GUSSET PLATE STRESS

Gusset plates are 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. Gusset plates are also used to connect diagonal members to the chords and vertical members of trusses.

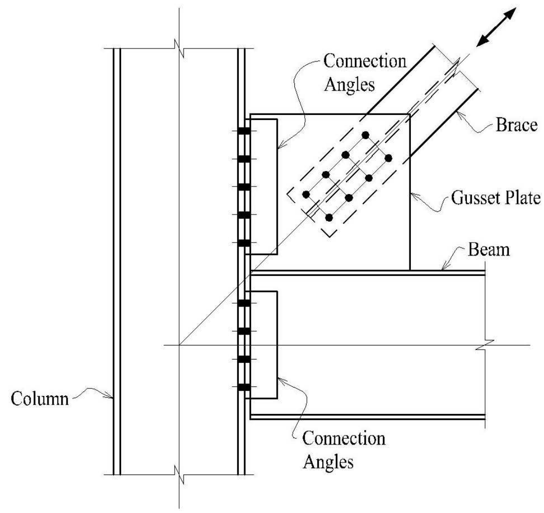


Fig. 1. Vertical brace connection.

A large number of research projects have been dedicated to the stresses in gusset plates. The research includes laboratory tests, finite element models, and theoretical studies. Many different failure modes have been identified, and design methods and specification requirements have been formulated based on the research.

Design information for gusset plates can be found in the AISC *Steel Construction Manual* (AISC, 2005); however, the behavior of gusset plates is very complex and cannot be fully defined by the available design procedures. Engineering judgment is critical in the design process; therefore, it is important that engineers understand the background of the guidelines.

This paper provides a design-based review of the available information on gusset plates and references for engineers who want to study the topic in-depth. Only the documents relevant to the evolution of the current design procedures are presented; however, additional references are listed in the bibliography. Other aspects of gusset plate design, such as stability, calculation if interface loads, and seismic design will be presented in future papers in this series. This paper is organized into three sections: Effective Width, Normal and Shear Stresses, and Combined Stresses.

EFFECTIVE WIDTH

Existing Literature

The first major experimental work on gusset plates was done by Wyss (1923). The stress trajectories were plotted for gusset plate specimens representing a warren truss joint as shown in Figure 2. 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.

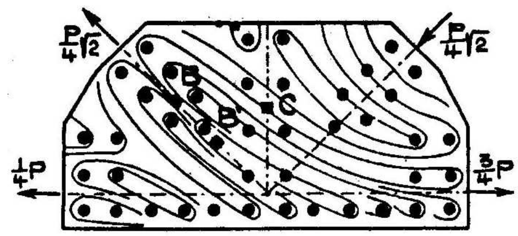


Fig. 2. Stress trajectories by Wyss (1923).

Sandel (1950) conducted a photoelastic stress analysis of a 1/22-scale model of a Warren truss joint. The stress trajectories are shown in Figure 3. He concluded that the normal stress at the end of the bracing members can be calculated more accurately using a stress trajectory angle of 35° instead of the 30° suggested by Wyss.

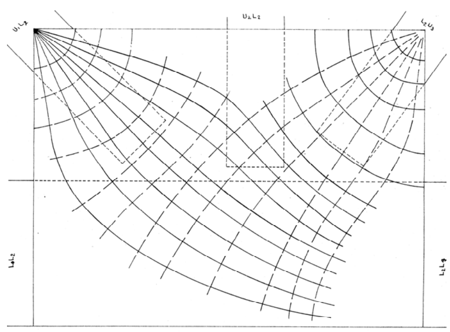


Fig. 3. Stress trajectories by Sandel (1950).

An experimental investigation was carried out by Whitmore (1952) to determine the stress distribution in gusset plates. The tests were conducted on 1/8 in. aluminum gusset plates with a yield strength of 39 ksi and a modulus of elasticity of 10,000 ksi. The specimen was a 1/4-scale model of a warren truss joint with double gusset plates. Strain gages were mounted on the gusset plates, and the data was used to plot stress trajectories. The stress trajectories are shown in Figure 4. These plots confirmed that the maximum normal stress was at the end of the members and the stress trajectories were along approximately 30° lines with the connected member.

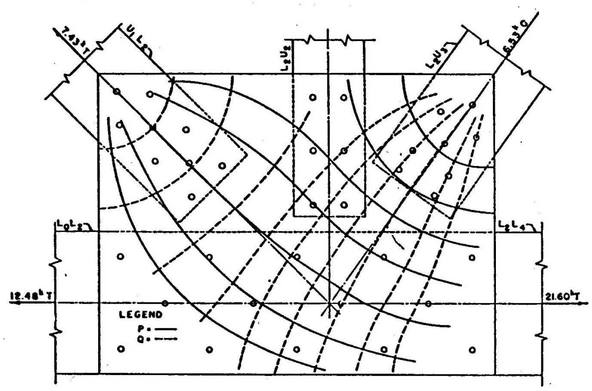


Fig. 4. Stress trajectories by Whitmore (1952).

Irvan (1957) conducted tests on a model of a pratt truss joint with double gusset plates. The gusset plates were ⅛ in. thick aluminum, with a yield strength of 35 ksi and a modulus of elasticity of 10,000 ksi. Data from strain gages was used to plot the tension, compression, and shear stresses in the gusset plate. The stress trajectories are shown in Figure 5. He proposed a method similar to Whitmore's for calculating the normal stress at the end of the truss members. The difference was that the 30° lines should project from the center of gravity of the rivet group instead of the outside fasteners on the first row.

