The Chevron Effect: Reserve Strength of Existing Chevron Frames

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ABSTRACT

Recently an analysis model has been developed to address the large shear forces (the so-called "chevron effect") that can develop in the connection regions of chevron-braced frames (Fortney and Thornton, 2015, 2017; Hadad and Fortney, 2020). These shear forces (and the corresponding moments) are the result of the application of the brace forces at the beam flange, eccentric to the beam centerline. Prior to the presentation of these methods, such forces were not generally considered in design, without apparent incident. Sabelli and Saxey (2021) presented an alternative model that determines substantially higher resistance in these connections. Both models resolve the shear and moment within the connection region such that forces outside that region are consistent with those determined using a centerline model. Greater resistance can be determined if the flexural strength of braces and the beam outside the connection region are used to resist a portion of the chevron moment. This paper presents a complete plastic-mechanism strength of the chevron frame with yielding of the beam web due to the local shear forces. This complete plastic mechanism strength can confirm the adequacy of existing designs that did not consider the chevron effect.

Keywords: gusset plates, braced frames, truss connections, chevron braces, analysis.

INTRODUCTION

Recent publications (Fortney and Thornton, 2015, 2017; Hadad and Fortney, 2020) have drawn attention to potentially large shear forces in the beam web in the connection region of chevron-braced frames (the "chevron effect"). Members in the braced frame are typically designed using a centerline model where forces are in equilibrium at the point where the member centerlines meet (the "work point"). The chevron effect arises when the transfer of brace forces to the beam occurs along the beam flange, which is offset from the beam centerline. This eccentricity results in a moment along the length of the gusset and a corresponding shear in the beam web. Subsequently, Sabelli and Saxey (2021) presented an alternative model of internal forces that significantly reduces the required beam shear strength. They termed this model the Concentrated Stress Model (CSM) to distinguish it from the Uniform Stress Model (USM), the term they used to describe the Fortney and Thornton method (2015, 2017). The CSM is adapted from the "optimal plastic method," while the USM employs the "conventional plastic method" as described in the AISC *Steel Construction Manual*, Part 8 (2018). Sabelli and Saxey presented design equations for both USM and CSM for new construction.

Prior to these publications, many building designs did not address such forces within the connection region, without apparent incident. Subsequently, Roeder et al. (2021) published an analysis of the seismic response of a complete frame, including flexural resistance at brace and beam connections. This analysis showed that (at least in the inelastic drift range) the local stresses in the beam web in the chevron-connection region were low. As such, there exists some reserve capacity in the chevron frame not accounted for in the CSM and USM methods.

This paper presents two methods for determining the adequacy of existing chevron beams not designed for this local effect. The first is an "internal mechanism" based on the CSM stress distribution. The second method is an "external mechanism," which includes the CSM strength and, in addition, takes advantage of the full plastic mechanism strength. This mechanism requires rotation at the beam-to-column connections and at each end of each brace. The authors consider the CSM method better suited for design of new construction due to both its relative simplicity and its independence from reliance on moments in adjoining members. The full-plastic-mechanism method is presented only as a method to check existing construction.

INTERNAL MECHANISM

Sabelli and Saxey (2021) provide guidance for beam selection and gusset sizing that ensures adequacy using CSM evaluation. The CSM methods can also be used to establish simple formulae for evaluating existing connection designs.

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This approximate method is based on determining the gusset plate length required to develop the sum of the horizontal components of the brace forces.

The chevron moment is:

$$M_{ch} = \frac{d_b}{2} \sum F_{br} \cos\theta \tag{1}$$

where

F_{br} = brace axial force, kips

d_b = beam depth, in.

θ = brace angle, with respect to the horizontal, degrees

If no other information is available, the sum of the horizontal components of the brace forces can be assumed to be the sum of the axial forces in the beam segments adjoining the connection (which may be presented in the design drawings):

$$\sum F_{br} \cos\theta = \sum P_u \tag{2}$$

The shear in the beam due to the chevron moment is:

$$V_u = \frac{M_{ch}}{e_z} \tag{3}$$

where

L_g = gusset plate length, in.

e_z = length of moment arm (see Sabelli and Saxey, 2021), in.

$$= L_g - 2z \tag{4}$$

z = length required to transfer for V_u from gusset to beam considering weld strength, gusset strength, web local yielding, and web crippling.

For the USM, the moment arm length e_z is assumed to be $\frac{1}{2}L_g$. For the CSM, it is longer; for a rough estimate it may be assumed to be $0.8L_g$. This estimate must ultimately be confirmed by evaluation of the connection for required weld size, gusset thickness, beam shear strength, and web local yielding and web crippling. For new construction, Sabelli and Saxey present minimum gusset length equations based on these limit states.

The maximum force may be limited by the shear strength of the beam:

$$V_u = \phi V_n \tag{4}$$

where

ϕV_n = available shear strength of the beam determined in accordance with AISC *Specification* (2016) Chapter G, kips

This available shear strength should be reduced in cases in which the beam is required to resist shear in the gusset region due to gravity load or unbalanced vertical components of brace forces.

Thus, the approximate minimum gusset length for the CSM is:

$$L_g \geq 1.25 \frac{M_{ch}}{\phi V_n} \tag{5}$$

If the gusset does not meet this minimum length, the engineer may use the explicit evaluation presented by Sabelli and Saxey.

If the USM is used to evaluate the gusset length, the internal connection forces are consistent with conventional plastic method, which is likely consistent with the gusset and weld design. If the evaluation is made using the CSM, internal forces will be similar to the optimized plastic method, and connection forces may need to be investigated.

Chevron beams with only two braces connecting at the midpoint (i.e., beams in stacked V or stacked inverted V frames) can generally be shown to be adequate using this CSM method due to the design of the beam for the axial force corresponding to the brace forces. However, the gusset, welds, and local beam-web limit states must also have sufficient strength to transfer the moment with a force couple:

$$R_u \geq \frac{M_{ch}}{e_z} \tag{6}$$

This force couple is transferred in two zones, each approximately $z = \frac{1}{2}(L_g - e_z)$ in length. See Sabelli and Saxey (2021) for additional guidance.

