The Chevron Effect Further Demystified through the Lower Bound Theorem

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ABSTRACT

The transfer of brace forces at braced frame connections in V, inverted-V, and X brace configurations is discussed. Force transfer under these configurations considers both wide-flange (WF) and rectangular hollow structural sections (HSS) for the horizontal beam or strut. The analogous condition of truss web-to-chord connections using gusset plates is discussed. Analogies are expanded to address the condition of beam-to-column brace connections using a common gusset plate to receive the horizontal beam/strut and braces above and below the beam/strut. The potential to redistribute forces using the assumptions of the lower bound theorem is investigated. Additionally, the transposition of shear forces to axial forces in connections at X-brace configurations is illustrated. In conclusion, several methods can address the transfer of brace forces in gusset plate connections. Shear forces in the beam/strut, truss chord, or column members for the various conditions can be reduced by redistributing forces to properly proportioned gusset plates using the assumptions of the lower bound theorem. A reduction in shear forces in the primary members can effectively avoid localized member reinforcement, such as doubler plates or upsizing of members.

Keywords: chevron effect, braced frame connections, lower bound theorem, Uniform Force Method.

INTRODUCTION

Several methods have been presented in recent years to address the force transfer at the intersection of beams and braces in V and inverted-V (chevron) braced frame configurations (Fortney and Thornton, 2015, 2017; Sabelli et al., 2021; Sabelli and Saxey, 2023). Traditionally, the methodology used at V and inverted-V configurations has been applied to super-X (also known as two-story X) configurations by assuming the super-X has a V configuration above the beam and an inverted-V below the beam (see Figure 1). Differences between the behaviors are discussed herein. Alternative methods for addressing the force transfer of each condition are presented.

V AND INVERTED-V BRACE CONNECTIONS

Figure 2 illustrates a free-body diagram (FBD) of a typical inverted-V connection where the story shear A_b is resisted by two brace forces *P,* one in compression and the other in tension. The associated directional forces are represented with *V* and *H* for vertical and horizontal force components, respectively. Previous work has shown that concentrically detailed braces in V and inverted-V configurations produce shears in the beam and gusset plate assembly between the brace attachment points, a phenomenon known as the "chevron effect" (Fortney and Thornton, 2015). Under an elastic analysis, a portion of the shear resulting from the vertical components of the brace force, *V*, as illustrated in Figure 2, is distributed to the beam based on the relative stiffness of the beam to the stiffness of the beam and gusset-plate assembly. The remaining portion of the shear is transferred within the gusset plate. Gravity load effects are neglected herein to simplify the discussion.

Axial shortening and lengthening of the braces in compression and tension results in a story drift and a localized rotation in the beam at the interface with the brace connections. Assumed force distributions and neglect of incidental forces associated with frame deformations in connection design are made possible by implementing the lower-bound theorem (LBT). The LTB is discussed in numerous texts addressing plastic solid mechanics (e.g., Bower, 2010; Barker and Puckett, 2021) and has the following three fundamental attributes as it relates to steel connection design:

1. An assumed load path that satisfies static equilibrium.
2. Sufficient capacity for all limit states associated with the assumed load path.
3. Adequate ductility to allow for force redistribution to the assumed load path (i.e., detailing for inelastic behavior).

Traditionally, steel connection design is performed through a force-based approach using the LBT rather than

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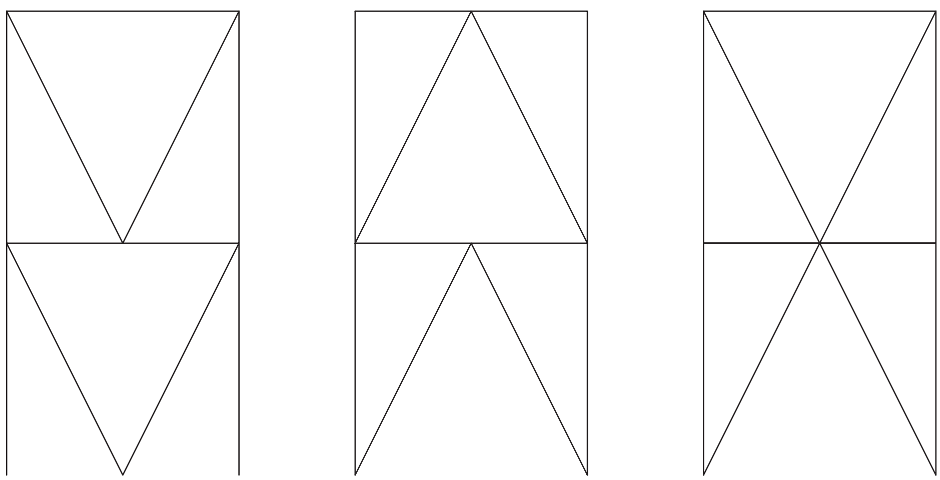
a stiffness-based approach. In some cases, certain rules or limitations may be applied to ensure acceptable ductility. The Uniform Force Method (UFM) for the design of braced-frame connections, as presented in the AISC *Steel Construction Manual* (2023), hereafter referred to as the AISC *Manual*, is an example of such a force-based approach. The fundamental assumption of ductility under this method allows for the redistribution of forces, such as that of redistributed shear forces under Special Case 2 as illustrated in the AISC *Manual*. By applying the LBT to V and inverted-V connections, it can be recognized that yielding within the beam is not detrimental to the desired behavior, or performance, provided all limit states associated with an assumed load path are adequately addressed.

Two fundamental force transfer models bound the required shear the beam must carry in a concentric V or inverted-V brace configuration. The first assumes that each brace's axial force is transferred into and out of the beam in its entirety. The second assumes a continuous gusset transfers the entirety of the vertical shear such that only horizontal components of brace forces need to be transferred into the beam. The first approach effectively discounts any contribution of the gusset in resisting the vertical shear and is consistent with the required force transfer in a discontinuous gusset condition as illustrated in Figure 3. This approach is common where brace angles measured from horizontal are shallow and/or at deep beams. In these conditions, the use of separate gusset plates for each brace-to-beam connection can avoid an excessively long, continuous gusset plate. Under this force transfer assumption, the geometry of the welds for each gusset plate can be balanced about each workline to avoid any eccentricity on the weld. Alternatively, the weld geometry can deviate from a concentric condition provided the moments due to any eccentricity are considered.

Where gusset plates are made continuous between braces, the opportunity exists to utilize the vertical shear capacity of the gusset plate to reduce the vertical shear from



(a) Stacked V *(b) Stacked inverted V* *(c) Super X*

Fig. 1. Typical braced-frame configurations.

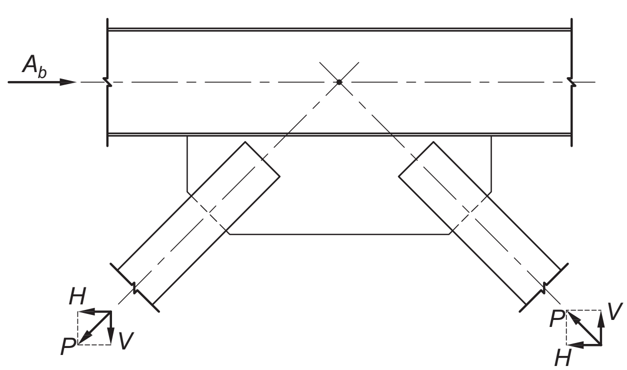


Fig. 2. Inverted-V brace configuration force transfer at connection.

the braces theoretically transferred to the beam. Figure 4 shows a force distribution generally consistent with Special Case 2 of the UFM, whereby a portion of the vertical shear, ΔV_b, can be redistributed to the gusset plate to reduce the shear in the beam as long as the resulting force actions—namely, M_g—are considered to provide equilibrium. At one extreme, ΔV_b can be set equal to V, thereby eliminating assumed shear to the beam due to the vertical component of the brace forces as long as the connection between the gusset plate and beam accounts for the horizontal shear, $2H$, and the moment $M_g = \Delta V_b L_{wl}$, which, in this case, becomes equal to $2Hd_b/2 = A_b d_b/2$. Though nomenclature differs, these forces are consistent with those illustrated in Figures 4, 5, and 6 of AISC Design Guide 29, assuming braces with balanced forces at equal angles of inclination (Muir and Thornton, 2014). As noted by Fortney and Thornton (2015), the force distribution becomes more complicated with the addition of gravity loads, unequal brace angles, and/or unbalanced loads, though the general theory remains unchanged.

As an example, assume the condition shown in Figure 4 with the following attributes where all loads are assumed factored using load and resistance factor design (LRFD):

Beam = W18×40 ASTM A572/A572M Gr. 50 (ASTM, 2021)
Braces = HSS8×8×⅜ oriented at 45° angles from horizontal ASTM A500/A500M Gr. C (ASTM, 2023)
Gusset plate thickness = ½ in. (ASTM A572/A572M Gr. 50)
$L_g = 50$ in.
$d_g = 14$ in.
$A_b = 400$ kips

Therefore, $P = 283$ kips and $H = V = 200$ kips.