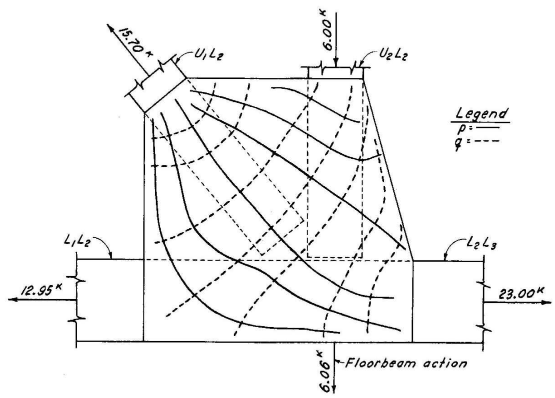


Fig. 5. Stress trajectories by Irvan (1957).

Chesson and Munse (1963) tested thirty riveted and bolted truss connections with gusset plates. The specimens were loaded in tension until one of the components failed. Of the ten specimens that failed in the gusset plate, two failure modes were identified: net section fracture and splitting of the plate down one line of fasteners. The authors recommended that the normal stress at the end of bracing members be calculated using a stress trajectory angle of 22° for plates with punched holes and 25° for plates with drilled holes.

Yamamoto, Akiyama, and Okumura (1985) investigated the stress distribution of eight Warren and Pratt type truss joints with double gusset plates. Test specimens were made of 8-mm-thick (0.315-in.-thick) gusset plates. They plotted the stress distribution using data from strain gages mounted on the gusset plates. The researchers used the finite element method to perform an inelastic analysis on the plates. They found that "the plastic region, which appears in the inner part of the gusset plate at the earlier loading stage, develops toward the outer part with the load increasing." Using the results of "numerical evaluations of a great variety of bolt arrangements", the researchers proposed a method similar to Whitmore's for calculating the normal stress at the end of the truss members, except that they recommended using a stress trajectory of 22° instead of 30°.

Dietrich (1999) presented the results of six cyclic tests of ½-scale double gusset plate connections that were representative of the connections for the San Francisco-Oakland Bay Bridge. The specimens were ¼-in. and ⅜-in. plates of A36 steel. The brace members connected to the gusset plates were loaded with axial load and moment. Strain gages were mounted on the plates at the presumed critical section. The failure mode was fracture along the Whitmore effective width. The following interaction equation was proposed to determine the ultimate compression and moment capacity on the effective width of gusset plates,

$$\left(\frac{P}{P_y}\right)^{1.25} + \frac{M}{M_p} \leq 1.0 \tag{1}$$

where

P = applied compression force
M = applied moment
P_y = axial yield load based in the Whitmore width
M_p = plastic moment capacity based on the Whitmore width

Current Design Practice

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 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 6 for various connection configurations.

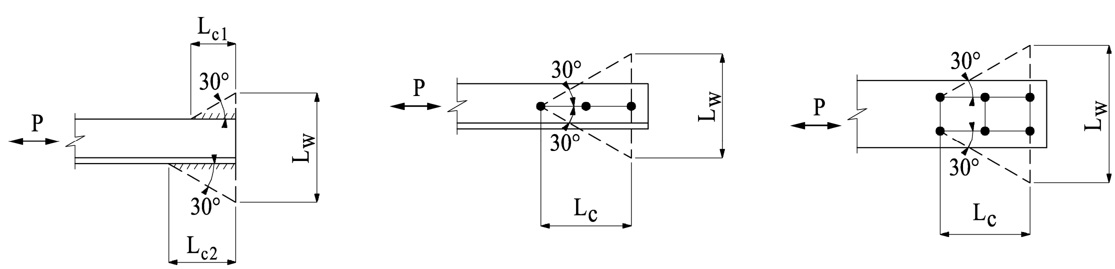


Fig. 6. Effective width for various connection configurations.

When calculating the effective width, it is important to remember that the usable width does not extend beyond the boundaries of the plate. Figure 7 shows two cases where the Whitmore width is only partially effective. For bolted members, the effective width extends across at least one bolt hole; however, it is standard practice to use the gross area in the calculations. While this is true for bolt holes, heavy bracing connections sometimes require hand holes to be cut in the gusset plate for bolt installation. If practical, these should be located outside the Whitmore zone; otherwise, the hole width should be subtracted from the Whitmore width.

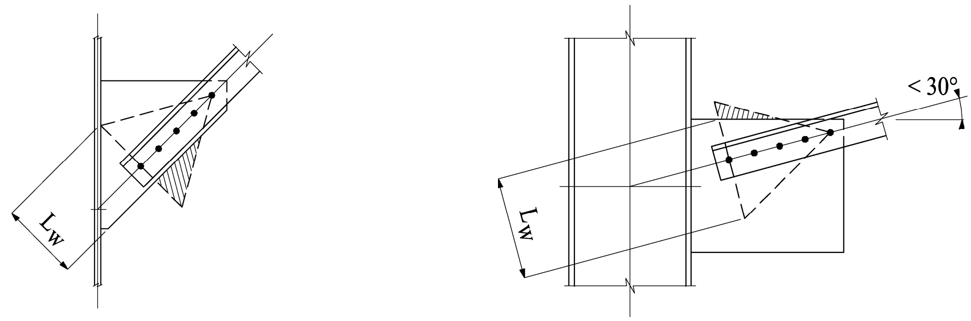


Fig. 7. Cases where the Whitmore width is only partially effective.

