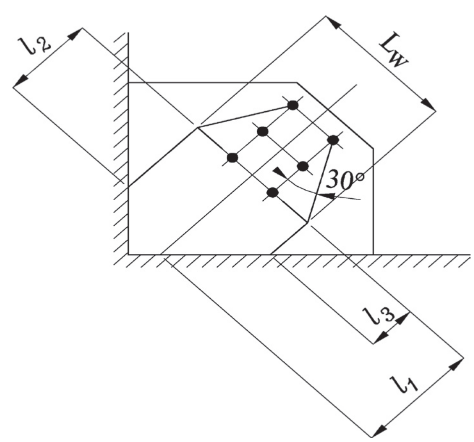

Fig. 4. Geometry of effective column.

- Although the effective width is taken at the last fastener at the end of the brace, the stress continues to spread out beyond the last fastener.

- Due to stress redistribution when the plate yields, the effective width in the inelastic range is larger than predicted using the 30° angle. Cheng and Grondin (1999) have proposed that the effective width be calculated using a 45° angle to account for the stress redistribution.

EXISTING RESEARCH

A large number of research projects have been dedicated to the behavior of gusset plates. The research includes laboratory tests, finite element models, and theoretical studies. A literature review revealed a total of 170 experimental specimens and finite element models with compressive loads applied. There were 10 linear finite element models, 67 nonlinear finite element models, and 93 experimental specimens.

The first major experimental work on gusset plates was by Wyss (1926). The stress trajectories were plotted for gusset plate specimens representing a Warren truss joint. The maximum normal stress was at the end of the brace member. Wyss noted that the stress trajectories were along approximately 30° lines with the connected member. The experimental investigations and finite element models of Sandel (1950), Whitmore (1952), Irvan (1957), Lavis (1967), Raburn (1983), and Bjorhovde and Chakrabarti (1985) confirmed Wyss' results.

Chakrabarti and Richard (1990) used finite element models to determine the buckling capacity of gusset plates in Warren truss joints and vertical brace joints. They modeled four of the eight specimens that were tested experimentally by Yamamoto, Akiyama, and Okumura (1988). The interface between the gusset plate and the chord was fixed against translation and rotation. The plate was held from out-of-plane translation where the diagonal members connected to the plate. The test results as well as the finite element models indicated that the gusset plates buckled inelastically. The loads from the inelastic models compared well with the experiments.

Brown (1988) tested 24 half-scale vertical brace connections with corner gusset plates. The plates were 15 in. square and fabricated from ASTM A36 steel. They were $\frac{3}{16}$ in., $\frac{1}{4}$ in., and $\frac{3}{8}$ in. thick. The brace angles varied from 26° to 55°. Two bracing members with different bolt patterns were used. Most of the tests failed by buckling of the longer free edge.

Gross and Cheok (1988) tested three nearly full-scale braced frame subassemblies. The specimens were loaded in tension and compression. The main parameters of study were the gusset geometry, eccentricity of forces in the connection, and orientation of the column. All but one of the gusset plates failed by buckling. All specimens exhibited yielding before reaching the ultimate load.

Astaneh (1992) conducted three experiments on gusset plate specimens representing a brace-to-beam connection for a chevron bracing system subjected to cyclic loading. The $\frac{1}{4}$-in.-thick gusset plates of A36 steel were welded to the beam flange. Both braces were double channels, C4×7.25, and were bolted to the plate at 45° angles. Specimen 3 failed due to "dramatic and almost sudden buckling." The buckling was attributed to the high-stress concentration in the plate at the end of the bracing member and the long, horizontal, free edge at the top of the plate.

Cheng and Hu (1987) investigated the behavior of full-scale gusset plates in compression. The primary failure mode was buckling of the gusset plates.

Yam and Cheng (1993) tested 19 gusset plate specimens in compression. The primary failure mode was buckling of the gusset plates. Yielding was observed in most of the specimens prior to buckling.

Rabinovitch and Cheng (1993) tested five brace member and gusset plate assemblies under cyclic loading. The compressive failure mode was overall buckling.