EXTERNAL MECHANISM

It is possible to realize additional strength in the connection by taking advantage of the flexural strength of the frame members external to the connection. The development of a full plastic mechanism corresponding to beam shear yielding requires rotation at the beam-to-column connections and rotation at each end of each brace. If moment can be resisted at these locations and in these members, additional resistance to the formation of the plastic mechanism can be realized in the system.

The following is a derivation of the complete plastic mechanism strength of both single-story V-type bracing and two-story X-type bracing. This procedure is recommended for verifying the capacity of existing connections. For new construction, providing a gusset of sufficient length such that the internal mechanism suffices is recommended over the calculation and coordination effort required to take advantage of any additional strength from brace flexure. It is expected that the majority of existing chevron connections have sufficient strength regardless of whether the chevron effect was explicitly checked in design.

Single-Story Frame

The plastic mechanism strength of a single-story chevron-braced frame is derived using the geometry shown in Figure 1. Point 1 is the center of the beam at midspan, point 2 is the point at which the brace crosses the gusset (the location of a potential plastic hinge), point 3 is the centerline of the beam at the end of the dimension e_x (the ends of the potential shear-yielding zone per Equation 4), and point 4 is the intersection of beam and column centerlines. Note that point 3 does not precisely align vertically with the gusset edge nor with point 2.

The complete plastic mechanism is shown in Figure 2, along with the associated rotations and displacements.

The proposed plastic mechanism entails shear yielding in the chevron zone, similar to the inelastic deformation of a shear governed eccentrically braced frame (EBF). As with the link in the EBF mechanism, the rotation of the gusset, θ_1, is centered on the original position of point 1, not the displaced position. As shown in Figure 2, the gusset merely rotates with the shear-deformed beam segment and is not required to yield for this mechanism to occur. A similar mechanism with flexural plastic hinges in the beam at each end of the gusset zone in lieu of shear yielding was not investigated.

To simplify the analysis, the shear yielding is assumed to result in rotation at each end of the length e_z, rather than at the end of the gusset. For conditions with sufficient weld strength and resistance to local web limit states, the dimension e_z will approach the full gusset length L_g using the CSM, and thus the assumption is reasonable. For conditions in which the maximum possible dimension e_z is significantly shorter than the gusset length L_g due to limitations of weld or web strength (and in which the system has insufficient strength considering the plastic mechanism analysis), shear yielding may be accompanied by failure of the weld or a local web limit state. In any case, however, sufficient plastic mechanism strength determined using the dimension e_z demonstrates adequacy, although it neglects some internal work corresponding to the separation of gusset and flange in the regions outside of e_z.

In addition to the work associated with shear yielding, the plastic mechanism engages flexural plastic hinges in the braces and in the beam at the beam-to-column connection. These members are subject to significant axial force,

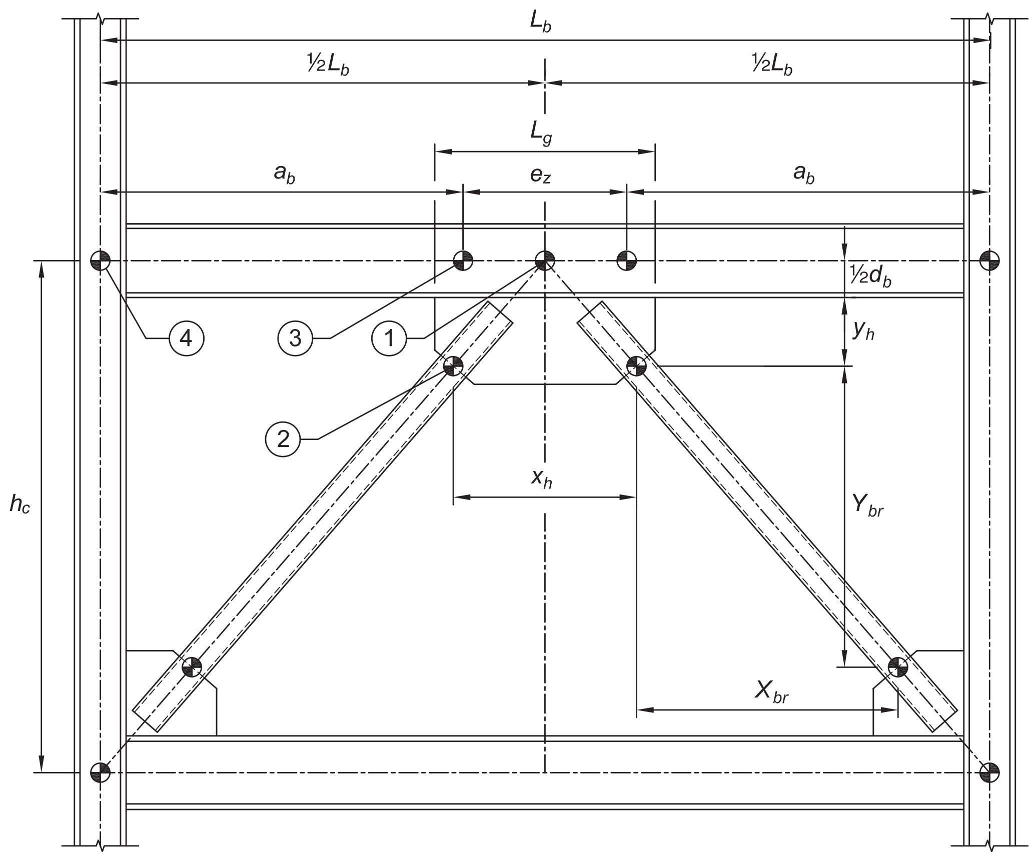

Fig. 1. Single-story chevron frame geometry.

and thus their flexural strength is reduced, as discussed in a later section.

From the geometry shown in Figure 1:

$$L_b = 2a_b + e_z \tag{7}$$

Depending on the connection details, use of the CSM may be justified, or, more conservatively, this moment arm can be set to that of the USM.