Discussion

The design procedure used today was developed using scale models and very thin plates, and the research shows that it gives a crude approximation of the true stress distribution. Gusset plates in some industrial structures and long-span trusses can be several inches thick, and transfer thousands of kips. Engineers should know the background and limitations of the design procedure so judgment can be used. However, the effective width approach has proven adequate for design. In addition to the research projects discussed above, the 30° stress trajectories have been verified by the research of Lavis (1967), Rabern (1983), Chakrabarti (1983), Bjorhovde and Chakrabarti (1985), Gross and Cheok (1988), and Girard, Picard and Fafard (1995).

In some cases, Special Concentrically Braced Frames are arranged so the brace buckles in the plane of the gusset plate. In this case, the connection must be designed to transfer the expected moment capacity of the brace into the gusset plate, and Dietrich's model can be used to predict the capacity of the Whitmore section.

Although the effective width approach is adequate to determine the local stress distribution, it can be very unconservative when used to design gusset plates in compression. Gusset plates in compression will be discussed in part 2 of this paper.

NORMAL AND SHEAR STRESSES

Existing Literature

In Wyss' tests (Wyss, 1923), the chord members were spliced at the joint, and the normal and shear stresses were plotted at the vertical section of the joint where the vertical web member was riveted to the gusset plate. Stress Diagram d of Figure 8 shows the normal stresses due to the combination of the net horizontal force and the moment. The moment is present because the net horizontal force is below the center of the gusset plate. The curvature of the line illustrates the deviation from the straight-line beam equation. Large stress concentrations at the rivet holes are shown in Diagram c. The gross and net shear stresses are shown in Diagrams a and b respectively.

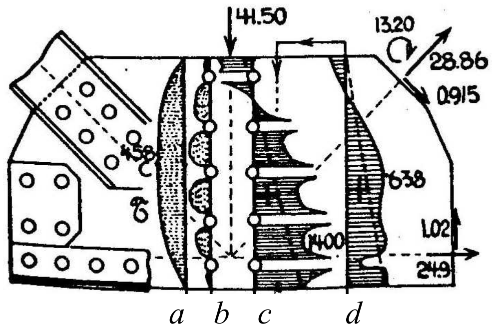


Fig. 8. Normal and shear stresses by Wyss (1923).

When discussing the calculation of the elastic bending stresses in gusset plates, Shedd (1934) noted, "these calculations assume the applicability of the beam formula to a beam which is deeper than it is long and in which the loading is quite unlike that in the usual beam...At present the method used seems to be the only practicable one and appears to give satisfactory results for design purposes." An example from his book is shown in Figure 9, which shows the calculated stress distribution in a gusset plate with a discontinuous bottom chord member at the panel point of a truss.

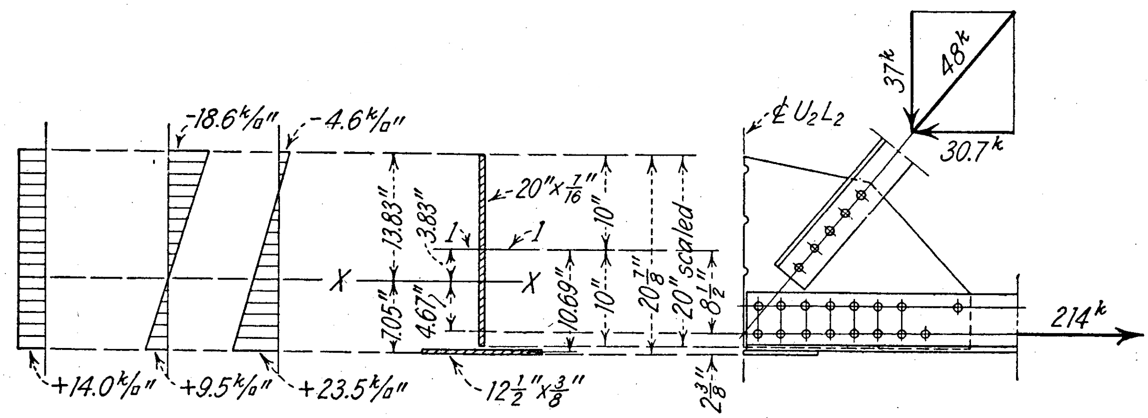
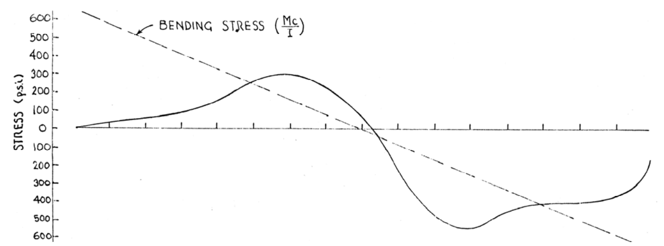


Fig. 9. Calculated stress distribution in a gusset plate (Shedd, 1934).

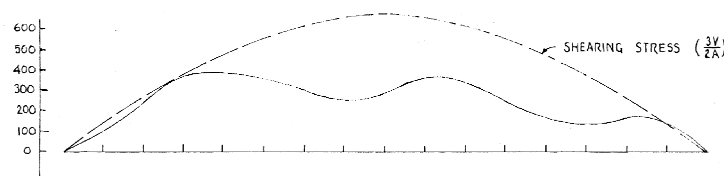
Rust (1938) published the results of a photoelastic study on the transfer of stress in gusset plates. He proposed a qualitative set of design rules and noted that a "generalized stress solution has not been found." He wrote "If an unsupported edge is stressed in compression, the edge will buckle before failure, throwing more moment and direct stress into the interior of the plate." In a discussion to the paper by Rust, Grinter noted that a simple elastic analysis using beam theory overestimated the observed stress by 20% to 30%.