Walbridge et al. (1998) developed finite element models of gusset plates that were validated with the experimental results of Yam and Cheng (1993) and Rabinovitch and Cheng (1993). They found that the capacity of the gusset plates could be accurately predicted using a linear elastic–perfectly plastic material model, a 2-mm quarter sine wave initial imperfection, and full restraint at the splice member.

Nast, Grondin, and Cheng (1999) conducted a numerical and experimental investigation of the effects of free edge stiffeners and brace member–gusset plate interaction on the behavior of corner gusset plates subjected to cyclic loading. They tested four full-scale gusset plate assemblies, but only one unstiffened specimen was designed to fail by buckling.

Sheng, Yam, and Iu (2002) conducted a parametric study of the buckling capacity of gusset plates using the results from finite element models. Both material and geometric nonlinearities were considered in their models. The beam and column boundaries were fully fixed and infinite rotational restraint was provided to the brace member.

RESULTS

A review of the 170 experimental specimens and finite element models revealed only 59 experimental specimens and 56 finite element models that produced reliable data for comparison with the calculated nominal capacities. The remaining 55 gusset plates were excluded from the database for a variety of reasons. Chakrabarti and Richard (1990) concluded that elastic finite element models are not accurate in predicting the buckling capacity of gusset plates; therefore, the finite element models with elastic material models were excluded. An experimental specimen was excluded for the following reasons: It was not loaded to failure, it had a failure mode other than buckling, it was loaded with an out-of-plane eccentricity, the plate had edge stiffeners, or sufficient data were not available.

The current design procedure is semi-empirical, with the empirical aspects of the design method being the 30° stress trajectory in the plate, the buckling length, and the effective length factor. The accuracy of the 30° stress trajectory in the elastic range is well established; therefore, only the buckling length and the effective length factor were examined in this paper.

Walbridge et al. (1998) showed that the accuracy of finite element models is dependent on the level of mesh refinement, the initial out-of-flatness of the plate, the material model, the boundary conditions, and other characteristics of the model. Because of this, the experimental specimens were given a higher level of confidence in the selection of the proposed effective length factors.

For corner gusset plates, where there were many experimental results, the finite element models of Sheng et al. (2002) consistently predicted capacities that were higher than average. This was taken into account in the selection of the effective length factor for extended corner braces and single braces, where almost all of the specimens were from Sheng et al. (2002).

Bracing Setback Dimension

Figure 5 shows the bracing setback dimension, l_1 for a corner gusset plate. Compact corner gusset plates generally buckled in a nonsway mode. The specimens with very large setback dimensions had dramatically decreased buckling capacities. An obvious reason for this decrease in capacity is the increased effective length, but the experimental results summarized in Tables 1 and 2 show a more dramatic decrease in capacity than the increased length alone would cause. To account for this, a limiting gusset plate thickness is derived based on the column bracing requirements in the AISC Specification (AISC, 1999). At some point, if the setback dimension is large enough, gusset plates will buckle in a sway mode. The following derivation is to determine the minimum gusset plate thickness that will force the gusset plate to buckle in a nonsway mode. This will be used to determine if the gusset plate is compact or noncompact.

According to AISC Specification (AISC, 1999) Section C3.3, a brace will be effective if the actual stiffness is greater than the required stiffness. The required stiffness for relative bracing at a column is

$$\beta_{br} = \frac{2P_u}{\phi L_b} \tag{1}$$

P_u is the applied axial load, L_b is the distance between braces, and $\phi = 0.75$. An equation for the required gusset plate thickness can be determined from the model shown in Figure 6. The lateral bracing is provided by a 1-in. strip of the plate in bending with a length c. When the vertical strip buckles, a virtual out-of-plane displacement occurs that is resisted by the horizontal strip. The horizontal strip is modeled using a guided cantilever. If the horizontal strip is infinitely stiff, the theoretical effective length factor for the vertical strip is 0.5. If the horizontal strip has no stiffness, the theoretical effective length factor for the vertical strip is 1.0. Dimension c is the shorter of the two distances from the connected edge of the plate to the brace connection as shown in Figure 5.