From Figure 2:

$$\Delta_{x1} = \frac{d_b}{2}\gamma \tag{8}$$

$$\Delta_{x4} = \Delta_{x1} \tag{9}$$

$$\Delta_{x4} = h_c\theta_4 \tag{10}$$

$$\Delta_{y2} = X_{br}\theta_2 \tag{11}$$

$$\Delta_{y2} = \frac{x_h}{2}\theta_1 \tag{12}$$

$$\Delta_{y3} = \frac{e_z}{2}\theta_1 \tag{13}$$

$$\Delta_{y3} = a_b\theta_3 \tag{14}$$

$$\gamma = \theta_1 + \theta_3 \tag{15}$$

From Equations 11 and 12:

$$\theta_2 = \frac{x_h}{2X_{br}}\theta_1 \tag{16}$$

From Equations 13 and 14:

$$\theta_3 = \frac{e_z}{2a_b}\theta_1 \tag{17}$$

From Equations 15 and 17:

$$\gamma = \left(1 + \frac{e_z}{2a_b}\right)\theta_1 \tag{18}$$

Using Equation 7, this simplifies to:

$$\gamma = \frac{L_b}{2a_b}\theta_1 \tag{19}$$

From Equations 8 and 19:

$$\Delta_{x1} = \frac{L_b d_b}{4a_b}\theta_1 \tag{20}$$

From Equations 9, 10, and 20:

$$\theta_4 = \frac{L_b d_b}{4a_b h_c}\theta_1 \tag{21}$$

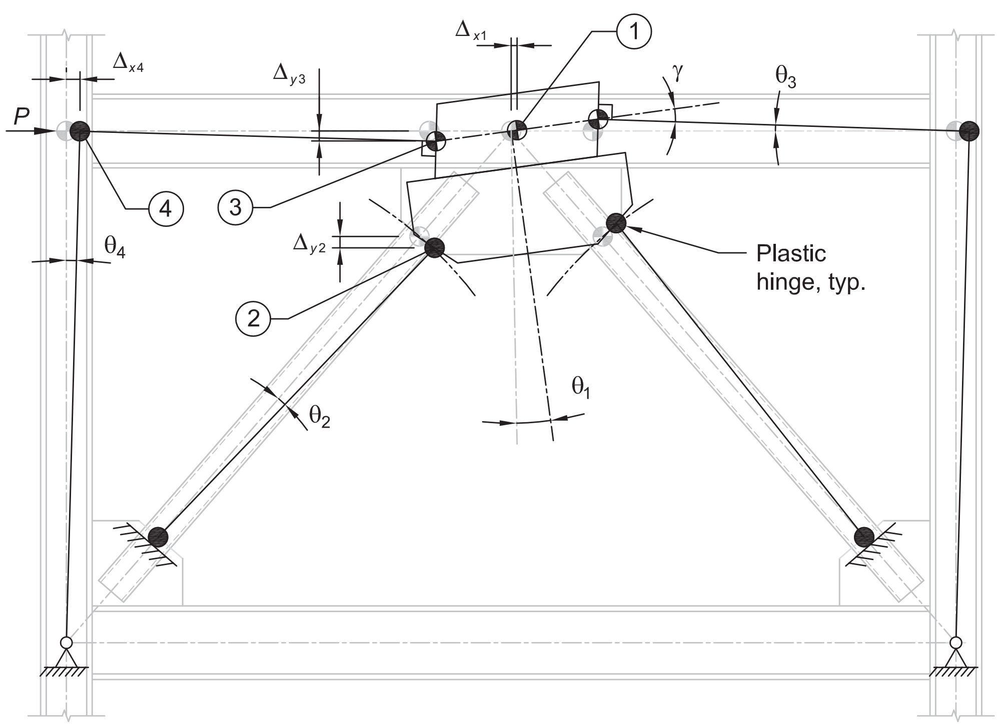

Fig. 2. Complete plastic mechanism for single-story chevron frame.

The external work applied to the frame is:

$$W_{external} = P\Delta_{x4} \tag{22}$$

which becomes:

$$W_{external} = P\frac{L_b d_b}{4a_b}\theta_1 \tag{23}$$

The internal work due to the frame mechanism is:

$$W_{internal} = \left[\gamma e_z V_n + 2|\theta_3 - \theta_4| M_{Pbm} + 2(2\theta_2 + \theta_1) M_{Pbr}\right] \tag{24}$$

where

M_{Pbm} = beam moment strength in the presence of axial force, kip-in

M_{Pbr} = brace moment strength in the presence of axial force, kip-in.

which becomes:

$$W_{internal} = \theta_1\left[\frac{L_b}{2a_b}e_z V_n + \frac{e_z}{a_b}\left|1 - \frac{L_b d_b}{2e_z h_c}\right| M_{Pbm} + 2\left(\frac{x_h}{X_{br}} + 1\right) M_{Pbr}\right] \tag{25}$$

For $W_{external} = W_{internal}$:

$$P\frac{d_b L_b}{4a_b}\theta_1 = \left[\frac{L_b e_z}{2a_b}V_n + \frac{e_z}{a_b}\left|1 - \frac{L_b d_b}{2e_z h_c}\right| M_{P_{bm}} + 2\left(\frac{x_h}{X_{br}} + 1\right) M_{P_{br}}\right]\theta_1 \tag{26}$$

Solving for P:

$$P = \begin{bmatrix} \dfrac{2e_z}{d_b}V_n \\ +4\left|\dfrac{e_z}{d_b L_b} - \dfrac{2}{h_c}\right| M_{Pbm} \\ +\dfrac{8a_b}{L_b d_b}\left(\dfrac{x_h}{X_{br}} + 1\right) M_{Pbr} \end{bmatrix} \tag{27}$$

This lateral force corresponds to yielding of the beam web in the connection region, as well as rotation of the brace ends and beam-to-column connection. If this lateral force is less than the required frame strength (or the capacity of the braces for seismic design), the chevron mechanism does not occur.

The beam moment, M_{Pbm}, is limited by the moment capacity of the beam end connection. This term can be neglected from Equation 27 to avoid having to check the moment capacity of the beam connection. If the plastic mechanism strength of the braces alone is insufficient to resist the forces, this term can be taken into consideration. The braces and their connections should be evaluated for the moment being transferred to them.

Two-Story Frame

The two-story plastic mechanism strength is derived using the geometry shown in Figures 3 and 4. The two-story frame has two differences compared to the single story. First, the internal work of the frame is supplemented by two additional braces. The beam contributions from both shear yielding at the chevron connection and flexural hinges at the column connection are the same as in the single-story case. Second, the external work contains contributions from loads at each of the two stories.