The available LRFD shear strength of the beam is determined using AISC *Specification* Equation G2-1:

$$\begin{aligned}\phi R_n &= \phi 0.6 F_y A_w C_{v1} \qquad (1)\\ &= 1.0(0.6)(50\text{ ksi})(0.315\text{ in.})(17.9\text{ in.})(1.0)\\ &= 169\text{ kips}\end{aligned}$$

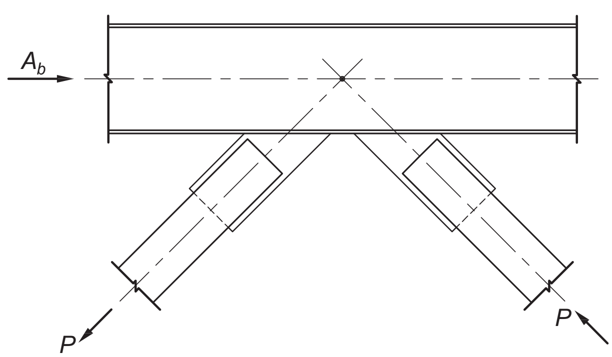
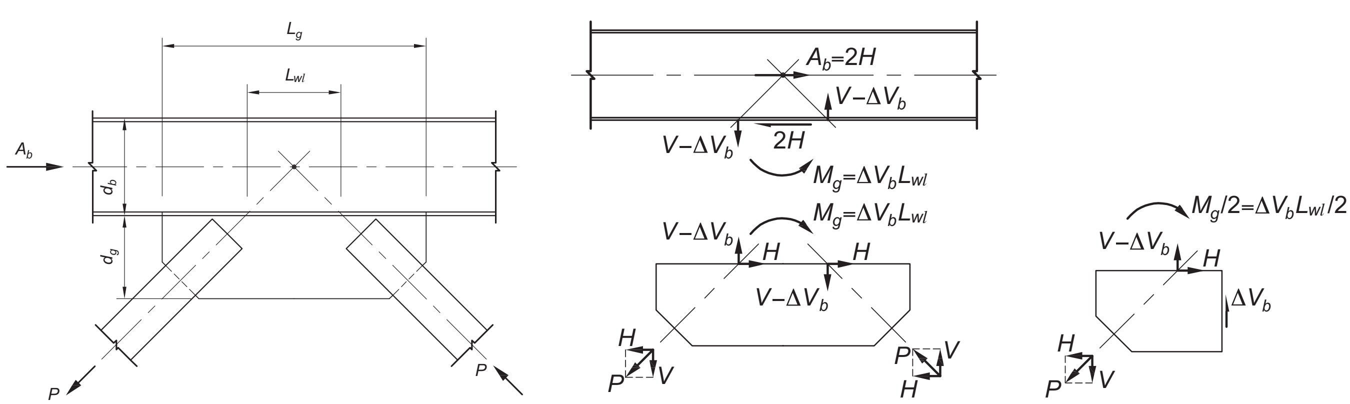


Fig. 3. Inverted-V connection with discontinuous gusset.



(a) Connection assembly *(b) FBD illustrating load transfer* *(c) Left half of the gusset plate*

Fig. 4. Inverted-V brace configuration force transfer.

Therefore, for the limit state of shear yielding, the beam alone is insufficient to resist the vertical shear, V, of 200 kips. This can be illustrated clearly through finite element analysis (FEA) of the example configuration, but with the gusset made discontinuous similar to the configuration shown in Figure 3, which mandates the distribution of all vertical shear to the beam. Force-controlled FEA was performed using Comsol Multiphysics® software (2023). Partial models (joint only), such as those used in an attempt to reflect a system's overall behavior, can be sensitive to boundary conditions. To best address this issue, the models herein incorporate a 15-ft floor-to-floor height and 30-ft column-to-column spacing. The full length of beams and braces were included in the models, each with pinned ends, to adequately capture the influence of member axial, flexural, and shear stiffness. Nonlinear material characteristics were implemented to allow for inelastic behavior. An example of a model is shown in Figure 5.

Figure 6 shows substantial yielding in the shear region between gusset plates with large shear deformation due to inelastic strains. By calculation, an average shear stress of 59 ksi is required across the full depth of the beam to resist the applied shear. The nonlinear FEA model allowed for strain hardening and was able to achieve static equilibrium at stresses of approximately 60 ksi uniformly spread across the region between gusset plates consistent with the calculated requirement, the difference likely due to the minimal contribution of brace flexure. The large inelastic strains resulting from failure of the yield criterion are unacceptable for a connection intended to behave substantially elastic under the applied loading.

By making the gusset plate continuous consistent with the intended configuration shown in Figure 4, the required shear can be redistributed to the gusset plate. Assuming the entire required vertical shear is distributed to the gusset plate as discussed previously, the required vertical shear in the beam becomes zero. The LRFD forces associated with this assumption are as follows:

$$\begin{aligned}\Delta V_b &= V = 200 \text{ kips}\\ 2H &= A_b = 400 \text{ kips}\\ M_g &= 2H\frac{d_b}{2}\\ &= (400 \text{ kips})\frac{(17.9 \text{ in.})}{2}\\ &= 3{,}580 \text{ kip-in.}\end{aligned} \tag{2}$$

Per AISC *Specification* Equations J4-3 and J4-4 for shear yielding and shear rupture, respectively, the available LRFD shear strengths of the gusset plate along a vertical plane are determined as:

$$\begin{aligned}\phi R_{n-yield} &= \phi 0.6 F_y A_{gv}\\ &= 1.0(0.6)(50 \text{ ksi})(0.5 \text{ in.})(14 \text{ in.})\\ &= 210 \text{ kips} > 200 \text{ kips} \quad \textbf{o.k.}\end{aligned} \tag{3}$$

$$\begin{aligned}\phi R_{n-rupture} &= \phi 0.6 F_u A_{nv}\\ &= 0.75(0.6)(65 \text{ ksi})(0.5 \text{ in.})(14 \text{ in.})\\ &= 204 \text{ kips} > 200 \text{ kips} \quad \textbf{o.k.}\end{aligned} \tag{4}$$

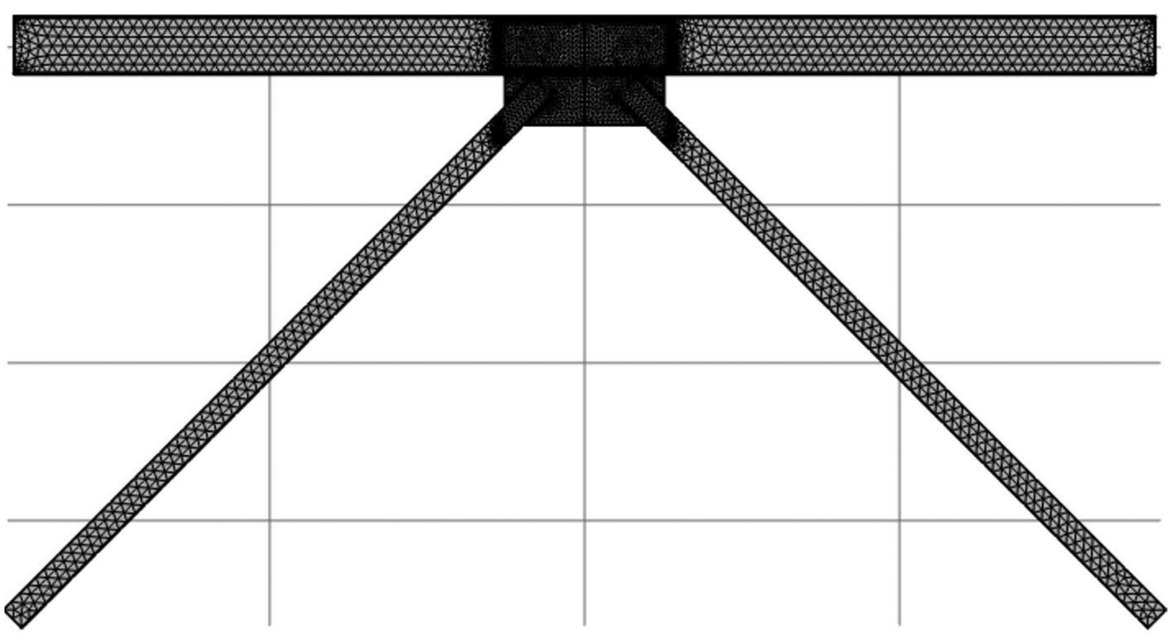


Fig. 5. Example inverted-V brace configuration FEA model.