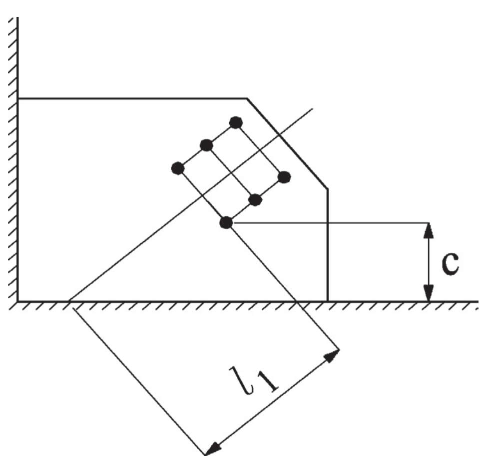

Fig. 5. Definition of plate cantilever length, c.

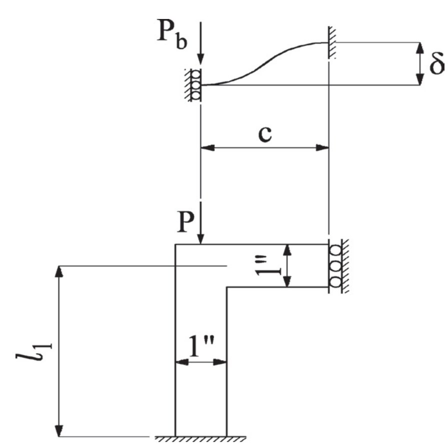

Fig. 6. L-shaped model used to determine the brace stiffness of a gusset plate.

The maximum design load that can be carried by the 1-in.-vertical strip in compression is

$$P_{max} = 0.85(1\text{ in.})F_y\, t \tag{2}$$

F_y is the yield strength of the plate. Substitute P_{max} into Equation 1 and replace L_b with l_1 to get

$$\beta_{br} = \frac{(2)(0.85)(1\text{ in.})F_y t}{(0.75)l_1} = 2.27\frac{F_y t}{l_1} \tag{3}$$

For a guided cantilever with a point load at the tip, the end deflection is

$$\delta = \frac{P_b}{12EI}c^3 \tag{4}$$

P_b is the point load at the end of the cantilever, and E is the modulus of elasticity. The moment of inertia of the 1-in.-horizontal strip is

$$I = \frac{(1\text{ in.})t^3}{12} \tag{5}$$

The actual stiffness of the horizontal strip is

$$\beta = \frac{P_b}{\delta} = E\left(\frac{t}{c}\right)^3 \tag{6}$$

Set the actual stiffness equal to the required stiffness.

$$E\left(\frac{t}{c}\right)^3 = 2.27\frac{F_y t}{l_1} \tag{7}$$

Solve for the required plate thickness, t_β.

$$t_\beta = 1.5\sqrt{\frac{F_y c^3}{El_1}} \tag{8}$$

The gusset plate is compact if $t \geq t_\beta$, and noncompact if $t < t_\beta$.

The model shown in Figure 6 can also be used to determine the required strength to brace the gusset plate. The AISC Specification (AISC, 1999) provision for required strength of relative bracing at a column is

$$P_{br} = 0.004P_u \tag{9}$$

Substitute P_{max} from Equation 2 into Equation 1 to get

$$P_{br} = (0.0034\text{ in.})F_y\, t \tag{10}$$

The moment in the horizontal strip in double curvature is

$$M_u = P_b\, c/2 = (0.0017\text{ in.})F_y\, ct \tag{11}$$

The design moment capacity of the horizontal strip is

$$\phi M_n = 0.9F_y\frac{(1\text{ in.})t^2}{4} = 0.225t^2F_y \tag{12}$$

Set the applied moment to the moment capacity and solve for t.

$$t_p = 0.0075c \tag{13}$$

From Equation 13, it can be seen that the strength requirement is insignificant for any practical gusset plate geometry; therefore, only the stiffness requirement will be used to determine the buckling mode.