Perna (1941) tested a small-scale photoelastic model of a Pratt truss joint. He found that the stress distribution differed greatly from the stresses calculated using the beam equations. The tests showed that the normal stresses at the edge of the plate were much smaller than the calculated stresses, but at the interior of the plate, they were much greater than the calculated stresses. Although the maximum test stress did not occur in the same location as the maximum calculated stress, the author noted that the beam equations appear to be conservative because the maximum calculated stress exceeded the maximum test stress.

Sandel's photoelastic studies (Sandel, 1950) of a 1/22-scale model of a Warren truss joint showed that the use of the beam equations to determine the stresses at critical sections in the gusset is "highly incorrect" but conservative. He suggested that the shear stress be calculated using a plastic stress distribution instead of the parabolic distribution dictated by beam theory. The bending stress distribution is shown in Figure 10a and the shear stresses are shown in Figure 10b. The solid lines show the experimental stresses, and the dashed lines represent the distribution according to the beam equations.



a. Bending stress.



b. Shear stress.

Fig. 10. Bending and shear stresses by Sandel (1950).

Whitmore (1952) plotted the distribution of bending and shear stresses at the critical section of the plates in his original study. He concluded that the use of simple beam formulas to calculate the stresses led to erroneous results. The maximum experimental bending stresses were slightly lower than the maximum calculated stresses, but they occurred in a different location as shown in Figure 11. The maximum calculated shear stress was about 20% higher than the maximum experimental stress.

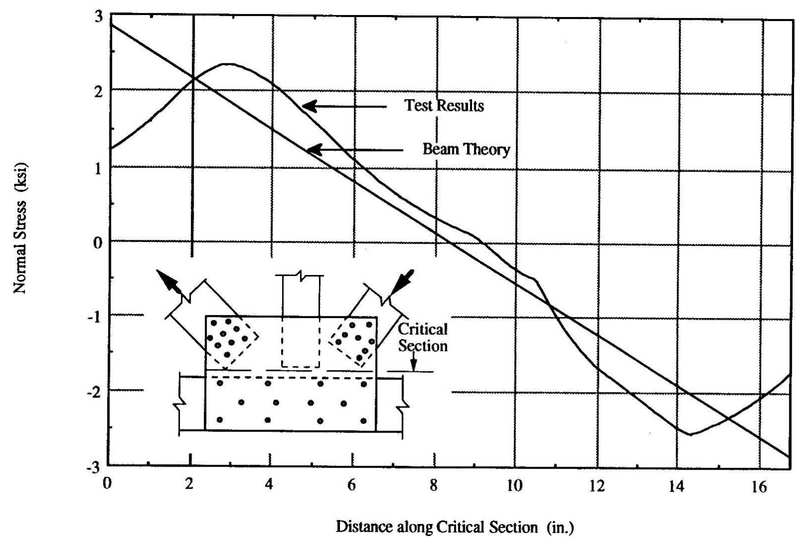


Fig. 11. Bending stress by Whitmore (as re-drawn by Yam and Cheng, 1993).

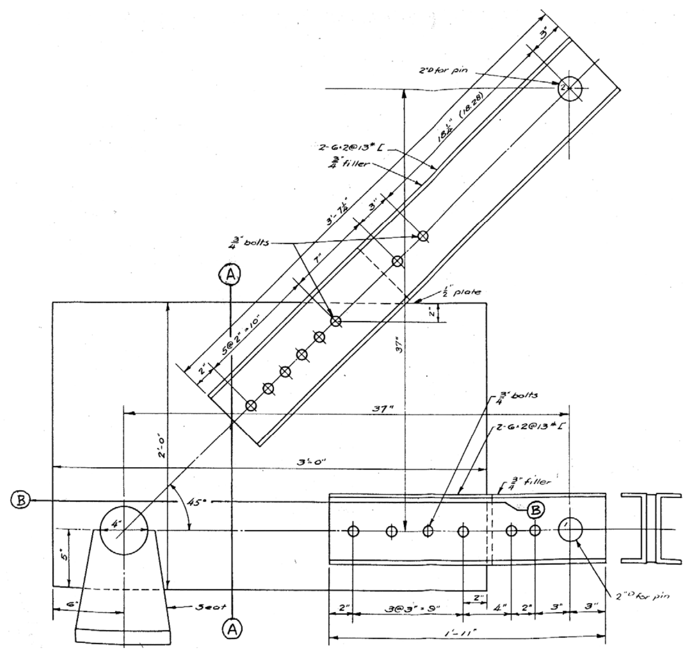
Experiments were carried out by Sheridan (1953) on 21 rectangular plates loaded in tension. All of the plates were 0.507 in. thick and loaded only in the elastic range. The modulus of elasticity was 29,900 ksi and poisson's ratio was 0.272. In some of the tests, the plates were loaded with an in-plane eccentricity to determine the stress distribution with combined axial load and bending moment. Strain gages were mounted on the gusset plate and the data was used to plot the normal stress distribution in the plate. He concluded that, for the specimens with small eccentricity, the experimental stresses "differed greatly" from the stresses calculated using beam equations. The calculated stresses approached the experimental stresses as the eccentricity increased. For design purposes, it was recommended that the stresses calculated using the beam equations be multiplied by a correction factor that depends on the load eccentricity.