Per AISC *Specification* Equations J4-3 and J4-4 for shear yielding and shear rupture, respectively, the available LRFD shear strengths of the gusset plate along a horizontal plane at the interface with the beam is determined as:

$$\begin{aligned}\phi R_{n-yield} &= \phi 0.6 F_y A_{gv} \\ &= 1.0(0.6)(50\text{ ksi})(0.5\text{ in.})(50\text{ in.}) \\ &= 750\text{ kips}\end{aligned} \tag{5}$$

$$\begin{aligned}\phi R_{n-rupture} &= \phi 0.6 F_u A_{nv} \\ &= 0.75(0.6)(65\text{ ksi})(0.5\text{ in.})(50\text{ in.}) \\ &= 729\text{ kips (governs)}\end{aligned} \tag{6}$$

The plastic LRFD flexural strength of the gusset plate along the interface with the beam is determined as:

$$\begin{aligned}\phi M_{n-yield} &= \phi F_y Z_g \\ &= 0.9(50\text{ ksi})\frac{(0.5\text{ in.})(50\text{ in.})^2}{4} \\ &= 14{,}060\text{ kip-in. (governs)}\end{aligned} \tag{7}$$

$$\begin{aligned}\phi M_{n-rupture} &= \phi F_u Z_n \\ &= 0.75(65\text{ ksi})\frac{(0.5\text{ in.})(50\text{ in.})^2}{4} \\ &= 15{,}230\text{ kip-in.}\end{aligned} \tag{8}$$

The interaction for planar stresses on the gusset plate is determined using AISC *Manual* Equation 9-1:

$$\begin{aligned}\frac{M_r}{M_c} + \left(\frac{P_r}{P_c}\right)^2 + \left(\frac{V_r}{V_c}\right)^4 &= \frac{3{,}580\text{ kip-in.}}{14{,}060\text{ kip-in.}} + \left(\frac{400\text{ kips}}{729\text{ kips}}\right)^4 \\ &= 0.346 < 1.0 \quad \textbf{o.k.}\end{aligned} \tag{9}$$

The remaining checks are for the beam web and are discussed in detail later.

Though the vertical component of the brace force has been resolved into the gusset, the presence of a moment on the gusset-to-beam interface signifies there is still an unavoidable state of shear stress in the beam. The same is true when using AISC *Manual* Special Case 2. The redistribution of the shear from the shear connection to the gusset-to-column connection eliminates shear resulting from the force in the brace on the beam connection. However, the moment imposed on the gusset-to-beam interface resulting from the redistribution of shear must be resisted by a portion of the beam web, but at a location slightly away from the beam connection. The location and magnitude of the shear in the beam web can be manipulated by changing the length of the gusset-to-beam interface. In the case of the V or inverted-V condition, the vertical shear resulting from the moment is related to the horizontal shear at the

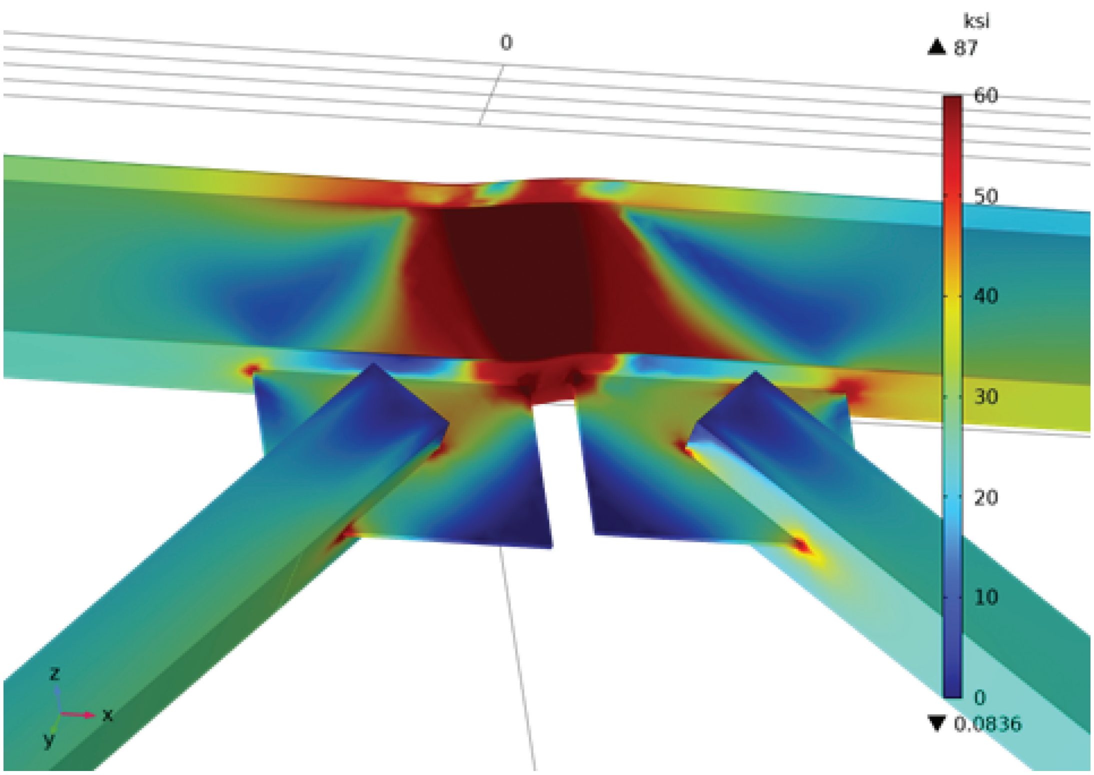


Fig. 6. Von Mises stress profile from FEA model of inverted-V connection with discontinuous gusset.

bottom flange since the only moment present results from the shear itself.

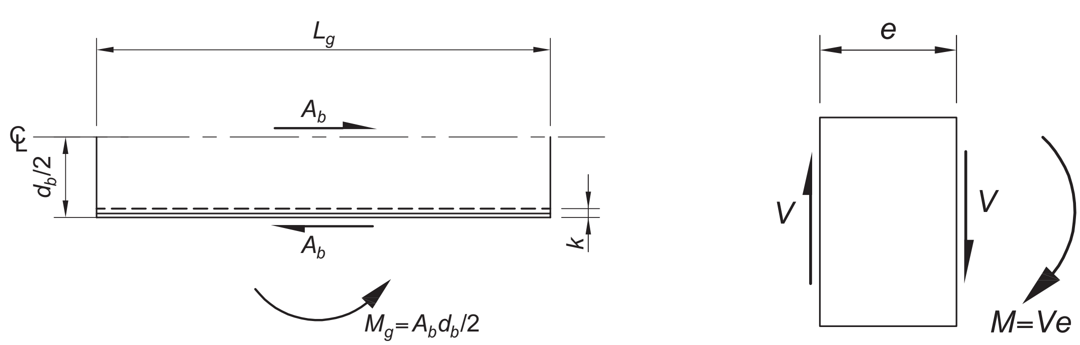
Both the uniform stress method (USM) presented by Fortney and Thornton (2015, 2017) and the concentrated stress method (CSM) presented by Sabelli et al. (2021) convert the applied moment into a force couple, where the magnitude of the forces in the couple is used to determine the vertical shear demand on the beam. In this fashion, the vertical shear is dependent on the assumptions made to convert the moment to a force couple. The conversion of a moment to a force couple is particularly convenient where dissimilar materials are used, where cross sections are not uniform, or to approximate inelastic behavior. This conversion is not necessary for the web of a wide-flange (WF) beam, recognizing through mechanics of materials that when the only moment applied to a rectangular element is due to the offset (eccentricity) of a shear applied in a given direction, the shear stress in that direction is equal to the shear stress in the orthogonal direction. Therefore, in an isotropic material such as steel, resolving the state of stress at the most critical section inherently resolves the state of shear stress in the orthogonal direction.

To illustrate, Figure 7(a) shows a profile of the lower half of the beam over the length of the gusset plate with the required forces assuming all vertical shear from the braces is distributed to the gusset plate. The forces on this section of the beam are identical to those assumed to act on a typical extended single plate as shown in Figure 7(b), except that the conditions are rotated 90° from one another. A similar diagram results when looking at the left half of a simply supported beam with a concentrated load at the center, which can also be a good illustration of how average horizontal and vertical shear stress are equal in an isotropic beam with uniform cross section. In the design of a bare steel beam, only vertical shear is evaluated, recognizing that horizontal shear is inherently addressed in doing so. Under the extended single-plate methodology in AISC *Manual* Part 10, only the critical stress state along a vertical plane is evaluated under the interaction of shear and moment, recognizing that the horizontal shear state is inherently addressed. The same can be done for the beam web in the V and inverted-V condition except that the critical stress state is along the horizontal plane at a distance of k from the bottom of the beam, where k is the design distance from the face of the beam flange to the intersection of the fillet with the web.

The interaction equation for extended single-plate connections in AISC *Manual* Part 10 (Equation 10-8) differs slightly from AISC *Manual* Equation 9-1 in that both the shear and moment ratios are squared and no axial ratio is included. Because a vertical normal force, such as from gravity loads, could be present in many instances, it is recommended that AISC *Manual* Equation 9-1 be used as it was for evaluation of the gusset plate. AISC *Specification* Section J10.2 allows for a distance of 2.5k to be added to each available side of the bearing length when evaluating the limit state of web local yielding. It is obvious this increase in length is available to resist horizontal shear as well. Therefore, it is reasonable to take advantage of this increased length when evaluating the forces on the beam web. Additionally, the moment at the critical section in the web (intersection with the fillet) could be reduced, recognizing that the eccentricity is decreased by the k dimension at the critical location. The authors generally recommend reserving use of the additional length of section and reduced moment for the evaluation of existing structures.