Yielding Design for Compact Corner Gusset Plates

Because compact corner gusset plates generally buckle in the inelastic range as discussed by Cheng and Grondin (1999), a lower-bound solution to the test data is the yield capacity of the plate at the effective section. The yield capacity is calculated with an effective width L_w, which is based on a 30° spread of the load. It is determined with the following equation,

$$P_y = F_y\, tL_w \tag{14}$$

Table 2 shows the yield loads of the compact corner-brace specimens, and compares them with the experimental and finite element loads. There were eight separate projects with a total of 68 specimens: 37 were experimental and 31 were finite element models. The mean ratio of experimental load to calculated capacity, P_{exp}/P_{calc} is 1.36, and the standard deviation is 0.23.

Effective Length Factors

Tables 3 through 6 compare the results from the tests and finite element models with the nominal buckling capacities. The nominal buckling capacities were calculated with Thornton's design model for effective length with the column curve in the AISC Specification (AISC, 1999). The statistical results for noncompact corner braces indicated that l_{avg} is a more accurate buckling length than l_1. For the other gusset plate configurations, l_1 is as accurate as l_{avg}. The proposed effective length factors were correlated for use with l_{avg} at the noncompact corner gusset plates and l_1 at the other configurations.

The results for noncompact corner braces are summarized in Table 3. There were two projects with a total of 12 experimental specimens. Using a buckling length, l_{avg} and an effective length factor of 1.0, the mean ratio of experimental load to calculated capacity, P_{exp}/P_{calc} is 3.08. The standard deviation is 1.94.

The results for extended corner braces are summarized in Table 4. There were a total of 13 specimens from two separate projects. Only one of the specimens was experimental, and 12 were finite element models. Using a buckling length l_1, and an effective length factor of 0.60, the mean ratio of experimental load to calculated capacity, P_{exp}/P_{calc} is 1.45. The standard deviation is 0.20.

The results for single braces are summarized in Table 5. There was only one project with nine finite element models. Using a buckling length, l_1 and an effective length factor of 0.70, the mean ratio of experimental load to calculated capacity, P_{exp}/P_{calc} is 1.45. The standard deviation is 0.20.

The results for chevron braces are summarized in Table 6. There were two separate projects with a total of 13 specimens—nine were experimental and four were finite element models. Using a buckling length, l_1 and an effective length factor of 0.75, the mean ratio of experimental load to calculated capacity, P_{exp}/P_{calc} is 1.25. The standard deviation is 0.22.

CONCLUSION

Using the experimental and finite element data from the previous studies, the capacity of gusset plates in compression were compared with the current design procedures. Based on a statistical analysis, effective length factors were proposed for use with the design procedures. Table 7 summarizes the proposed effective length factors.

It was determined that compact corner gusset plates can be designed without consideration of buckling effects, and yielding at the effective width is an accurate predictor of their compressive capacity. Due to the high variability of the test-to-predicted ratios for the noncompact corner gusset plates, an effective length factor was proposed that was conservative for most of the specimens. For the extended corner gusset plates, the single brace gusset plates, and the chevron brace gusset plates, effective length factors were proposed that resulted in reasonably accurate capacities when compared with the test and finite element capacities.

REFERENCES

AISC (2001), *Manual of Steel Construction, Load and Resistance Factor Design*, Third Edition, American Institute of Steel Construction, Inc., Chicago, IL.

AISC (1999), *Load and Resistance Factor Design Specification for Structural Steel Buildings*, American Institute of Steel Construction, Inc., Chicago, IL.

AISC (1995), *Manual of Steel Construction, Load and Resistance Factor Design, Volume II*, *Connections*, American Institute of Steel Construction, Chicago, IL.

Astaneh, A. (1992), "Cyclic Behavior of Gusset Plate Connections in V-Braced Steel Frames," *Stability and Ductility of Steel Structures Under Cyclic Loading*, Y. Fukomoto and G.C. Lee, editors, CRC Press, Ann Arbor, MI, pp. 63–84.

ASTM (2004), "Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling," ASTM A6, ASTM International, West Conshohocken, PA.

Bjorhovde, R. and Chakrabarti, S.K. (1985), "Tests of Full-Size Gusset Plate Connections," *Journal of Structural Engineering*, ASCE, Vol. 111, No. 3, March, pp. 667–683.