Sheridan (1953) also tested a gusset plate similar to those used at the ends of bridge trusses, with the bottom chord member and the diagonal member intersecting at a 4 in. diameter support pin. The test arrangement is shown in Figure 12a. The plate was 1/2 in. thick grade SAE 1020. The diagonal and bottom chord members were 6 in. deep double-channels. The diagonal was loaded in compression and the bottom chord was in tension. The reaction pin carried vertical load only. The applied forces were kept relatively small to minimize the possibility of plate buckling and to ensure the stresses in the plate remained elastic. Data from strain gages mounted on the gusset plate was used to plot the normal stress distribution.

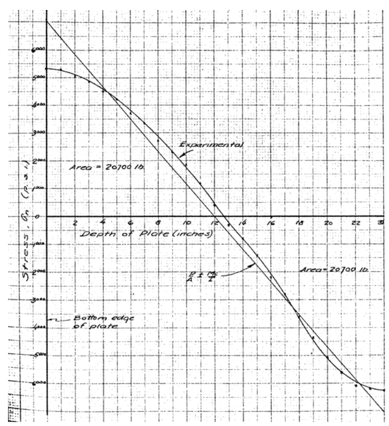
Stresses were reported on a vertical section of the plate, between the truss web members and the reaction pin (8.5 in. from the support pin.). The net horizontal load on the section was zero; therefore, the only normal stresses were due to bending. Experimental stresses on the vertical section of the plate agreed well with the stresses calculated with beam theory. The normal and shear stresses are shown in Figures 12b and 12c, respectively.

Stresses were also reported on a horizontal section of the plate, 3 in. above the center of the support pin. The normal and shear stresses are shown in Figures 12d and 12e, respectively.

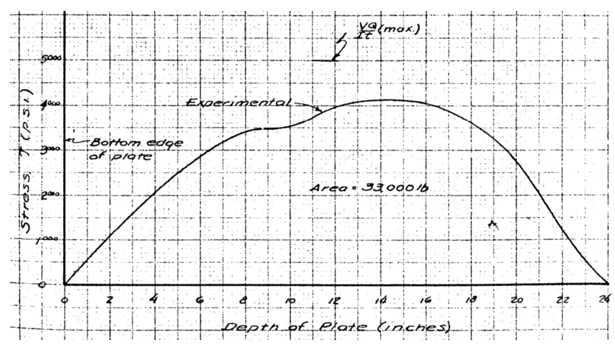
The calculated stresses do not agree with the measurements. Sheridan wrote that it is probable that the beam equations are “not satisfactory for use” on the horizontal section. The closeness of the horizontal section to the reaction pin “create a pattern of stresses which are influenced much by stress concentrations.”



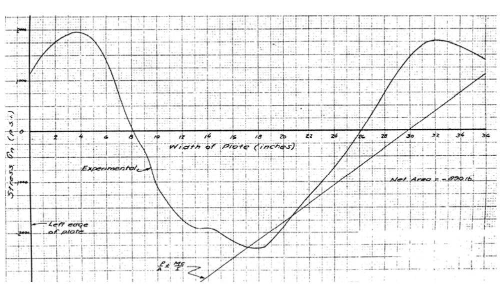
a. Test arrangement



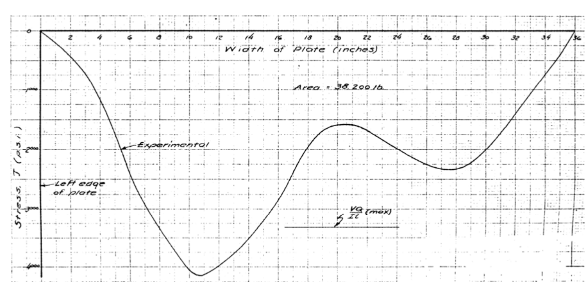
b. Normal stress at section A-A



c. Shear stress at section A-A



d. Normal stress at section B-B



e. Shear stress at section B-B

Fig. 12. Tests by Sheridan (1953).

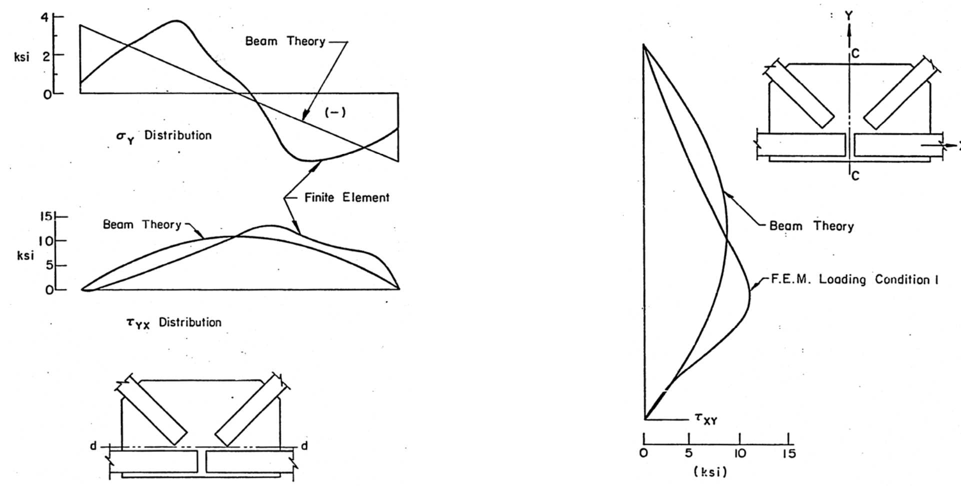
Irvan (1957) came to a conclusion that was similar to Whitmore's with respect to the calculation of stresses at the critical sections. He wrote: "The assumption that all of the beam formulas apply in calculating primary stress distribution on any cross-section (either vertical or horizontal) is considerably in error".