In continuation of the example, the available LRFD capacities of the beam web for the limit states of shear yielding and flexure, respectively, are determined as:

$$\begin{aligned}\phi R_{n-yield} &= \phi 0.6 F_y A_{gv} \\ &= 1.0(0.6)(50\text{ ksi})(0.315\text{ in.})(50\text{ in.}) \\ &= 473\text{ kips}\end{aligned} \quad (10)$$



(a) Portion of beam between gusset and beam centerline *(b) Extended single plate*

Fig. 7. Free-body diagram.

$$\phi M_{n-yield} = \phi F_y Z_g \tag{11}$$
$$= 0.9(50 \text{ ksi})\frac{(0.315 \text{ in.})(50 \text{ in.})^2}{4}$$
$$= 8{,}859 \text{ kip-in.}$$

Per AISC *Manual* Equation 9-1 the interaction for planar stresses on the beam web is determined as:

$$\frac{M_r}{M_c} + \left(\frac{P_r}{P_c}\right)^2 + \left(\frac{V_r}{V_c}\right)^4 = \frac{3{,}580 \text{ kip-in.}}{8{,}859 \text{ kip-in.}} + \left(\frac{400 \text{ kips}}{473 \text{ kips}}\right)^4 \tag{12}$$
$$= 0.915 < 1.0 \quad \textbf{o.k.}$$

Web local crippling is the final check for local effects on the beam, for which the traditional approach of converting the required moment into a force couple is taken assuming a plastic stress distribution. The required demand is determined as:

$$R_u = \frac{M_u}{\frac{L_g}{2}} \tag{13}$$
$$= \frac{3{,}580 \text{ kip-in.}}{\frac{50 \text{ in.}}{2}}$$
$$= 143 \text{ kips}$$

Using AISC *Manual* Equation 9-64a, the available strength is determined as:

$$\phi R_n = 2\left[\phi R_5 + l_b(\phi R_4)\right] \tag{14}$$
$$= 2\left[42 \text{ kips} + (25 \text{ in.})(3.60 \text{ kips/in.})\right]$$
$$= 264 \text{ kips} > 143 \text{ kips} \quad \textbf{o.k.}$$

Lastly, the weld may be evaluated using AISC *Manual* Table 8-4 for the condition where k, as defined in the weld diagram, is taken equal to zero. Setting the eccentricity, $al = d_b/2$ results in $a = (17.9 \text{ in.})/[2(50 \text{ in.})] = 0.179$. Interpolation within the table results in the coefficient, $C = 3.58$. The required weld size in 16ths of an in., assuming $C_1 = 1.0$ for E70XX electrodes, is then determined as:

$$D \geq \frac{P_u}{\phi C C_1 L_g} \tag{15}$$
$$= \frac{400 \text{ kips}}{0.75(3.58)(1.0)(50 \text{ in.})}$$
$$= 2.98$$

Therefore, use a 3⁄16 in. fillet weld.

To ensure ductility and recognizing it as one of the fundamental requirements of the LBT, it is prudent to verify that the weld can develop the lesser strength of the gusset plate or the beam web. This can be done by recognizing that the condition of loading and detailing is generally consistent with that of extending single-plate connections such that the weld size need only meet or exceed $\tfrac{5}{8}t_p$, where t_p is taken as the lesser of the gusset plate and beam web thicknesses. Under this approach, the required weld size to ensure ductility is determined as:

$$t_{weld} \geq 0.625 t_w \tag{16}$$
$$= 0.625(0.315 \text{ in.})$$
$$= 0.197$$

Therefore, it is recommended to increase the fillet weld size by ¼ in.

The interaction on the beam calculated in the example was 0.915, suggesting the gusset plate was slightly longer than the minimum required. By iteration, the gusset length optimized to result in an interaction of 1.0 is found to be 48.5 in. It is often desirable to determine a required gusset length from the outset of design prior to checking other limit states. The shear term in AISC *Manual* Equation 9-1 can be conservatively squared rather than taken to the fourth power to conveniently allow for the gusset length to be isolated in the equation. Because this is a more conservative formulation, it is reasonable to take advantage of the $5k$ increase in effective web length mentioned previously. The result is a required gusset length determined as:

$$L_{g,\min} = \sqrt{\frac{4M_r}{\phi F_y t_w} + \left(\frac{P_r}{\phi F_y t_w}\right)^2 + \left(\frac{V_r}{\phi 0.6 F_y t_w}\right)^2} - 5k \tag{17}$$

Using this equation for the previous example results in a required length of 48.3 in., nearly matching the length required by iteration using AISC *Manual* Equation 9-1.

The design is adequate by calculation under the applicable limit states. Figure 8 shows the stress profile of the example connection with the continuous gusset. Contrary to the discontinuous model shown in Figure 6, the continuous gusset model exhibits minimal local deformation and stresses well below yield in the upper portion of the beam web between braces. Comparison of the continuous gusset model to the discontinuous gusset model affirms a distribution of shear to both the gusset and beam web. The continuous model also illustrates aspects of the LBT—namely, that minimal inelastic strain (evident in the lower portion of the beam web) is needed to redistribute stress. The field of high stress is most prominent at the bottom of the beam web where moment is highest, then dissipates to lower levels near the center of the beam where shear is assumed to have been transferred to the workline and moment is assumed to be zero. This behavior is consistent with the idealized shear and moment model of Figure 7(a). For this reason, the connection can be assumed to behave essentially elastic from the perspective of global analysis of the frame, though some local yielding is likely to exist. The example using the continuous gusset supports the use of properly proportioned gusset plates to address the transfer of vertical shear using the LBT.

While inelastic deformations can be accommodated in many ways, it is generally desirable to provide ductility through global material yielding rather than highly localized yielding (stress concentrations) or buckling. As such, where WF beams are used in a web-vertical orientation, it is prudent to utilize beam sections that are not susceptible to shear buckling of the web—that is, beam sections should satisfy the h/t_w limit in AISC *Specification* Section G2.1(a) (AISC, 2022a). As noted in the example, proportioning welds to develop the lesser strength of the gusset plate or beam also ensures ductility.

TRUSS CONNECTIONS

Another application of this method is for truss web-to-chord connections. Trusses often utilize WF chord members with webs oriented vertically to improve flexural properties where loading occurs between truss panel points. Simple-span trusses can be highly optimized to save material such that relatively light chord members are used near the truss supports where chord axial forces are low. These same locations have the largest axial forces in the truss web members as these are the locations of the greatest global shear in the truss. By designing the gusset to transfer all vertical shear, such as along section A in Figure 9, the lighter chord members can be used without a need for doubler plates to address a potential shear overstress. The interface forces shown in Figure 9(b) reflect this redistribution of all shear to the gusset plate, which is the equivalent at setting $\Delta V_b = V$ in Figure 2. Conversely, toward the center of a simple-span truss where chord members tend to be heavy and axial forces in truss web members are low, shear can be distributed primarily to the chord member to potentially achieve more efficient gusset plates.

BRACE-TO-COLUMN CONNECTIONS

As illustrated by Sabelli et al. (2021), the force distribution at braced frame beam-to-column connections with a common gusset plate for both the brace and beam connections has similar force actions to V and inverted-V connections at brace intersections. Where the horizontal components of the brace forces, shown as H_1 and H_2 in Figure 10, are

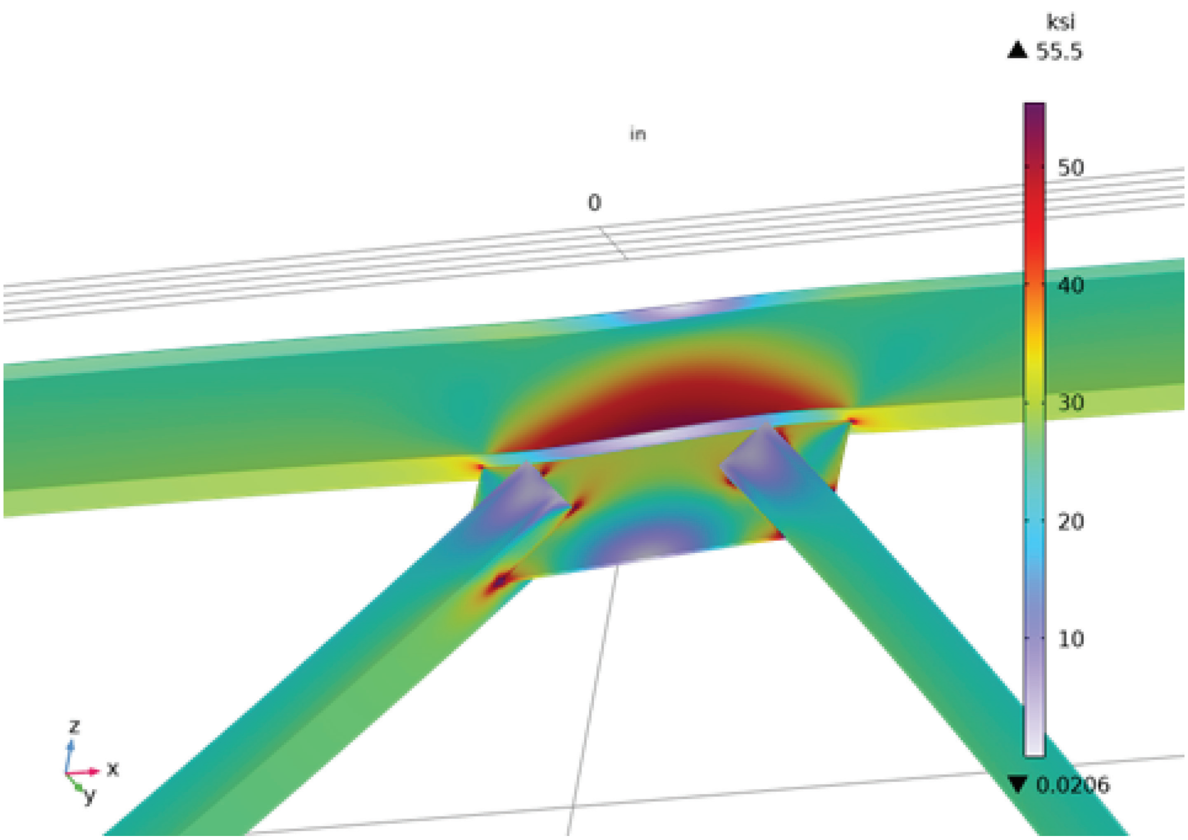


Fig. 8. Von Mises stress profile from FEA model of inverted-V connection with continuous gusset.