Brown, V.L. (1988), "Stability of Gusseted Connections in Steel Structures," Ph.D. Dissertation, University of Delaware.

Chakrabarti, S.K. and Richard, R.M. (1990), "Inelastic Buckling of Gusset Plates," *Structural Engineering Review*, Vol. 2, pp. 13–29.

Chakrabarti, S.K. (1987), *Inelastic Buckling of Gusset Plates*, Ph.D. Dissertation, University of Arizona.

Cheng, J.J.R. and Grondin, G.Y. (1999), "Recent Development in the Behavior of Cyclically Loaded Gusset Plate Connections," *Proceedings, 1999 North American Steel Construction Conference*, American Institute of Steel Construction, Chicago, IL.

Cheng, J.J.R. and Hu, S.Z. (1987), "Comprehensive Tests of Gusset Plate Connections," *Proceedings, 1987 Annual Technical Session*, Structural Stability Research Council, pp. 191–205.

Dowswell, B. (2005), *Design of Steel Gusset Plates with Large Cutouts*, Ph.D. Dissertation, University of Alabama at Birmingham.

Fisher, J.W., Galambos, T.V., Kulak, G.L., and Ravindra, M.K. (1978), "Load and Resistance Factor Design Criteria for Connectors," *Journal of the Structural Division*, ASCE, Vol. 104, No. ST9, September, pp. 1427–1441.

Fouad, F.H., Davidson, J.S., Delatte, N., Calvert, E.A., Chen, S., Nunez, E., and Abdalla, R. (2003), "Structural Supports for Highway Signs, Luminaries, and Traffic Signals," NCHRP Report 494, Transportation Research Board, Washington, DC.

Gross, J.L. and Cheok, G. (1988), "Experimental Study of Gusseted Connections for Laterally Braced Steel Buildings," National Institute of Standards and Technology, Gaithersburg, MD, November.

Irvan, W.G. (1957), "Experimental Study of Primary Stresses in Gusset Plates of a Double Plane Pratt Truss," University of Kentucky, Engineering Research Station Bulletin No. 46, December.

Lavis, C.S. (1967), "Computer Analysis of the Stresses in a Gusset Plate," Masters Thesis, University of Washington.

Nast, T.E., Grondin, G.Y., and Cheng, J.J.R. (1999), "Cyclic Behavior of Stiffened Gusset Plate Brace Member Assemblies," *Structural Engineering Report No. 229*, University of Alberta, Department of Civil and Environmental Engineering, December.

Rabinovitch, J. and Cheng, J.J.R. (1993), "Cyclic Behavior of Steel Gusset Plate Connections," *Structural Engineering Report No. 191*, University of Alberta, Department of Civil and Environmental Engineering, August.

Raburn, D.A. (1983), "Stress, Strain and Force Distributions in Gusset Plate Connections," Masters Thesis, University of Arizona.

Sandel, J.A. (1950), "Photoelastic Analysis of Gusset Plates," Masters Thesis, University of Tennessee, December.

Sheng, N., Yam, C.H., and Iu, V.P. (2002), "Analytical Investigation and the Design of the Compressive Strength of Steel Gusset Plate Connections," *Journal of Constructional Steel Research*, Vol. 58, pp. 1473–1493.

Thornton, W.A. (1984), "Bracing Connections for Heavy Construction," *Engineering Journal*, American Institute of Steel Construction, 3rd Quarter, pp. 139–148.

Timoshenko, S. and Woinowsky-Krieger, S. (1959), *Theory of Plates and Shells*, 2nd edition, McGraw-Hill, New York.

Walbridge, S.S., Grondin, G.Y., and Cheng, J.J.R. (1998), "An Analysis of the Cyclic Behavior of Steel Gusset Plate Connections," *Structural Engineering Report No. 225*, University of Alberta, Department of Civil and Environmental Engineering, September.

Whitmore, R.E. (1952), "Experimental Investigation of Stresses in Gusset Plates," Bulletin No. 16, University of Tennessee Engineering Experiment Station, May.

Wyss, T. (1926), "Die Kraftfelder in Festen Elastischen Korpern und ihre Praktischen Anwendungen," Berlin.