The external work applied to the frame is:

$$P_{ef} = P_1 + P_2\left(\frac{h_{c1} + h_{c2}}{h_{c1}}\right) \tag{29}$$

$$W_{external} = P_{ef}\Delta_{x4} \tag{30}$$

Combining Equations 10, 21, 29, and 30:

$$W_{external} = P_{ef}\frac{d_b L_b}{4a_b}\theta_1 \tag{31}$$

The internal work due to the frame mechanism is similar to that for the one-story mechanism (Equation 25), with the exception that two additional braces participate:

$$W_{internal} = \theta_1\left[\frac{L_b e_z}{2a_b}V_n + \frac{e_z}{a_b}\left|1 - \frac{L_b d_b}{2e_z h_c}\right| M_{Pbm} + 2\left(\frac{x_{h1}}{X_{br1}} + 1\right) M_{Pbr1} + 2\left(\frac{x_{h2}}{X_{br2}} + 1\right) M_{Pbr2}\right] \tag{32}$$

Thus

$$P_{ef} = \begin{bmatrix} \dfrac{2e_z}{d_b}V_n \\ +4\left|\dfrac{e_z}{d_b L_b} - \dfrac{2}{h_c}\right| M_{Pbm} \\ +\dfrac{8a_b}{L_b d_b}\left(\dfrac{x_{h1}}{X_{br1}} + 1\right) M_{Pbr1} \\ +\dfrac{8a_b}{L_b d_b}\left(\dfrac{x_{h2}}{X_{br2}} + 1\right) M_{Pbr2} \end{bmatrix} \tag{33}$$

As with the one-story mechanism, this lateral force is compared to the forces corresponding to the required frame strength (or the capacity of the braces for seismic design).

Approximate Method

While Equations 27 and 33 are not complicated, for many cases, the contribution from the beam is fairly small, and the distinction between certain horizontal dimensions has negligible effect. A simpler version of Equation 27 can produce conservative values for rapid preliminary checks.

Setting

$$X_{br} \approx a_b \tag{34}$$

and

$$x_h \approx e_z \tag{35}$$

the amplification of the brace end moment becomes negligible for realistically small values of x_h/L_b.

Setting

$$M_{Pbm} = 0$$

for the one-story mechanism, the force that would cause the chevron mechanism is:

$$P \geq \frac{2e_z}{d_b}V_n + \frac{8}{d_b}\left(1 - \frac{a_b}{L_b}\right)M_{Pbr} \tag{36}$$

which can be conservatively reduced to:

$$P \geq \frac{2e_z}{d_b}V_n + \frac{4}{d_b}M_{Pbr} \tag{37}$$

If the lateral force does not exceed this value, the connection is adequate. A somewhat greater strength can be obtained form Equation 27.

For the two-story mechanism, the equivalent force per Equation 30 that would cause the chevron mechanism is:

$$P_{ef} \geq \frac{2e_z}{d_b}V_n + \frac{4}{d_b}\left(M_{Pbr1} + M_{Pbr2}\right) \tag{38}$$

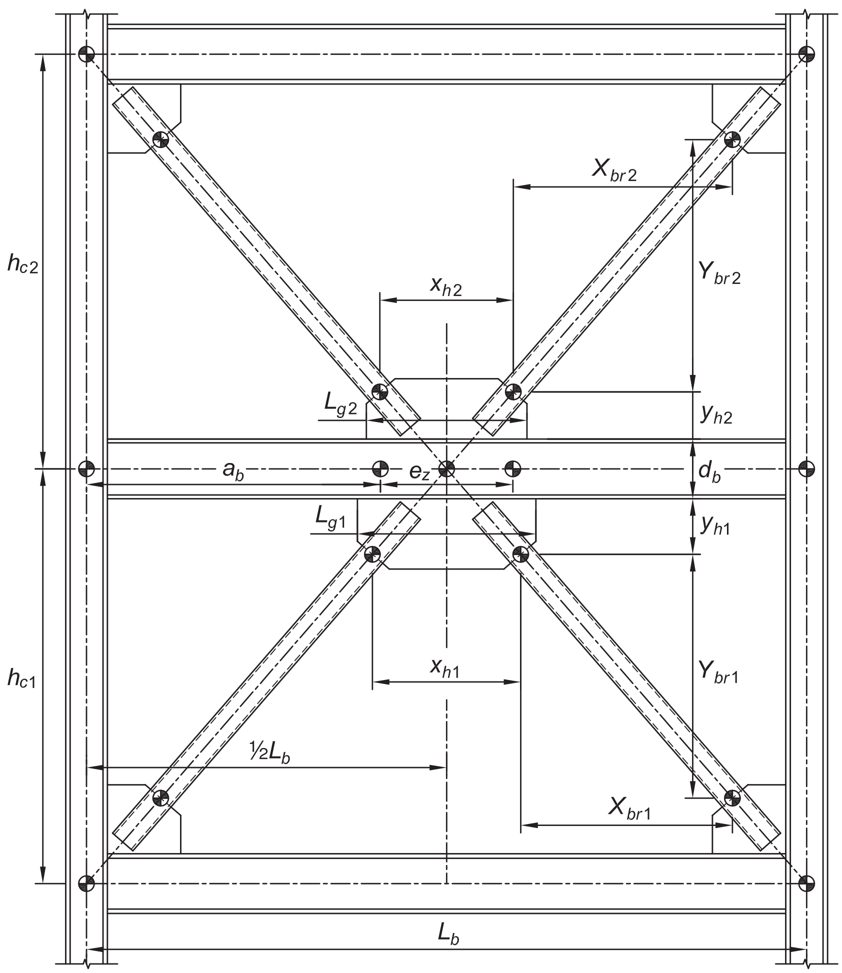

Fig. 3. Two-story chevron frame geometry.