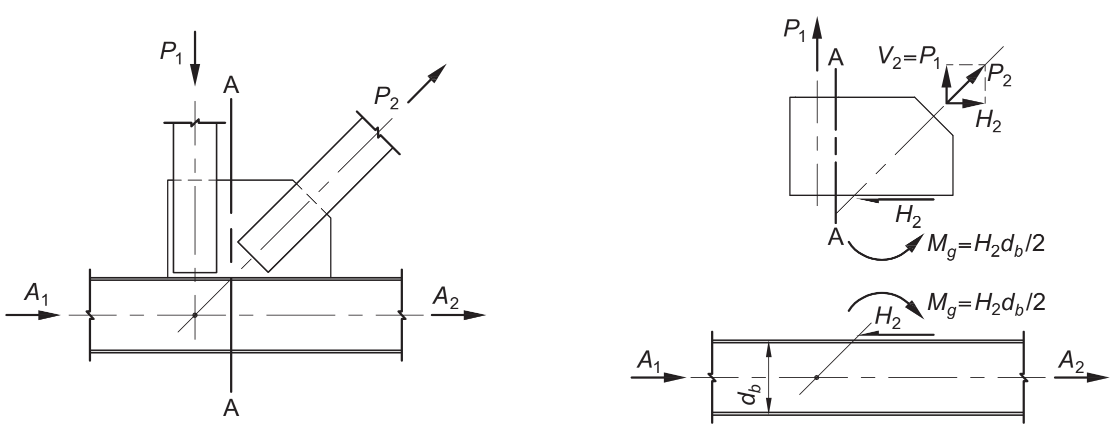
assumed to be transferred into the column, the column can often be deficient in shear capacity. The shear to the column can be reduced by redistributing some or all of the horizontal forces to the gusset plate. The gusset plate can then be proportioned to provide sufficient capacity to carry the required shear. The gusset-column interface designed for the moment must satisfy statics under the redistribution.

The moment resulting from the brace connection above the brace, the moment from the beam-to-column connection, if any, and the moment from the brace connection below the beam can be summed and addressed as a total moment, M_g, resolved over the full length of the interface, L_g, as shown in Figure 10. Note that the condition in Figure 10 assumes the bolt groups connecting the beam web to the gusset plate are designed for the eccentricity measured from the center of the respective bolt groups to the face of the column. In this manner, no moment from the beam-to-column connection is applied to the gusset-column interface. The column is assumed to be designed for the largest unbalanced moment, M_c, resulting from the beam connections on each side imposing their vertical end reactions, V_{b1} and V_{b2}, at their respective column faces. These assumptions are commonly made within commercial software for column design. Here the unbalanced moment is conservatively shown, assuming V_{b1} is at full force while V_{b2} is taken as zero.

As an alternate approach to the design of the gusset-column interface, the moments may be addressed independently over their respective compartmentalized connection lengths, L_1 and L_2, in the case of Figure 10(b). The former approach described in the previous paragraph is generally more economical as the section properties over the full length of the gusset plate are more effective as the square of the larger length rather than the sum of the squares of shorter lengths. On the other hand, the latter compartmentalized approach can be more effective in addressing many different brace configurations in design schedules, as the detailer may be able to select individual designs for each compartment from separate schedules. For example, the detailer could select parameters from design tables for the condition of a brace above a beam, parameters for a different condition of a brace below the beam, and parameters from perhaps a separate table that addresses the beam connection interface forces over the depth of the beam. All parameters from these conditions are then combined to detail the overall condition—in this case, of braces above and below a beam.

Similar to the beam condition, redistribution of shear from the column to the gusset plate may depend on inelastic behavior of the panel zone within the column. If such behavior is deemed undesirable, perhaps for reasons such as serviceability or resiliency, the panel zone could be strengthened by upsizing the column or adding doubler plates in an elastic design with relatively small building drifts. Alternatively, panel zone stresses could be reduced by increasing the length (vertical dimension) of the gusset. For extreme events such as seismic, some inelastic action in the columns is anticipated due to large story drifts. Strengthening the panel zone, or increasing the gusset length, to avoid shear yielding may simply force flexural yielding just above and/or below the connections to accommodate drifts.

Columns in seismic braced-frame systems must be moderately ductile or highly ductile in accordance with the AISC *Seismic Provisions* (AISC, 2022b) to accommodate such inelasticity. One approach to minimize column deformations due to building drifts is to rotate columns such that gusset plates are connected to the web of WF columns. The out-of-plane flexibility and relatively small out-of-plane flexural strength of the WF web can accommodate gusset rotation with minimal localized inelastic behavior as discussed in the AISC *Seismic Provisions* Commentary. Using the UFM, connections to the column web mathematically



(a) Connection assembly *(b) FBD illustrating load transfer*

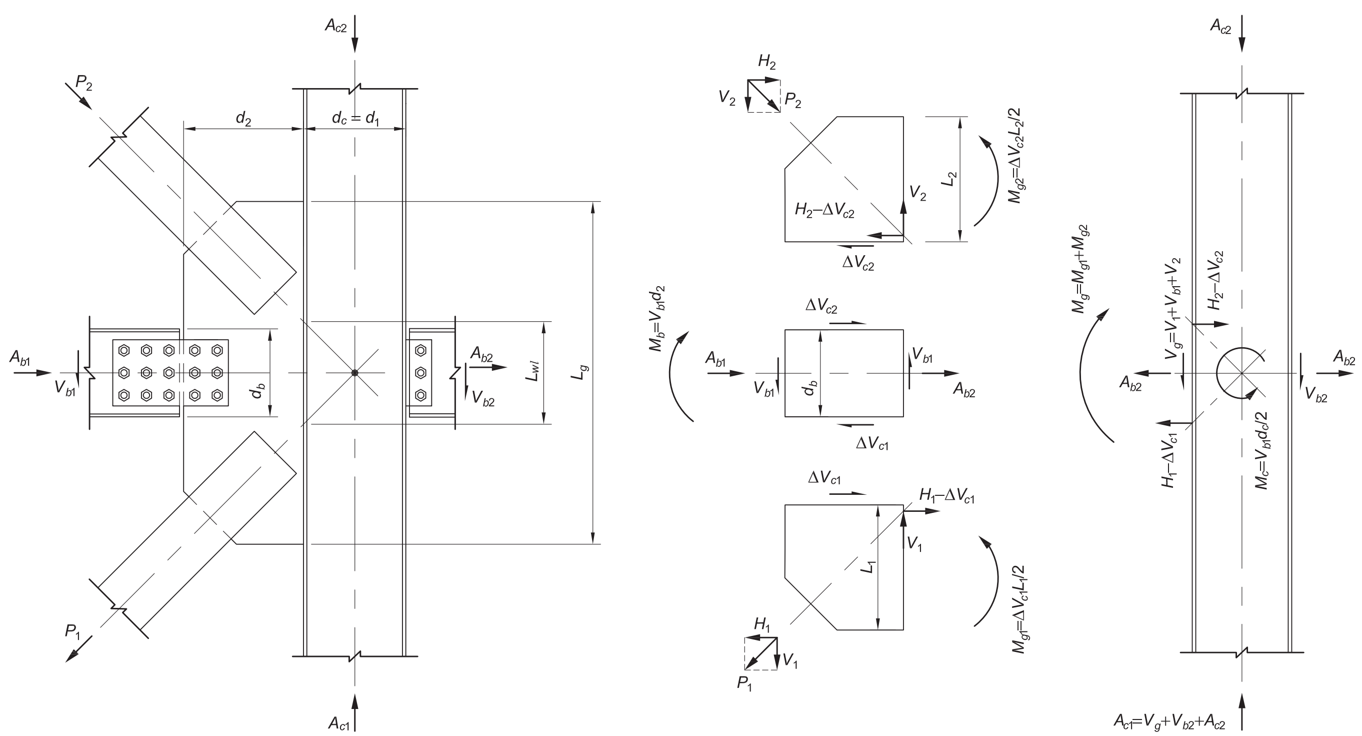
Fig. 9. Common truss connection.

eliminate shear in the column, thereby obviating the chevron effect. This is because the location of load application is effectively at the workline of the column such that any offset (eccentricity) over which vertical shear would need to be transferred is eliminated.