Yam, M.C.H. and Cheng, J.J.R. (1993), "Experimental Investigation of the Compressive Behavior of Gusset Plate Connections," University of Alberta Department of Civil Engineering Structural Engineering Report No. 194, September.

Yamamoto, K., Akiyama, N., and Okumura, T. (1988), "Buckling Strengths of Gusseted Truss Joints," *Journal of Structural Engineering*, ASCE, Vol. 114, No. 3, March, pp. 575-591.

Table 1. Summary of Current Effective Length Factors			
Gusset Configuration	Effective Length Factor	Buckling Length	$\frac{P_{exp}}{P_{calc}}$
Compact corner	0.5	l_{avg}	1.47
Noncompact corner	0.5	l_{avg}	0.98
Single brace	1.2	l_1	2.40

Table 2. Details and Calculated Capacity of Compact Corner Gusset Plates Full Yield Capacity

Spec. No.	t (in.)	L_w (in.)	F_y (ksi)	P_{calc} (k)	P_{exp} (k)	$\frac{P_{exp}}{P_{calc}}$
Reference: Chakrabarti (1987)						
1	0.25	7.62	36	68.6	68.7	1.00
2	0.25	7.62	36	68.6	70.3	1.03
3	0.25	7.62	36	68.6	71.4	1.04
Reference: Gross and Cheok (1988)						
1A	0.25	7.62	46.7	89.0	116	1.30
1B	0.25	7.62	46.7	89.0	96	1.08
2A	0.25	7.62	46.7	89.0	138	1.55
2B	0.25	7.62	46.7	89.0	148	1.66
3B	0.25	7.62	46.7	89.0	88	0.99
Reference: Brown (1988)						
1	0.251	9.2	48	110.8	180	1.62
2	0.196	8.66	45.2	76.7	120	1.56
3	0.198	9.2	45.2	82.3	82	1.00
9	0.192	8.66	45.2	75.2	79.6	1.06
10	0.197	8.66	45.2	77.1	109.5	1.42
11	0.25	9.2	48	110.4	165.9	1.50
13	0.248	9.2	48	109.5	139.4	1.27
14	0.248	8.66	48	103.1	134.6	1.31
15	0.25	9.2	48	110.4	154.2	1.40
16	0.25	8.66	48	103.9	147.1	1.42
17	0.194	9.2	45.2	80.7	120	1.49
18	0.251	8.66	48	104.3	154.5	1.48
20	0.376	8.66	45	146.5	176.1	1.20
Reference: Sheng et al. (2002)						
1	0.524	9.04	42.75	202.7	365.9	1.80
2	0.524	12.22	42.75	274.1	447.1	1.63
3	0.524	15.41	42.75	345.5	528.5	1.53
4	0.524	9.04	42.75	202.7	358.9	1.77
5	0.524	12.22	42.75	274.1	438.5	1.60
6	0.524	15.41	42.75	345.5	525.4	1.52
7	0.389	9.04	44.20	155.6	257.2	1.65
8	0.389	12.22	44.20	210.3	322.2	1.53
9	0.389	15.41	44.20	265.1	420.1	1.58
10	0.389	9.04	44.20	155.6	252.5	1.62
11	0.389	12.22	44.20	210.3	315.5	1.50
12	0.389	15.41	44.20	265.2	408.2	1.54
13	0.256	9.04	39.86	92.4	114.5	1.24
14	0.256	12.22	39.86	124.9	156.6	1.25
15	0.256	15.41	39.86	157.4	222.5	1.41
16	0.256	9.04	39.86	92.4	112.1	1.21
17	0.256	12.22	39.86	124.9	149.9	1.20
18	0.256	15.41	39.86	157.4	207.0	1.32

Table 2 (continued). Details and Calculated Capacity of Compact Corner Gusset Plates Full Yield Capacity