AVAILABLE FLEXURAL STRENGTH OF BRACES

The flexural strengths in the presence of axial force may be determined using AISC *Specification* Chapter H. Generally, braces will have axial forces such that AISC *Specification* Equation H1-1b will not apply. Equation H1-1a can be rewritten to solve for the available moment strength per *Specification* H1.3(a):

$$M_r \le \frac{9}{8}\left(1-\frac{P_r}{P_c}\right)\phi M_{px} \tag{39}$$

Equation H1-3 can be rewritten to solve for this available moment strength:

$$M_r \le C_b M_{cx}\sqrt{1.0-1.5\frac{P_r}{P_{cy}}+0.5\left(\frac{P_r}{P_{cy}}\right)^2} \tag{40}$$

For the mechanism in question, the brace undergoes reverse curvature with plastic hinging at each end. For this moment diagram:

$$\begin{aligned} C_b &= \frac{12.5M_{max}}{2.5M_{max}+3M_A+4M_B+3M_C} \\ &= \frac{12.5M_u}{2.5M_u+3(0.75M_u)+4(0)+3(0.75M_u)} \\ &= 1.79 \end{aligned}$$

(*Spec*. Eq. F1-1)

For a preliminary evaluation, the brace flexural strength in the presence of axial force may be estimated as 25% of the full brace flexural strength. (This would correspond to approximately 75% axial utilization in Equation 39).

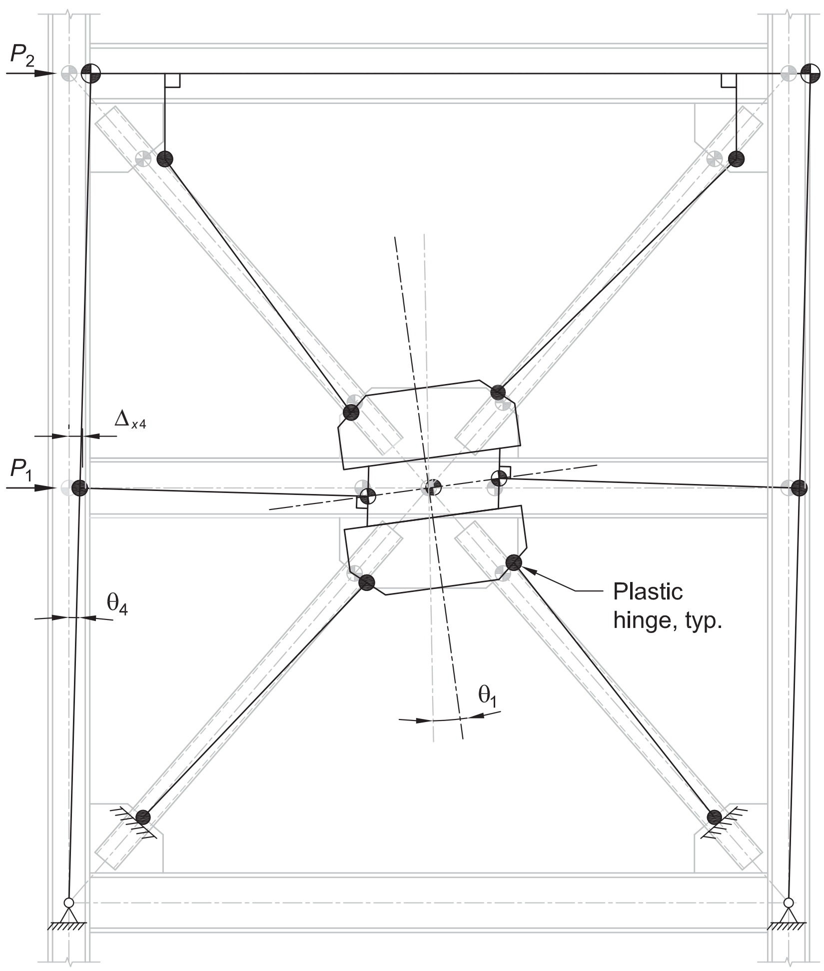

Fig. 4. External mechanism for two-story chevron frame.

EXAMPLE EVALUATION

Given:

The plastic mechanism strength will be used to evaluate an existing two-story chevron brace connection for which the available beam strength does not meet the required "chevron effect" forces determined using the Concentrated Stress Method. The connection detail and geometry of the frame are shown in Figures 5 and 6, respectively. The following brace dimensions are given for this example: X_{br1} = 162 in. and X_{br2} = 164 in. The brace forces and connection forces are summarized in Tables 1 and 2, respectively. Design is given for LRFD only. Both the beam and gusset plate are Grade 50 material. The braces are ASTM A500 Gr. C (F_y = 50 ksi).

Solution:

From AISC *Manual* Table 1-1:

W21×55
d = 20.8 in.
k_{des} = 1.02 in.
t_f = 0.522 in.
t_w = 0.375 in.

From AISC *Manual* Table 6-1, for a W21×55:

ϕV_n = 234 kips

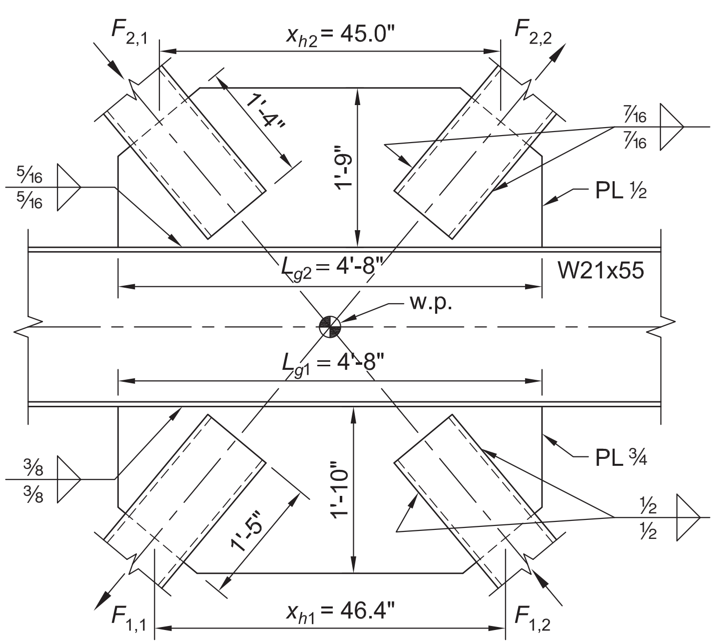

Fig. 5. Chevron connection detail.