SUPER-X BRACE CONNECTIONS

Super-X brace configurations are often treated as a combination of a V configuration above a beam and an inverted-V configuration below the same beam. While free-body diagrams can be developed that illustrate similar gusset-to-beam connection force transfers in each of these configurations, it should be recognized that the global load paths differ. Where braced frames consist entirely of either V or inverted-V configurations at all levels, forces are transferred from the braces above a beam into the beam as an axial force, then transferred again to the braces below the beam at another location. In super-X configurations, forces from the braces above the beam are transferred through the beam to the braces below the beam at a singular location in a localized manner. Any axial forces in the beam are primarily collector forces only. Thus, the brace force transfer through the beam is dominated by axial behavior. Because the axial stresses from braces in one direction are compressive and tensile in the opposite direction, the combined stress state within the beam web can be transposed to a state of shear stress. This state is illustrated in Figure 11, where the braces above and below the beam are oriented at a 45° angle of inclination and assumed to be of equal magnitude, such as in a multi-tier braced frame. From this condition, it is evident that if the beam web is inadequate to transfer the axial forces from the braces, then the web is inherently inadequate for the transposed shear state. This helps to explain why the chevron effect appears to be an issue in super-X configurations more so than repeated V or inverted-V configurations as surmised by Sabelli and Saxey (2023).

The super-X connection can be addressed as a transfer of horizontal and vertical shears, with the vertical shears redistributed to the gusset plates above and below the beam just as in the V and inverted-V illustrations. This approach again depends on the inelastic behavior of the beam web to provide sufficient ductility for the redistribution of forces. As an illustration, assume a two-story multi-tier brace configuration, where the story shears above and below the beam are equal using the brace forces and member sizes from the example presented previously in an inverted-V configuration. By redistributing the entire vertical shear associated with each story to the respective gusset plates above and below the beam, the resulting shear state in the beam is due only to the moments at the top and bottom beam faces, which are additive. As discussed previously, methods that resolve these moments into force couples to



(a) Connection assembly (b) FBD illustrating load transfer

Fig. 10. Brace connection to column.

obtain a required vertical shear are approximate by nature and can result in a state of stress that deviates somewhat from basic mechanics of materials to account for inelastic behavior.

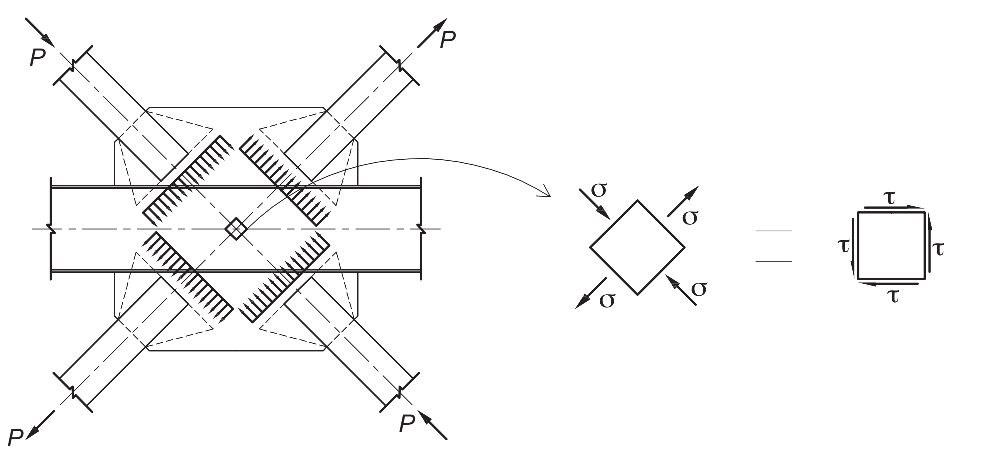
AISC *Manual* Equation 13-33 is derived using the USM, assuming the full beam depth is used to resist shear imposed by a gusset at either face. As such, the moments above and below must be conservatively added to arrive at a total vertical shear to be resisted by the full beam depth. For the example shown previously, the gusset length, L_g, required by AISC *Manual* Equation 13-33a is 42.3 in. for a single-sided condition. For a super-X configuration where the loading above and below the beam are the same, such as in a multi-tier braced frame, the total additive moment is double that of the single-sided condition, resulting in a required gusset length of 84.6 in., which is greater than the 50 in. detailed in the example.

Recall that a single-sided gusset length of 48.5 in. was required using the interaction method from the example problem. This required length is greater than that determined using the USM because the USM considers normal forces and shear forces independently (no interaction). The diagrams shown in Figure 7 illustrate that addressing the moment and horizontal shear applied by the gusset on one face of the beam inherently resolves the shear over the half-depth of the beam adjacent to the gusset. Applying a gusset to both sides of the beam simply creates an additional state of stress in the web on the opposing side of the beam centerline. Figure 12 illustrates this through modeling of the super-X condition wherein the forces in the braces above the beam are of the same magnitude as the those below (the gusset conditions are effectively mirrored about the beam centerline). Examination of the stress contours in comparison to the one-sided condition of Figure 6 show that the state of stress above the beam centerline is essentially a mirror image of the condition below as expected. This illustrates that the super-X configuration is no more severe than the single-sided condition with regard to the magnitude of maximum shear stress. The state of stress is simply present on both sides of the beam centerline (through the full depth) in the super-X configuration. Thus, gusset and beam design can be compartmentalized such that addressing each gusset condition independently, using the interaction method shown in the example, ensures an acceptable maximum state of stress within the beam web. Further, the required gusset length for each gusset remains 48.5 in. as opposed to the significantly more conservative 84.6 in. using the USM.

The CSM developed by Sabelli et al. (2021) derives a vertical shear resulting from an approximate force couple model that is more accurate than that of the USM. For example, a required length approximated using Equation 5 from Sabelli and Bolin (2022) using the nomenclature herein is determined as:

$$\begin{aligned} L_g &= \frac{1.25M_{tot}}{\phi V_n} \qquad (18) \\ &= \frac{1.25(400 \text{ kips})(17.9 \text{ in.})}{169 \text{ kips}} \\ &= 53.0 \text{ in.} > 50 \text{ in.} \quad \textbf{n.g.} \end{aligned}$$

This value is substantially less than that calculated using the USM but still exceeds the 50 in. detailed. Sabelli et al. (2021) provide for progressively more rigorous calculations to refine the required length if needed. Considering Equation 39 from Sabelli and Saxey (2021) recognizing the moments above and below the beam are each half of the total moment, the required length is determined as:



(a) Distribution from gusset plate　*(b) Approximate normal stress state in the web*　*(c) Rotation creates pure shear stress*

Fig. 11. Axial stress distribution in super-X connection.

$$L_g = \frac{M_g}{0.5\phi V_n} + \frac{0.5\phi V_n}{\phi F_y t_g} \tag{19}$$

$$= \frac{3{,}580 \text{ kip-in.}}{84.5 \text{ kips}} + \frac{84.5 \text{ kips}}{0.9(50 \text{ ksi})(0.5 \text{ in.})}$$

$$= 46.1 \text{ in.} < 50 \text{ in.} \quad \textbf{o.k.}$$

This value is slightly less than the 48.5 in. determined using the interaction approach. Sabelli et al. (2021) provide for further refinements that could reduce the required length slightly below 46.1 in. These methods also consider the limit states under normal forces independently of those under shear forces rather than addressing in a combined interaction, which explains the modestly lower length as compared to the interaction method.

As an alternative to the UFM using the USM, CSM, or interaction approach, the axial force transfer can be addressed directly by simply carrying the forces on the Whitmore sections (AISC *Manual* Part 9) for each brace into the web of the beam and designing for those forces directly. In this manner, several approaches can be taken to address a deficiency in beam-web capacity. For example, the length of the brace-to-gusset connection could be increased, thereby increasing the width of the Whitmore section and reducing the stress on the beam web. This approach increases gusset size, which is an analogous solution to determining an adequate gusset length using the USM or CSM. Note that overlap of the Whitmore sections for opposing braces is acceptable, recognizing that the opposing stresses do not occur in the same direction and are, thus, not additive. The effect of these increases in connection length, gusset size, and Whitmore section width to reduce stress is illustrated in Figure 13. Regarding local web stability using this method, the vertical force component on the projected Whitmore section should be checked in accordance with AISC *Specification* Section J10. Another method of addressing local web stability and a potentially deficient web is to provide diagonal stiffeners such as those shown in Figure 13. Of little surprise, these stiffeners effectively reinforce the panel zone for increased shear resistance. Downsides to this approach include more complicated welding at the interface of the skewed stiffener with the inside of the beam flange and potential complications with stiffener geometry, where braces above and below the beam occur at substantially different inclination angles. FEA has shown that adding vertical and/or diagonal stiffeners does change the locations of the highest stress within the beam web somewhat, but the overall behavior is unchanged.

This approach of axial force transfer using the Whitmore section can be applied to the super-X connection example considered previously. The connection parameters are shown in Figure 14, where the length of the brace-to-gusset connection is 12 in.