Spec. No.	t (in.)	L_w (in.)	F_y (ksi)	P_{calc} (k)	P_{exp} (k)	$\frac{P_{exp}}{P_{calc}}$
Reference: Yam and Cheng (1993)						
GP1	0.524	12.22	42.75	274.0	440.1	1.61
GP2	0.386	12.22	44.20	208.8	305.1	1.46
GP3	0.256	12.22	39.86	124.8	167.0	1.34
GP1R	0.524	12.22	42.75	274.0	462.8	1.69
GP2R	0.386	12.22	44.20	208.8	334.6	1.60
GP3R	0.256	12.22	39.86	124.8	177.8	1.42
AP1	0.524	12.22	42.75	274.0	387.0	1.41
AP2	0.386	12.22	44.20	208.8	272.3	1.30
AP3	0.256	12.22	39.86	124.8	163.8	1.31
MP1	0.524	12.22	42.75	274.0	434.9	1.59
MP2	0.386	12.22	44.20	208.8	296.1	1.42
MP3	0.256	12.22	39.86	124.8	162.2	1.30
MP3A	0.256	12.22	39.86	124.8	184.3	1.48
MP3B	0.256	12.22	39.86	124.8	184.7	1.48
Reference: Rabinovitch and Cheng (1993)						
A1	0.367	15.49	65.07	370.3	378.5	1.02
A2	0.243	15.49	64.20	242.3	253.8	1.05
A3	0.367	15.49	65.07	370.3	450.9	1.22
A4	0.243	15.49	64.20	242.3	258.5	1.07
Reference: Nast et al. (1999)						
T2-FE	0.378	15.49	61.45	360.6	444.2	1.23
T2	0.378	15.49	61.45	360.6	380.3	1.05
Reference: Walbridge et al. (1998)						
GP1B1	0.236	15.49	43.48	159.3	154.8	0.97
GP1B3	0.236	15.49	43.48	159.3	155.7	0.98
GP2B7	0.354	15.49	43.48	238.9	290.7	1.22
GP3B11	0.472	15.49	43.48	318.6	403.4	1.27
GP1MC	0.524	12.22	42.75	274.0	466.4	1.70
GP2MC	0.386	12.22	44.20	208.8	302.0	1.45
GP3MC	0.256	12.22	39.86	124.8	160.0	1.28
A2CL2	0.243	15.49	64.20	242.3	246.2	1.02
A4CL4	0.243	15.49	64.20	242.3	252.0	1.04

Table 3. Details and Calculated Capacity of Noncompact Corner Gusset Plates
k = 1.0

Spec. No.	t (in.)	L_w (in.)	l_{avg} (in.)	F_y (ksi)	E (ksi)	P_{calc} (k)	P_{exp} (k)	$\frac{P_{exp}}{P_{calc}}$
Reference: Cheng and Hu (1987)								
C1-Free	0.264	22.36	8.94	73.19	30595	113.5	99.4	0.875
C2-Free	0.122	22.36	8.94	34.78	28565	10.6	27.5	2.60
C3-Free	0.264	22.36	14.50	73.19	30595	43.1	85.5	1.98
C4-Free	0.122	22.36	14.50	34.78	28565	4.0	20.2	5.01
C1-Fixed	0.264	22.36	8.94	73.19	30595	113.5	205.7	1.81
C2-Fixed	0.122	22.36	8.94	34.78	28565	10.6	31.6	2.98
C3-Fixed	0.264	22.36	14.50	73.19	30595	43.1	152.6	3.54
C4-Fixed	0.122	22.36	14.50	34.78	28565	4.0	32.7	8.13
Reference: Yam and Cheng (1993)								
SP1-Free	0.524	18.59	16.33	42.75	30102	206.2	361.4	1.75
SP2-Free	0.386	18.59	16.33	44.20	30479	88.1	227.3	2.58
SP1-Fix	0.524	18.59	16.33	42.75	30102	206.2	396.0	1.92
SP2-Fix	0.386	18.59	16.33	44.20	30479	88.1	332.3	3.77

Table 4. Details and Calculated Capacity of Extended Corner Gusset Plates
k = 0.60