Table 1. Summary of Brace Forces				
Brace	Brace Size	Brace Axial Force, *F*, kips	Shear Component, *F*(cosγ), kips	Normal Component, *F*(sinγ), kips
F1,1	HSS10×10×⅝	586	375	450
F1,2	HSS10×10×⅝	586	375	450
F2,1	HSS10×10×½	390	250	300
F2,2	HSS10×10×½	390	250	300

Table 2. Summary of Connection Forces			
	Gusset 1	Gusset 2	Combination (or Difference)
F_V, kips	750	500	1250
F_N, kips	0	0	0
M_f, kip-in.	7800	5200	13000

Concentrated Stress Method

First, the approximate method is attempted:

$$L_g \geq 1.25\frac{M_{ch}}{\phi V_n} \tag{5}$$

$$= 1.25\left(\frac{13{,}000 \text{ kip-in.}}{234 \text{ kips}}\right)$$

$$= 69.4 \text{ in.} > 56.0 \text{ in.} \quad \textbf{n.g.}$$

The gusset does not meet this requirement. The explicit method from Sabelli and Saxey (2021) is attempted for the bottom gusset.

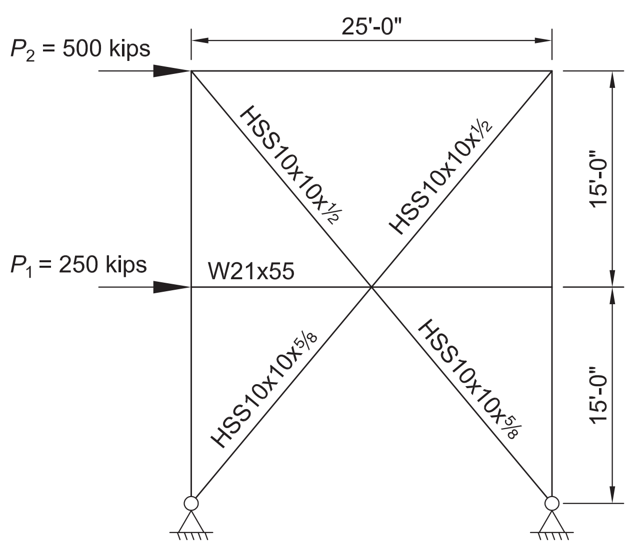

Fig. 6. Frame geometry.

From Sabelli and Saxey Equation 56:

$$V_{efTot} = \phi V_n - \left| \frac{F_{N1}}{2} - \frac{F_{N2}}{2} \right| - |V_m|$$
$$= 234 \text{ kips} - 0 - 0$$
$$= 234 \text{ kips}$$

From Sabelli and Saxey Equation 51:

$$V_{ef} = \frac{M_f}{M_{Tot}} V_{efTot}$$
$$= \left(\frac{7{,}800 \text{ kip-in.}}{13{,}000 \text{ kip-in.}} \right) (234 \text{ kips})$$
$$= 140 \text{ kips}$$

From Sabelli and Saxey Equation 39:

$$L_g > \frac{M_f}{V_{ef}} + \frac{V_{ef}}{\phi F_y t_g}$$
$$= \frac{7{,}800 \text{ kip-in.}}{140 \text{ kips}} + \frac{140 \text{ kips}}{0.90(50 \text{ ksi})(0.75 \text{ in.})}$$
$$= 59.9 \text{ in.} > 56.0 \text{ in.} \quad \textbf{n.g.}$$

A more precise (and less conservative) evaluation can be made by computing the largest possible value of the dimension e_z considering local web and gusset-yield limit states. Considering web local yielding, the minimum length, z, is determined using Sabelli and Saxey Equation 40:

$$z \geq \frac{L_g}{2} - \sqrt{\frac{{L_g}^2}{4} - \frac{M_f}{\phi_w F_y t_w}} - 5k$$
$$= \frac{56.0 \text{ in.}}{2} - \sqrt{\frac{(56.0 \text{ in.})^2}{4} - \frac{7{,}800 \text{ kip-in.}}{(1.00)(50 \text{ ksi})(0.375 \text{ in.})}} - 5(1.02 \text{ in.})$$
$$= 3.72 \text{ in.}$$

Considering web crippling, the minimum length, z, is determined using Sabelli and Saxey Equation 42:

$$z \geq \left[\frac{V_{ef}}{\phi_n 0.80 t_w^2} \sqrt{\frac{t_w}{E F_y t_f}} - 1 \right] \left(\frac{d_m}{3} \right) \left(\frac{t_f}{t_w} \right)^{1.5}$$
$$= \left[\frac{140 \text{ kips}}{(0.75)(0.80)(0.375 \text{ in.})^2} \sqrt{\frac{0.375 \text{ in.}}{(29{,}000 \text{ ksi})(50 \text{ ksi})(0.522 \text{ in.})}} - 1 \right] \left(\frac{20.8 \text{ in.}}{3} \right) \left(\frac{0.522 \text{ in.}}{0.375 \text{ in.}} \right)^{1.5}$$
$$= 1.91 \text{ in.}$$

Considering gusset yield, the minimum length, z, is determined using Sabelli and Saxey Equation 41:

$$z = \frac{L_g}{2} - \sqrt{\frac{{L_g}^2}{4} - \frac{M_f \big/ \phi_t}{\sqrt{\left(F_y t_g\right)^2 - \left(\frac{F_V}{\phi_v 0.60 L_g}\right)^2}}}$$

$$= \frac{56.0 \text{ in.}}{2} - \sqrt{\frac{(56.0 \text{ in.})^2}{4} - \frac{\left(7{,}800 \text{ kip-in.} \big/ 0.90\right)}{\sqrt{\left[(50 \text{ ksi})(0.75 \text{ in.})\right]^2 - \left[\frac{750 \text{ kips}}{(1.0)(0.60)(56.0 \text{ in.})}\right]^2}}}$$

$$= 5.72 \text{ in.}$$

Using the maximum length $z = 5.72$ in.:

$$e_z = L_g - 2z \tag{4}$$

$$= 56.0 \text{ in.} - 2(5.72 \text{ in.})$$

$$= 44.6 \text{ in.}$$

The reaction corresponding to this moment arm is:

$$R_u \geq \frac{M_{ch}}{e_z} \tag{6}$$

$$= \frac{13{,}000 \text{ kip-in.}}{44.6 \text{ in.}}$$

$$= 291 \text{ kips}$$

$$R_u = 291 \text{ kips} > V_{ef} = 140 \text{ kips} \quad \textbf{n.g.}$$

The CSM evaluation is discontinued at this point. If this check indicated adequacy, the other (top) gusset would be similarly evaluated, as would the weld and the local limit states of web local yielding and web crippling. See Sabelli and Saxey (2021) for additional information on the Concentrated Stress Method.