Hardin's test specimen (Hardin, 1958) was similar to Irvan's except that the chord was spliced within the joint. The gusset plate was used to carry tensile loads from the spliced chord members in addition to the loads from the truss web members. The plates were 3/16" thick and had the same material properties as Irvan's test. As expected, large tension stresses developed in the gusset plate between the spliced chord members.

Lavis (1967) used the finite element method to investigate the elastic stress distribution in gusset plates. He compared the finite element results to Whitmore's test and the results of a photoelastic model. His results compared well with Whitmore's. He noted that the use of the beam equations "appears to be conservative."

Vasarhelyi (1971) published the results of experiments on a gusset plate model. The tests were conducted on 1/4 in. thick gusset plates of A36 steel. The specimen was a warren truss joint with double gusset plates. Strain gages were mounted on the gusset plate. The data from the strain gages was used to plot the stress distribution. He also conducted photoelastic tests and analytical studies of the stress distribution. Vasarhelyi concluded, "The various analytical methods indicate that the maximums of stress found in a gusset by various simplified methods are only slightly different; the major deviations are in the locations of those maximums." He also wrote, "The present elementary analysis appears to be adequate for most cases."

Struik (1972) analyzed gusset plates using an elastic-plastic finite element program, and plotted the stresses at the critical sections as shown in Figure 13. Figure 13a shows the results at a horizontal section of the plate, along with the theoretical beam stresses. Figure 13b shows the shear stress distribution at a vertical section as well as the calculated elastic shear stress from beam theory. The results of his studies indicated that current design procedures which utilize the beam equations produced "substantial variations in the factor of safety." He wrote "the finite element analysis differs significantly from beam theory. However, the difference is not necessarily an unsafe one. None of the stresses exceeded the maximum values predicted by beam theory by a significant amount."



a. Bending and shear at horizontal plane *b. Shear at vertical plane*

Fig. 13. Bending and shear stresses from Struik (1972).

Yamamoto, Akiyama, and Okumura (1985) measured the stresses at the critical sections of the plates described above and found that the maximum elastic shear stress in the plates can be closely approximated using the beam equations.

Astaneh (1992) conducted three experiments on gusset plate specimens representing a brace-to-beam connection for a chevron-bracing system subjected to cyclic loading. ¼-in.-thick gusset plates of A36 steel were welded to the beam flange. Both braces were double C4x7.25 and were bolted to the plate at 45° angles. One of the specimens failed by shear yielding and Astaneh recommended that the plastic stress distribution be used to calculate the horizontal shear capacity of the plate.

Current Design Practice

Gusset plates are currently designed so the stresses on any cross section of the plate do not exceed the available stresses. The selection of the most highly stressed (critical) section is at the discretion of the designer and is based on judgment and experience. Gusset plates are fabricated in many different configurations, but the most common ones are shown in Figure 14, where the critical sections that typically control the design are noted with dashed lines.

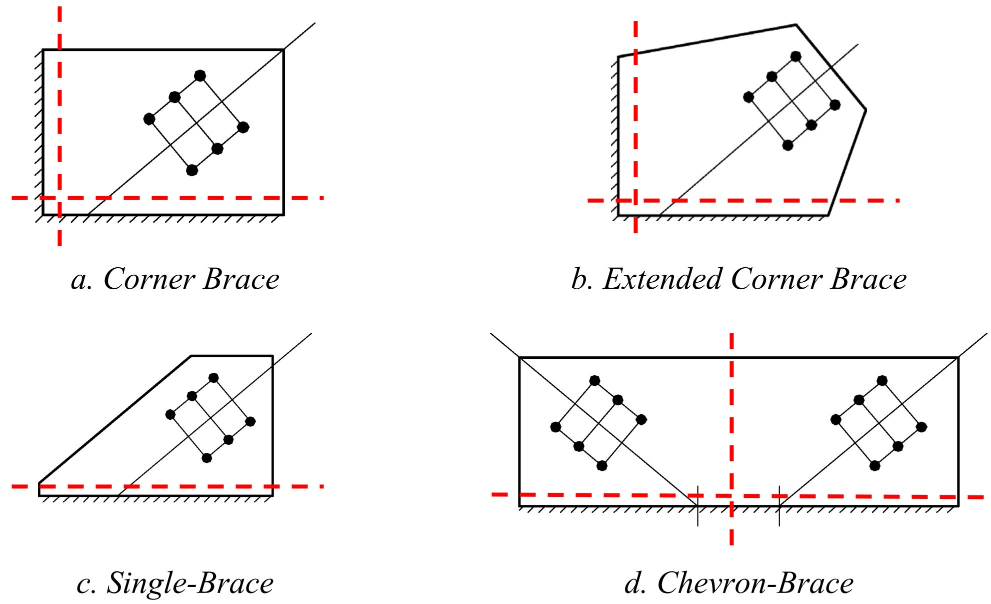


Fig. 14. Location of critical section.

The equation for the maximum normal stress is

$$f_a = \frac{P}{A} + \frac{M}{S} \tag{2}$$

where

P = normal force
A = cross sectional area of the plate
M = applied moment
S = section modulus of the plate.