Spec. No.	t (in.)	L_w (in.)	l_1 (in.)	F_y (ksi)	E (ksi)	P_{calc} (k)	P_{exp} (k)	$\frac{P_{exp}}{P_{calc}}$
Reference: Sheng et al. (2002)								
19	0.524	9.04	11.31	42.75	29000	178.7	333.0	1.86
20	0.524	12.22	8.55	42.75	29000	255.0	403.4	1.58
21	0.524	15.41	5.80	42.75	29000	334.2	496.8	1.49
22	0.524	9.04	11.31	42.75	29000	178.7	277.2	1.55
23	0.524	12.22	8.55	42.75	29000	255.0	335.3	1.31
24	0.524	15.14	5.80	42.75	29000	328.3	463.7	1.41
25	0.389	9.04	11.31	44.20	29000	122.8	197.6	1.61
26	0.389	12.22	8.55	44.20	29000	183.7	243.5	1.33
27	0.389	15.14	5.80	44.20	29000	244.7	327.2	1.34
28	0.256	9.04	11.31	39.86	29000	56.5	90.9	1.61
29	0.256	12.22	8.55	39.86	29000	94.2	126.0	1.34
30	0.256	15.14	5.80	39.86	29000	135.9	172.1	1.27
Reference: Rabinovitch and Cheng (1993)								
A5	0.367	15.49	15.20	65.07	29870	186.6	204.1	1.09

Table 5. Details and Calculated Capacity of Single Brace Gusset Plates
k = 0.70

Spec. No.	t (in.)	L_w (in.)	l_1 (in.)	F_y (ksi)	E (ksi)	P_{calc} (k)	P_{exp} (k)	$\frac{P_{exp}}{P_{calc}}$
Reference: Sheng et al. (2002)								
31	0.524	11.31	8.00	42.78	29000	151.1	216.2	1.43
32	0.524	8.55	9.59	42.78	29000	195.0	246.4	1.26
33	0.524	5.80	11.18	42.78	29000	239.7	332.6	1.39
34	0.389	11.31	8.00	44.22	29000	99.7	157.3	1.58
35	0.389	8.55	9.59	44.22	29000	137.2	181.4	1.32
36	0.389	5.80	11.18	44.22	29000	176.8	246.2	1.39
37	0.256	11.31	8.00	39.88	29000	41.8	80.3	1.92
38	0.256	8.55	9.59	39.88	29000	66.8	96.3	1.44
39	0.256	5.80	11.18	39.88	29000	95.8	124.9	1.30

Table 6. Details and Calculated Capacity of Chevron Brace Gusset Plates
k = 0.75

Spec. No.	T (in.)	L_w (in.)	l_1 (in.)	F_y (ksi)	E (ksi)	P_{calc} (k)	P_{exp} (k)	$\frac{P_{exp}}{P_{calc}}$
Reference: Chakrabarti and Richard (1990)								
1	0.472	14.8	9.8	43.3	29000	252	286	1.14
2	0.315	14.8	6.4	40	29000	158	222	1.41
3	0.315	14.8	6.4	43.2	29000	169	264	1.56
4	0.315	14.8	9.8	72.3	29000	168.7	292	1.73
5	0.315	21.6	11.2	44.7	29000	174.1	175	1.01
6	0.394	14.8	9.6	36.8	29000	173	191	1.11
7	0.512	14.8	8.8	46.7	29000	309	429	1.39
8	0.394	14.8	6.0	82.9	29000	400	477	1.19
1-FE	0.472	14.8	9.8	43.3	29000	252	274	1.09
2-FE	0.315	14.8	6.4	40	29000	158	201	1.27
5-FE	0.315	21.6	11.2	44.7	29000	174.1	228	1.31
8-FE	0.394	14.8	6.0	82.9	29000	400	431	1.08
Reference: Astaneh (1992)								
3	0.25	4.96	4.0	36.0	29000	40.8	42.4	1.04

Table 7. Summary of Proposed Effective Length Factors

Gusset Configuration	Effective Length Factor	Buckling Length	$\frac{P_{exp}}{P_{calc}}$
Compact corner	– [a]	– [a]	1.36
Noncompact corner	1.0	l_{avg}	3.08
Extended corner	0.6	l_1	1.45
Single-brace	0.7	l_1	1.45
Chevron	0.75	l_1	1.25

[a]Yielding is the applicable limit state for compact corner gusset plates; therefore, the effective length factor and the buckling length are not applicable.