Plastic Method Strength

The required lateral force based on the frame loading is:

$$P_{ef} = P_1 + P_2\left(\frac{h_{c1} + h_{c2}}{h_{c1}}\right) \tag{29}$$

$$= 250 \text{ kips} + 500 \text{ kips}\left(\frac{180 \text{ in.} + 180 \text{ in.}}{180 \text{ in.}}\right)$$

$$= 1{,}250 \text{ kip-in.}$$

$$V_p = V_{efTot}$$

$$= 234 \text{ kips}$$

Using the USM method:

$$e_z = 0.5 L_g$$

$$= 0.5(56.0 \text{ in.})$$

$$= 28.0 \text{ in.}$$

$$
\begin{aligned}
a_b &= \frac{L_b - e_z}{2} \\
&= \frac{300 \text{ in.} - 28.0 \text{ in.}}{2} \\
&= 136 \text{ in.}
\end{aligned} \tag{7}
$$

The flexural strength of the braces is checked in the presence of axial force. For the bottom and top braces, the available flexural and axial strengths are determined using AISC *Manual* Tables 3-13 and 4-4, respectively. The effective length for compression is conservatively assumed to be $L_c = 20$ ft.

Bottom braces (HSS10×10×⅝):

$\phi_b M_n = 275$ kip-ft

$\phi_c P_n = 706$ kips

Top braces (HSS10×10×½)

$\phi_b M_n = 228$ kip-ft

$\phi_c P_n = 583$ kips

The flexural strength of the braces is determined using Equations 39 and 40.

Bottom braces:

$$
\begin{aligned}
M_r &\le \frac{9}{8}\left(1 - \frac{P_r}{P_{cx}}\right)\phi M_{px} \\
&= \frac{9}{8}\left(1 - \frac{586 \text{ kips}}{706 \text{ kips}}\right)(275 \text{ kip-ft}) \\
&= 52.6 \text{ kip-ft} \\
&= 631 \text{ kip-in.}
\end{aligned} \tag{39}
$$

$$
\begin{aligned}
M_r &\le C_b M_{cx} \sqrt{1.0 - 1.5\frac{P_r}{P_{cx}} + 0.5\left(\frac{P_r}{P_{cx}}\right)^2} \\
&= 1.79(275 \text{ kip-in.})\sqrt{1.0 - 1.5\left(\frac{586 \text{ kips}}{706 \text{ kips}}\right) + 0.5\left(\frac{586 \text{ kips}}{706 \text{ kips}}\right)^2} \\
&= 155 \text{ kip-ft} \\
&= 1{,}860 \text{ kip-in.}
\end{aligned} \tag{40}
$$

Use $M_r = 631$ kip-in.

Top braces:

$$
\begin{aligned}
M_r &\le \frac{9}{8}\left(1 - \frac{P_r}{P_{cx}}\right)\phi M_{px} \\
&= \frac{9}{8}\left(1 - \frac{390 \text{ kips}}{583 \text{ kips}}\right)(228 \text{ kip-ft}) \\
&= 84.9 \text{ kip-ft} \\
&= 1{,}020 \text{ kip-in.}
\end{aligned} \tag{39}
$$

$$M_r \le C_b M_{cx}\sqrt{1.0-1.5\frac{P_r}{P_{cx}}+0.5\left(\frac{P_r}{P_{cx}}\right)^2} \tag{40}$$

$$= 1.79(228 \text{ kip-in.})\sqrt{1.0-1.5\left(\frac{390 \text{ kips}}{583 \text{ kips}}\right)+0.5\left(\frac{390 \text{ kips}}{583 \text{ kips}}\right)^2}$$

$$= 192 \text{ kip-ft}$$

$$= 2{,}300 \text{ kip-in.}$$

Use M_r = 1,020 kip-in.

$$P_{ef} = \frac{2e_z}{d_b}V_n + 4\left|\frac{e_z}{d_b L_b} - \frac{2}{h_c}\right| M_{p_{bm}} + \frac{8a_b}{L_b d_b}\left(\frac{x_{h1}}{X_{br1}}+1\right)M_{p_{br1}} + \frac{8a_b}{L_b d_b}\left(\frac{x_{h2}}{X_{br2}}+1\right)M_{p_{br2}} \tag{33}$$

$$= \frac{2(28.0 \text{ in.})}{20.8 \text{ in.}}(234 \text{ kips}) + 0 + \frac{8(136 \text{ in.})}{(300 \text{ in.})(20.8 \text{ in.})}\left(\frac{46.4 \text{ in.}}{162 \text{ in.}}+1\right)(631 \text{ kip-in.})$$

$$+ \frac{8(136 \text{ in.})}{(300 \text{ in.})(20.8 \text{ in.})}\left(\frac{45.0 \text{ in.}}{164 \text{ in.}}+1\right)(1{,}020 \text{ kip-in.})$$

$$= 630 \text{ kips} + 0 + 142 \text{ kips} + 227 \text{ kips}$$

$$= 999 \text{ kips} < 1{,}250 \text{ kips} \quad \textbf{n.g.}$$

Try the CSM method:

$$e_z = 44.6 \text{ in.}$$

$$a_b = \frac{L_b - e_z}{2} \tag{7}$$

$$= \frac{300 \text{ in.} - 44.6 \text{ in.}}{2}$$

$$= 128 \text{ in.}$$

$$P_{ef} = \frac{2e_z}{d_b}V_n + 4\left|\frac{e_z}{d_b L_b} - \frac{2}{h_c}\right| M_{p_{bm}} + \frac{8a_b}{L_b d_b}\left(\frac{x_{h1}}{X_{br1}}+1\right)M_{p_{br1}} + \frac{8a_b}{L_b d_b}\left(\frac{x_{h2}}{X_{br2}}+1\right)M_{p_{br2}} \tag{33}$$

$$= \frac{2(44.6 \text{ in.})}{20.8 \text{ in.}}(234 \text{ kips}) + 0 + \frac{8(128 \text{ in.})}{(300 \text{ in.})(20.8 \text{ in.})}\left(\frac{46.4 \text{ in.}}{162 \text{ in.}}+1\right)(631 \text{ kip-in.})$$