The length of the Whitmore section, l_{wh}, is determined by geometry as:

$$l_{wh} = (8 \text{ in.}) + 2(12 \text{ in.})\tan(30°) \tag{20}$$

$$= 21.9 \text{ in.}$$

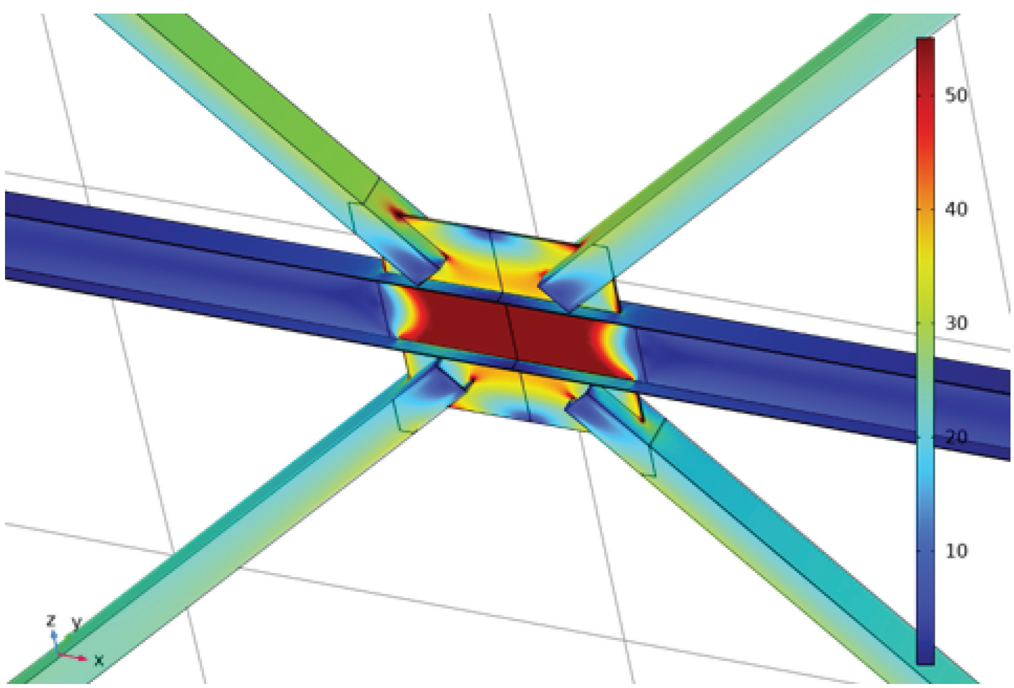


Fig. 12. Von Mises stress profile from FEA model of X connection.

The strength of the web for the limit state of axial yielding is determined as:

$$\phi P_n = \phi F_y t_w l_{wh} \tag{21}$$
$$= 0.9(50 \text{ ksi})(0.315 \text{ in.})(21.9 \text{ in.})$$
$$= 310 \text{ kips} > 283 \text{ kips} \quad \textbf{o.k.}$$

This calculation assumes web buckling and web crippling limit states are checked separately or mitigated with stiffeners. At a demand/capacity ratio of 0.91 for the limit state of axial yielding, the web geometry is relatively well optimized, though there is room for further refined. Figure 14 shows that the required gusset length to develop the full Whitmore section into the beam is 43 in. This is close to the 46 in. determined using the CSM, particularly recognizing even more precise approaches can be taken to the CSM than illustrated herein. The 43 in. length is very close to 42.3 in. for a one-sided connection using the USM. While there is conjecture in the absence of a refined FEA model using a 42.3 in. gusset length, this suggests the USM as intended for one-sided connections is also appropriate for each side of super-X conditions. This does make sense as the final form of the equation relates to horizontal shear only. Accepting this equation as accurate but recognizing the top and bottom conditions are resolved on their respective halves of the beam, as discussed using the interaction approach, produces the most optimized length of the methods considered.

One advantage of the axial force approach using the Whitmore section is that the Whitmore length must be determined in evaluating the brace-to-gusset connection such that the check of this projected stress field on the member web is an intuitive additional step requiring little design effort. Another advantage is the correlation of the stress field on the beam web to the geometry of the brace-to-gusset connection. This correlation can serve to mitigate ill-proportioned gusset plates, where the length of the gusset may be increased without considering the brace-to-gusset connection. In extreme cases, such increases in gusset length could induce bending in the gusset plate that may not be accounted for by the designer. The implications of the axial force approach using the Whitmore section are somewhat profound in that gusset plates need only be proportioned to develop sufficient Whitmore length for each brace into the member web to inherently satisfy the shear state within the member. Note that where additional

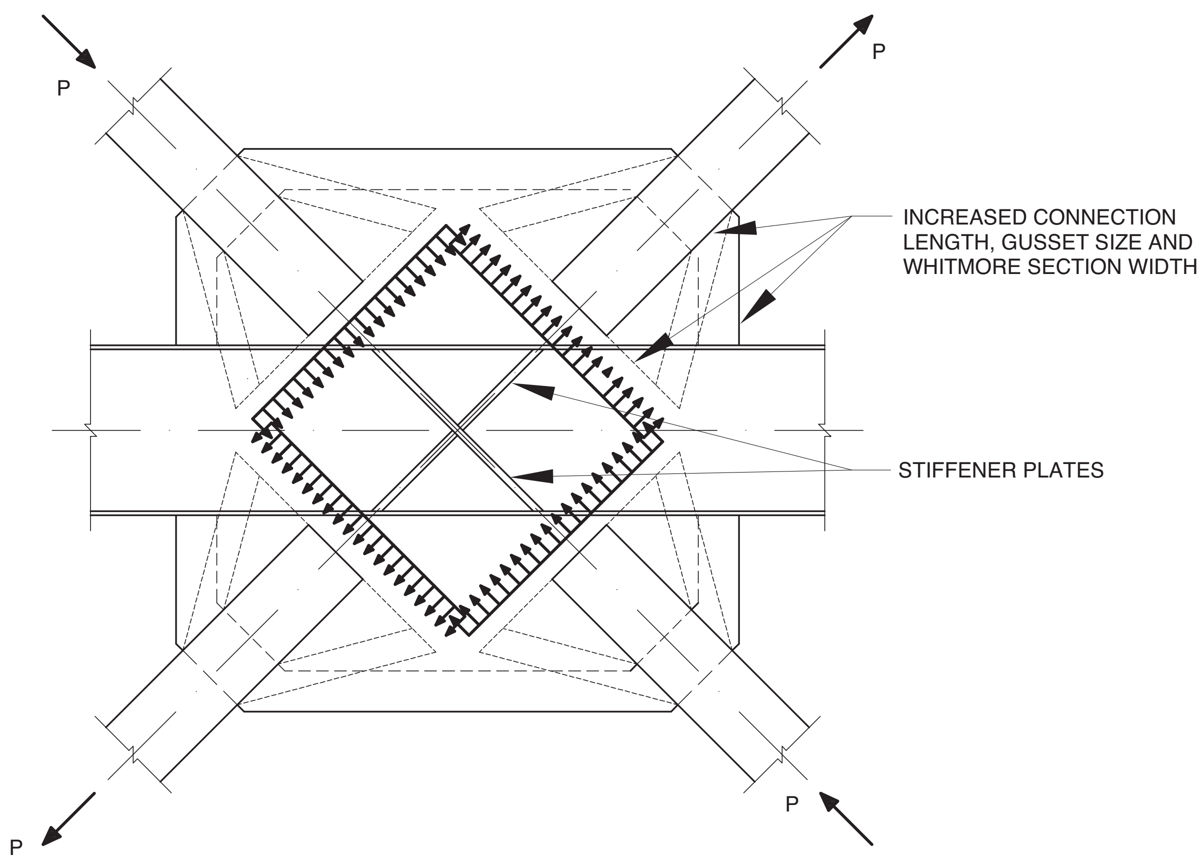


Fig. 13. Illustration of methods to address axial forces through beams in super-X connections.

axial forces are present, such as those from the beam at a beam-to-column condition, components of stress from overlapping stress fields contributing in the same direction would need to be combined to evaluate the total stress state at a given location.

SPECIAL CONSIDERATIONS FOR HSS MEMBERS

It is common in many structures to use a rectangular HSS strut as the horizontal member in a braced frame rather than a WF beam. Carrying forces through rectangular HSS members requires additional consideration. With the gusset plate centered on the horizontal face of the HSS member, the vertical walls of the HSS member (web members) are offset from the gusset plate rather than aligned. The horizontal face connected to the gusset plate typically lacks sufficient flexural strength to transfer appreciable vertical forces from the gusset plate to the HSS webs (side walls). Therefore, the gusset plate must be relied upon to transfer the entirety of the vertical components of force between braces through shear.

As discussed previously and shown in Figure 4, redistribution of vertical shear to the gusset plate results in a moment at the gusset-to-strut interface to resolve statics. Often the HSS is inadequate to transfer the vertical force couple associated with this moment due to insufficient wall plastification capacity. Figure 15 shows an FEA model of the same configuration and forces previously illustrated in Figure 12, except that the W18×40 has been replaced by a rectangular HSS18×6×⅜. The horizontal walls at the top and bottom of the HSS are inadequate to handle the moment imposed at the gusset-to-HSS interface, thereby violating a requirement of the LBT. Significant yielding with high strains and large associated gusset rotations resulted in the walls of the HSS flexing uncontrollably.