The plastic shear stress is used in design. The equation is

$$f_v = \frac{V}{A} \tag{3}$$

where

V = shear force
A = cross sectional area of the plate

Discussion

Shear and bending stresses in deep beams deviate significantly from the elastic stress distribution calculated using beam theory (Ahmed, Idris and Uddin, 1996; Barry and Ainso, 1983). Figure 15 shows a plot of the bending stress in a beam with a length-to-depth ratio of 1.0. The straight line is the theoretical stress and the curved line is from an elastic finite element model. The actual stress is 90% higher than the theoretical stress. The designer should be aware of this discrepancy for special cases such as fatigue loading; however, the calculation of theoretical beam stresses has been shown to be adequate for most gusset plate design applications.

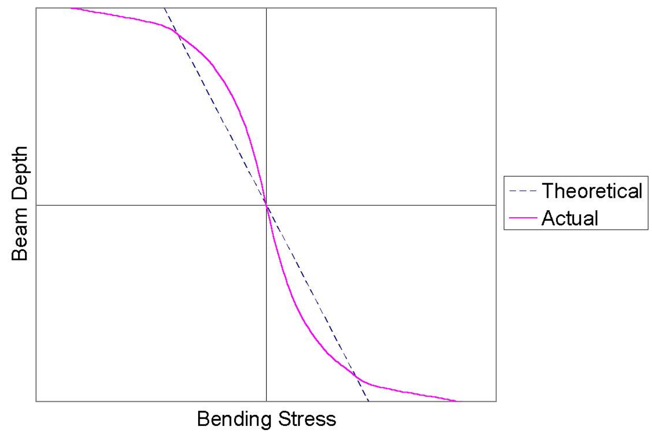


Fig. 15. Bending stress in a deep beam with a length-to depth ratio of 1.0.

Residual stresses are self-equilibrating stresses that are built into members as a result of manufacturing and fabrication operations. They exist in almost all structural steel due to uneven cooling of the material after hot rolling, welding, and flame cutting. Residual stresses can also be caused by cold bending and other fabrication operations. Welded and flame-cut members have tensile residual stresses at the location of the heat input. The magnitude of the tension residual stresses is typically at least equal to the yield stress of the material and is "generally around 60 to 70 ksi, regardless of the original material properties." (Bjorhovde, Engstrom, Griffis, Kloiber and Malley, 2001). Figure 16 shows a typical residual stress pattern for a plate with flame-cut edges.

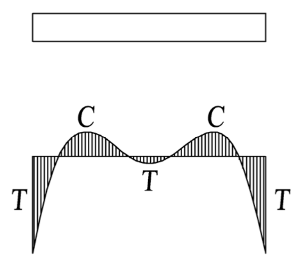


Fig. 16. Residual stress pattern for a plate with flame-cut edges.

Checking gusset plates for the elastic stress distribution in hopes that the stresses will not exceed the yield stress is futile, because, if the plate has flame-cut edges, it has yielded under the residual stresses before any external loading is applied. Although designing for the elastic stress distribution in gusset plates has provided safe designs in the past, the presence of residual stresses and inaccuracies of the design model make it difficult to predict the actual stress distribution in the gusset plate.

From a designer's perspective, the goal is to use the simplest procedure available that provides a safe and economical design. It is standard practice to calculate the shear capacity of gusset plates based on the plastic stress distribution. Because strength design is now being used for steel members and connections, it seems appropriate to design gusset plates using the plastic capacity in bending.

If the plastic capacity is used for bending of gusset plates, the plate must have sufficient rotational capacity to allow the stresses to redistribute without fracture or buckling. Schreiner (1935), and Jensen and Crispen (1938) tested plates with relatively low depth-to-thickness ratios (≤ 10) welded to a support, and determined that the plates can reach their plastic capacity if the weld is adequately sized to resist the moment. More recently, tests on single plate connections by Patrick, Thomas and Bennetts (1986), and Metzger (2006) revealed that the plastic moment capacity of the plate can be used in design. Additionally, gusset plates tested by Dietrich (1999), which were subjected to axial loading and moments, "were capable of supporting additional loads after buckling occurred"; therefore, edge buckling was not considered a primary failure mode. In the upcoming AISC Design Guide 28-Bracing Connections, the plastic modulus is used to calculate the strength of gusset plates.

COMBINED STRESSES

Existing Literature

Several theories have been proposed to predict the behavior of materials under multiaxial states of stress. von Mises' criterion is considered the most accurate for predicting the initiation of yield in ductile metals when loaded by various combinations of normal stress and shear stress. For plane stress (Figure 17), von Mises' equation reduces to,

$$\sigma_e = \sqrt{\sigma_x^2 + \sigma_y^2 - \sigma_x \sigma_y + 3\tau^2} \tag{4}$$

where

- σ_e = effective stress that is compared to the tension yield stress of the material
- σ_x = applied stress in the x-direction (tension positive)
- σ_y = applied stress in the y-direction (tension positive)
- τ = applied shear stress

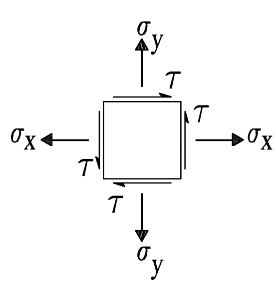


Fig. 17. Plane stress.

If $\sigma_e > F_y$, the material has exceeded its yield capacity.