$$+ \frac{8(128 \text{ in.})}{(300 \text{ in.})(20.8 \text{ in.})}\left(\frac{45.0 \text{ in.}}{164 \text{ in.}}+1\right)(1{,}020 \text{ kip-in.})$$

$$= 1{,}000 \text{ kips} + 0 + 133 \text{ kips} + 213 \text{ kips}$$

$$= 1{,}350 \text{ kips} > 1{,}250 \text{ kips} \quad \textbf{o.k.}$$

The flexural strength provided by the braces is adequate to supplement the beam shear strength. Assuming the gusset and its welds are adequate to transfer the moment with the moment arm of e_z, the deficiency is only 1,250 kips − 1,000 kips = 250 kips. This is 250 kips/(133 kips + 213 kips) = 72% of the brace flexural strength determined earlier. Braces and their connections should be evaluated for this moment (in combination with the required axial strength) to show adequacy, as well as the corresponding shear.

For the brace-to-gusset welds at the bottom gusset plate:

$$M_u = 0.72(631 \text{ kip-in.})$$
$$= 454 \text{ kip-in.}$$

$$P_{eq} = P_u + 2\frac{M_u}{d}$$
$$= 586 \text{ kips} + 2\left(\frac{454 \text{ kip-in.}}{10 \text{ in.}}\right)$$
$$= 677 \text{ kips}$$

$$R_n = 1.392DL$$
$$= 1.392(8)(17 \text{ in.})(4 \text{ welds})$$
$$= 757 \text{ kips} > 677 \text{ kips} \quad \textbf{o.k.}$$

For the brace-to-gusset welds at the top gusset plate:

$$M_u = 0.72(1{,}020 \text{ kip-in.})$$
$$= 734 \text{ kip-in.}$$

$$P_{eq} = P_u + 2\frac{M_u}{d}$$
$$= 390 \text{ kips} + 2\left(\frac{734 \text{ kip-in.}}{10 \text{ in.}}\right)$$
$$= 537 \text{ kips}$$

$$R_n = 1.392DL$$
$$= 1.392(7)(16 \text{ in.})(4 \text{ welds})$$
$$= 624 \text{ kips} > 537 \text{ kips} \quad \textbf{o.k.}$$

The gusset connection to the beam flange must be able to transmit the force R_z over the length z.

LIMITATIONS AND FURTHER STUDY

The plastic mechanism strengths derived in this study have not been subject to verification by test or by nonlinear frame analysis. As such, they invite further investigation of the response of frames—in particular, the local behavior that follows the extension of shear yield beyond the length e_z. Additionally, the effects of the rotation capacity of different shapes would further inform the understanding of this mechanism.

CONCLUSIONS

The beam shear strength of an existing chevron bracing connections is typically acceptable whether or not the chevron effect was taken into consideration during design. If an existing chevron connection is deemed inadequate using the USM, additional strength may be found using the CSM. Alternatively, the complete plastic mechanism strength presented in this study can be used to confirm the adequacy of designs that did not consider the chevron effect. For this purpose, both complete and approximate equations are provided. Evaluations utilizing the CSM or the complete plastic mechanism should include assessment of the elements engaged to resist the chevron moment. For the CSM, this includes evaluation of the gusset and its weld for the stress concentrated at each end. For the plastic mechanism, this includes evaluation of the braces and their connections for the combination of axial force and moment. An example shows a frame for which the CSM considered on its own indicates significant insufficiency, but the complete plastic mechanism indicates adequacy.

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SYMBOLS LIST

The following is a list of symbols used in this paper.

F_{br} Brace axial force, kips

L_b Beam span, in.

L_g Gusset plate length, in.

M_{ch} Chevron moment, kip-in.

M_{Pbm} Beam moment strength in the presence of axial force, kip-in.

M_{Pbr} Brace moment strength in the presence of axial force, kip-in.

M_{Pbr1}, M_{Pbr2} For two-story frame, the brace moment strength in the presence of axial force for top and bottom braces, respectively, kip-in.

M_{px} Major axis plastic moment strength, kip-in.

M_r Required flexural strength, kip-in.

M_u Required flexural strength. kip-in.

P Horizontal force applied to the single-story frame, kips

P_c Available axial strength, kips

P_{cy} Available axial strength for out-of-plane flexural buckling, kips

P_{ef} Effective horizontal force applied to the two-story frame, kips

P_r Required axial strength, kips

P_1 Horizontal force applied to the first story of a two-story frame, kips

P_2 Horizontal force applied to the second story of a two-story frame, kips

V_u Beam shear, kips

$W_{internal}$ Internal work due to frame action, kip-in.

$W_{external}$ External work applied to the frame, kip-in.

X_{br} Horizontal distance between points where the brace ends cross the gusset plates, in.

X_{br1}, X_{br2} For two-story frame, the horizontal distance between points where the brace ends cross the gusset plates of bottom and top braces, respectively, in.

Y_{br} Vertical distance between points where the brace ends cross the gusset plates, in.

a_b Length of beam from column centerline to e_z region, in.

d_b Depth of beam, in.

e_z Length of moment arm, in.

h_c First-story frame height, in.

x_h Horizontal distance from control point 2 (see Figure 1) of braces, in.

x_{h1}, x_{h2} For a two-story frame, the horizontal distance from control point 2 (see Figure 1) of bottom and top chevron connections, respectively, in.

y_h Vertical distance from beam flange to control point 2 (see Figure 1), in.

z Length required to transfer V_u from gusset to beam considering weld strength, gusset strength, web local yielding, and web crippling, in.

Δ_{x1} Horizontal displacement of control point 1 (see Figure 2), in.

Δ_{x4} Horizontal displacement of control point 4 (see Figure 2), in.

Δ_{y2} Vertical displacement of control point 2 (see Figure 2), in.

Δ_{y3} Vertical displacement of control point 3 (see Figure 2), in.

γ Shear angle of beam (see Figure 2)

θ Brace angle, with respect to the horizontal, degrees

θ_1 Rotation (see Figure 2)

θ_2 Rotation (see Figure 2)

θ_3 Rotation (see Figure 2)

θ_4 Rotation (see Figure 2)

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