Several approaches can be taken to provide an adequate connection. One option is to increase the thickness of the HSS walls and/or the length of the gusset plate until the required forces can be resisted (increase the wall plastification capacity). Under large forces, upsizing these elements sufficiently may prove impractical. A second option is to relocate the workpoints of the braces to the faces of the HSS

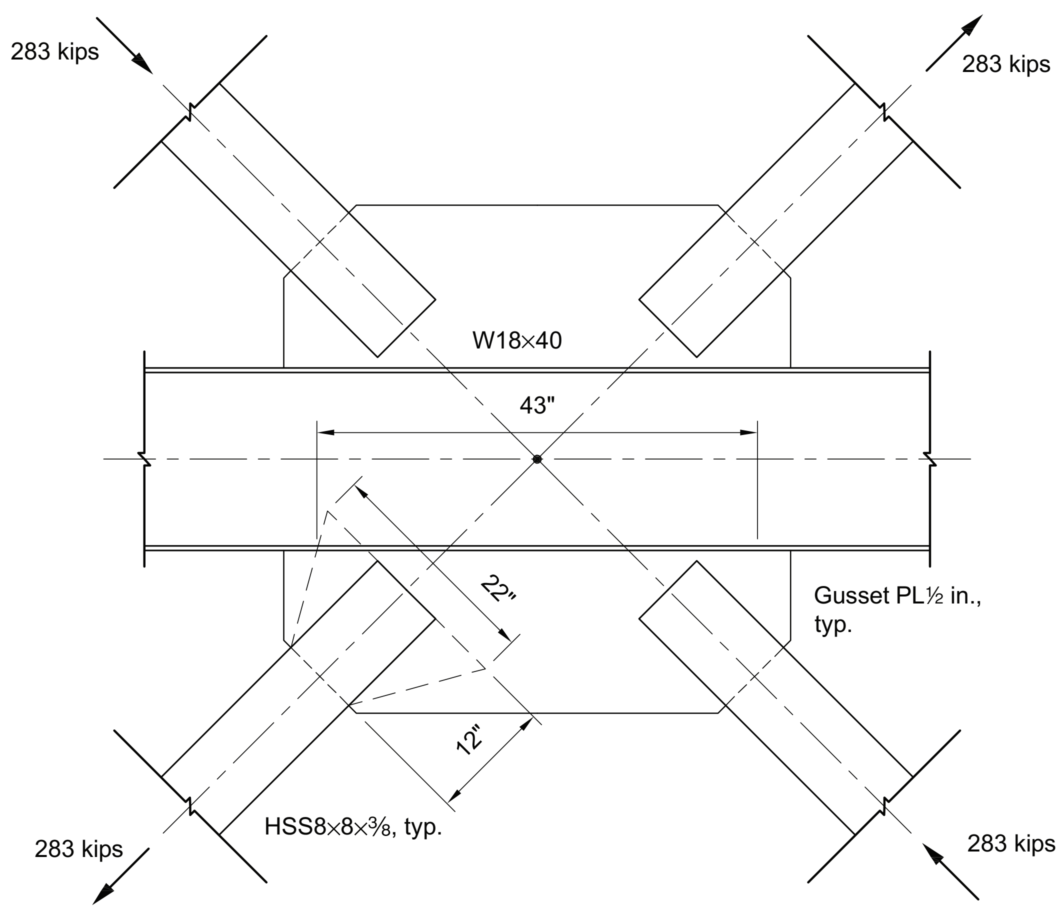


Fig. 14. Example super-X connection using Whitmore section axial force projection on beam web.

as shown in Figure 16(a). This approach requires only horizontal shear to be transferred at the gusset-to-HSS interface thereby eliminating the out-of-plane demands on the HSS wall. However, relocation of the workpoint imposes a flexure on the HSS member, which is resolved by vertical reactions at the member ends. The member must be designed for these actions, and any analysis should reflect the relocation of the workpoint by using rigid links or offsets. A third option is to slot the HSS and continue the gusset plate through both walls. This allows the required moment at the gusset-to-HSS interface to be addressed as a horizontal force couple through the longitudinal attachments to the HSS walls top and bottom, thereby eliminating the need for vertical force transfer transverse to the HSS walls. This approach is illustrated in Figure 16(b).

CONCLUSION

In conclusion, the chevron effect can have a significant impact on primary members depending on the approach taken to address the transfer of forces. The following conclusions are drawn from the discussions herein:

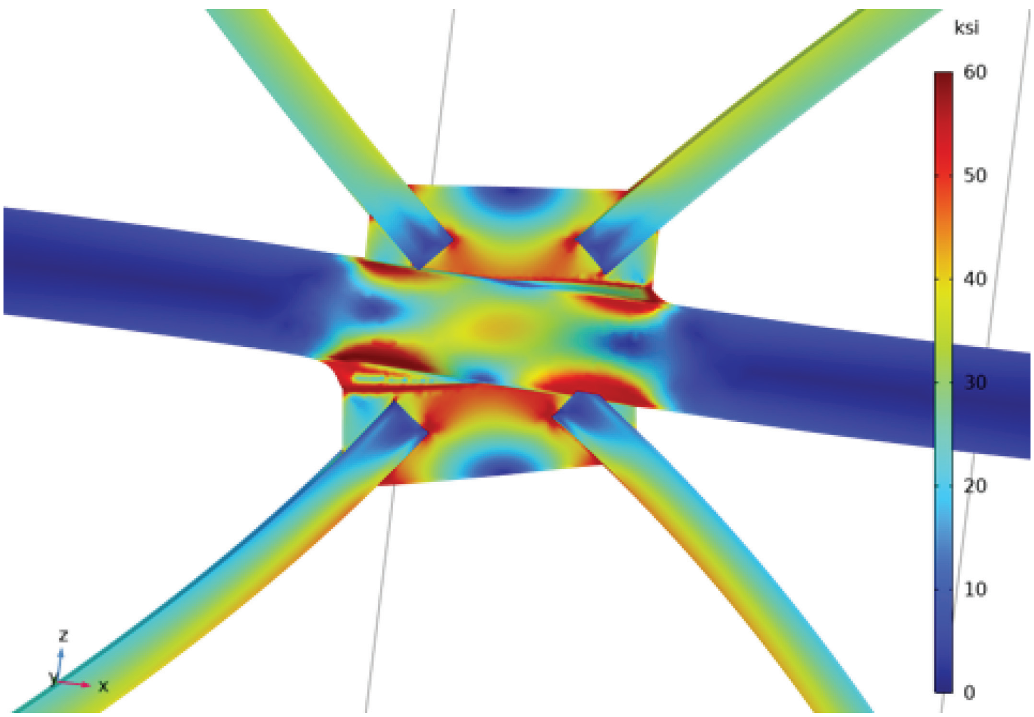
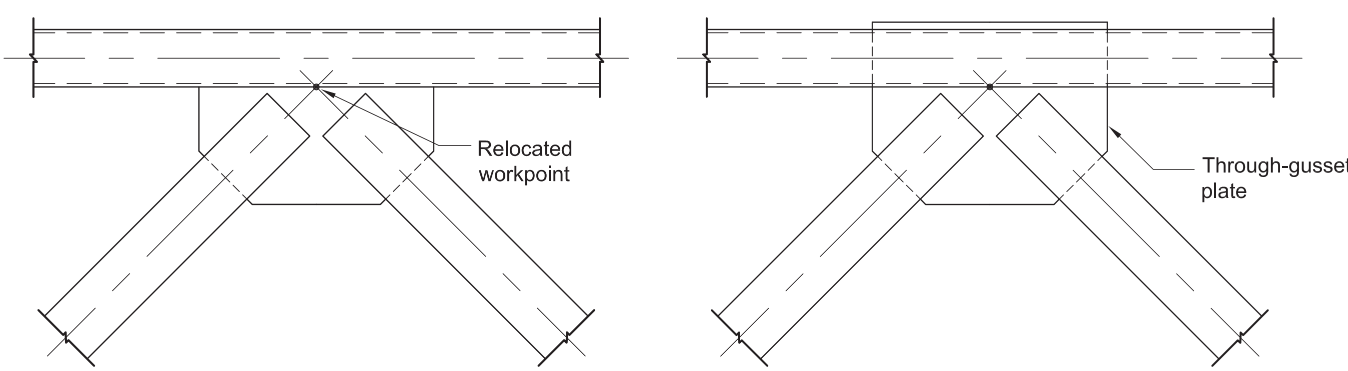


Fig. 15. Von Mises stress profile from FEA model of X connection with HSS horizontal strut.



(a) Workpoint relocated to face of HSS

(b) Through-gusset plate at slotted HSS

Fig. 16. Inverted-V brace connections to HSS horizontal strut.

1. Redistribution of forces under the LBT, including proper proportioning of gusset plates, can reduce the required shear on beam or strut members in V, inverted-V, and super-X brace configurations.
2. Similarly, force redistribution can be used to manipulate the required shear on columns in brace and beam connections to columns using a common gusset plate or in truss chords at truss diagonal-to-chord connections.
3. Because of the reliance on ductile behavior to redistribute forces to an assumed load path, WF beams or columns in braced frames and truss chords are recommended to satisfy the h/t_w limit of AISC *Specification* Section G2.1(a).
4. Connecting beam and brace connections to the web of WF column members can reduce the shear demands on columns and inelastic strains resulting from large story drifts in extreme events, such as an earthquake.
5. In super-X configurations, the beam web can be evaluated, assuming an axial force transfer on the Whitmore sections for each brace. Gusset plates can be proportioned and/or stiffener plates utilized to avoid the addition of doubler plates or upsizing of primary members, which may enhance the overall economy.
6. The transfer of forces from gusset plates to HSS beam and column members may warrant the use of through-gusset plates or relocated workpoints to address insufficient out-of-plane capacity of the HSS walls.

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