The plastic capacity of a rectangular cross section subjected to moment about one axis, axial load, and shear can be predicted with (Neal, 1961),

$$\frac{M}{M_p} + \left(\frac{P}{P_y}\right)^2 + \frac{\left(\frac{V}{V_p}\right)^4}{1 - \left(\frac{P}{P_y}\right)^2} \le 1.0 \tag{5}$$

where

M = applied moment
M_p = plastic moment capacity
P = applied axial load
P_y = axial yield load
V = applied shear
V_p = plastic shear capacity

With the objective of keeping the critical members in the elastic range during the Safety Evaluation Earthquake, Caltrans (Reno and Duan, 1997) published design criteria for the seismic retrofit of the San Francisco-Oakland Bay Bridge. Based on von Mises' yield criterion, the following equations were adopted to predict the initial yield capacity for plates subjected to moment, shear, and axial loads

$$\frac{M}{M_r} + \frac{P}{P_y} \le 1.0 \tag{6}$$

$$\left(\frac{P}{P_y}\right)^2 + \left(\frac{V}{V_n}\right)^2 \le 1.0 \tag{7}$$

where

M_r = elastic moment capacity
V_n = nominal shear strength
= $0.4F_yA$ for flexural shear
= $0.6F_yA$ for uniform shear

Astaneh (1998) summarized the previous research and relevant code provisions for the seismic design of gusset plates. He recommended that the critical section of gusset plates be checked against the combined actions of normal force, moment, and shear force using,

$$\frac{M}{M_p} + \left(\frac{P}{P_y}\right)^2 + \left(\frac{V}{V_p}\right)^4 \leq 1.0 \tag{8}$$

Current Design Practice
Elastic stress distributions are currently assumed for gusset plate design. Therefore, the bending and shear stresses are at their maximums at different locations along the cross section, and the normal stress and shear stress are usually considered separately. When axial loads are combined with shear, von Mises criterion or empirical interaction equations (Goel, 1986) are commonly applied.

Discussion
The research discussed in this paper shows that it is very difficult to predict the location of maximum bending stress along the critical section. If the exact stresses at a point were known, von Mises' criterion could only be used to predict the material's first yield load. It is not necessarily a good predictor of the strength. Although the Caltrans design criteria was based on von Mises' criterion, the main objective was to ensure that the stresses remained in the elastic range for the Safety Evaluation Earthquake. If the plastic capacity is used to design gusset plates, the inherent assumption is that gusset plate yielding will allow the stresses to redistribute. Therefore, von Mises first-yield criterion is not appropriate, and one of the plastic interaction equations should be used.

Also in favor of the plastic interaction equations is the fact that the beam equations lead to erroneous results for some plate geometries. In Figure 18, a simple hanger connection is shown where, if $a = b$, the gusset plate is subjected to a uniform tension stress. Figure 19 shows a plot of the normalized nominal capacity versus the b/a ratio. The normalized nominal capacity is

$$\frac{P_n}{2atF_y} \tag{9}$$

where

P_n = nominal capacity
a = plate dimension as shown in Figure 18
= half plate width for concentrically loaded case
t = plate thickness
F_y = yield strength

If a remains constant and b increases, it is intuitive that that the strength of the gusset plate will increase; however, the dashed line in Figure 19 shows that the beam equation (Equation 2) predicts a decrease in strength in the range of $1 < b/a < 5$. For the case where $b = 2a$, the plate is 50% wider than if $b = a$, but the beam equation predicts a strength of only 75% of the capacity. On the other hand, the plastic interaction equation (Equation 5), shown in Figure

19 by the solid line, conforms to the expected result—the strength increases as the plate width increases.

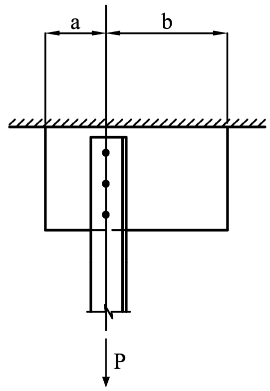


Fig. 18. Gusset plate connection with axial load and moment.

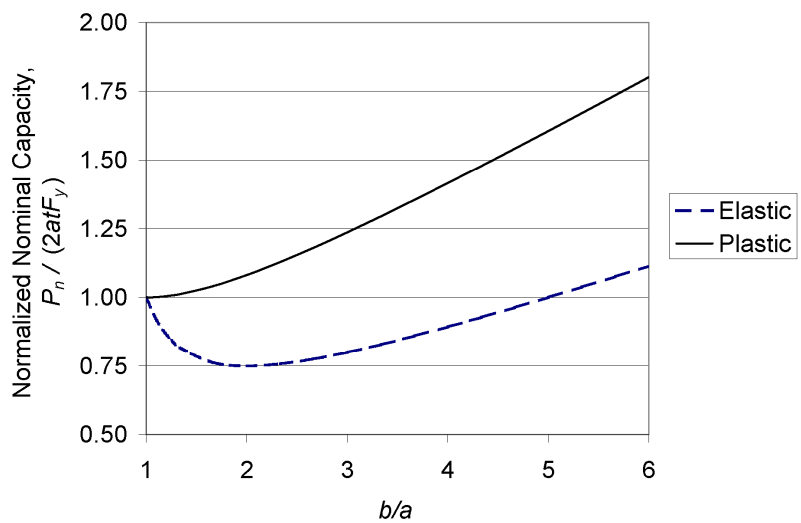


Fig. 19. Elastic versus plastic design for gusset plates with axial load and moment.

CONCLUSIONS

To provide background to the design guidelines for gusset plates, the existing research on stress distribution was reviewed. Current design practices and the future direction of gusset plate design were summarized. Knowledge of the internal stress distribution is essential in the design of gusset plates; however, this is only one of many important topics. The remaining papers in this series will focus on other aspects of gusset plate behavior, such as stability, calculation of interface loads, and seismic design.

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