Investigation on the Performance of a Mathematical Model to Analyze Concentrically Braced Frame Beams with V-Type Bracing Configurations

ALIREZA ASGARI HADAD and PATRICK J. FORTNEY

In memory of Patrick J. Fortney, who passed away in October 2019.

ABSTRACT

The chevron effect and the corresponding mathematical model used to predict the beam shear force and bending moment demands on beams in concentrically braced frames with V- or inverted V-type (also known as chevrons) configurations is studied in this paper. The common analysis approach considers beam span, work point location, and a concentrated force representing the unbalanced vertical forces of the braces while ignoring any local effects resulting from the brace connection geometry. The assumptions and load distribution mechanisms in the connection region that have been discussed in earlier literature are presented, and the performance of the mathematical model based on chevron effect (CE method) is examined by comparing its results with the results of a net vertical force method (NVF) in addition to those obtained by finite element analysis. The analysis procedure recently proposed for beams in chevron concentrically braced frames is used to design the beams in a group of 20 beam-gusset assemblies. The results revealed the presence of the chevron effect in chevron beams. It also showed the CE method is able to estimate the beam maximum shear force and bending moment. Additionally, the stress distribution along the gusset-to-beam interface is studied.

Keywords: beam, shear force, bending moment, concentrically braced frame, chevron, connection, stress distribution.

INTRODUCTION

Braced frames (concentric and eccentric), moment resisting frames, special truss moment frames, and steel plate shear walls are commonly used as lateral-force-resisting systems in steel structures. While new systems such as buckling restrained braced frames are gaining popularity, concentrically braced frames (CBFs) and moment resisting frames (MRFs) are considered as two of the most popular systems among these alternatives (Azad et al., 2017). CBFs have been quite popular since the 1960s mainly because of their economic advantages over the MRFs, particularly in cases where the drift requirements govern the design and higher stiffness and strength are required (Khatib et al., 1988; Nascimbene et al., 2012; Azad et al., 2017). Studies on the seismic response of braced frames progressed slowly until the 1970s. The expansion of offshore oil exploration in regions of seismic risk stimulated analytical and experimental investigations of the inelastic cyclic response of tubular steel–braced structures. At the same time, evidence was accumulating that buildings with braced frames were being damaged during major earthquakes. Such events motivated an increasing number of studies on the inelastic behavior of steel braced frames both analytically and experimentally (Khatib et al., 1988; Bertero et al., 1989; Lee and Lu, 1989; Wallace and Krawinkler, 1989; Tremblay, 2002; Sabelli et al., 2003; Tremblay et al., 2003; Tremblay, 2008). Special concentrically braced frames (SCBFs) were developed after 1988; they are employed in high seismic regions and use a response modification factor, *R*, to reduce the seismic design force. During large, infrequent earthquakes, SCBFs must sustain cyclic, inelastic tensile yielding and compressive buckling deformation of the brace without significant deterioration of stiffness and resistance for earthquakes exceeding the reduced design force (Sen et al., 2016). Brace buckling and tensile yield are the primary yield mechanisms of the system, which permit the frame to sustain the inelastic deformation and energy dissipation demands needed to provide collapse-prevention performance (Osteraas and Krawinkler, 1989; Naeim, 1998; Roeder et al., 2011; Sen et al., 2016).

For CBFs with buckling braces, the braces are used in opposing pairs to balance the difference in the tensile and compressive capacities. This difference is pronounced in chevron-configured CBFs. In addition to the chevron configuration for CBFs, also known as V- or inverted V-type concentrically braced frames, X-CBFs with braces spanning

Alireza Asgari Hadad, Graduate student, University of Cincinnati, Cincinnati, Ohio. Email: asgariaa@mail.uc.edu (corresponding)

Patrick J. Fortney, Associate Professor-Educator, University of Cincinnati, Cincinnati, Ohio.
In memory of Patrick J. Fortney, who passed away in October 2019.

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two consecutive floors meeting at one point in the middle beam were also used in CBFs. An inventory of older CBF buildings shows that the chevron brace configuration was common (Zhang et al., 2011; Sloat, 2014; Sen et al., 2016). This is logical because chevron bracing accommodates a wide range of architectural elements, including doors and windows, and fewer brace connections with respect to X-CBFs (Sen et al., 2016; Costanzo et al., 2017). Chevron-braced bays were found to fail in one of two mechanisms: (1) A weak beam mechanism where the buckling of the compression brace causes an unbalanced force to be applied on the beam and subsequently the formation of plastic hinges in the beam. Weak beams lead to considerable plastification in columns, beams, and braces, irrespective of brace slenderness. Slender braces increase the ductility demand in all remaining members, and rarely yield in tension. (2) A strong beam mechanism where buckling of the compression brace is followed by yielding of the tension brace, the beam being sufficiently strong to remain elastic despite the vertical unbalanced force applied to it by the braces at mid-span. Chevron-braced frames with strong beams and weak braces exhibit a more uniform distribution of yielding with height than frames with weak beams and strong braces (D'Aniello et al., 2015). It is not evident from the data in the literature which collapse mechanism is preferable. The weak beam mechanism concentrates the energy dissipation in the beams that are better energy dissipators than the braces. However, it causes a deteriorating force deformation curve, which leads to more energy dissipation demand. The strong beam mechanism provides a trilinear force deformation characteristic that is believed by many to be an ideal force deformation characteristic. On the other hand, it concentrates the energy dissipation in the braces (the least capable elements) and may require unreasonably stiff and strong beams. In addition, it increases the maximum column compression and hence the danger of column buckling. This is a very undesirable situation in that it might lead to column failure, soft-story formation, or incremental collapse (Roeder and Popov, 1978; Rega and Vestroni, 1984; Shen et al., 2015). The mentioned uncertainties regarding the demands on the beam in chevron CBFs with a V- or X-bracing configuration jeopardize the support of the floor and thus make chevron V- and X-bracing undesirable bracing configurations (ASCE, 1998).

In addition to the system behavior in CBFs, brace connection analysis and design and calculation of the connection interface forces have been a subject for several numerical and experimental studies (Thornton, 1984; Astaneh-Asl et al., 1985; Bjorhovde and Chakrabarti, 1985; Richard, 1986; Willibald et al., 2006; Shaw et al., 2010; Wigle and Fahnestock, 2010). The uniform force method (UFM) is a popular method typically used to distribute brace forces through the corner connections (Thornton, 1991). However, this method is limited to connections where braces are framed to a beam-column joint. There is very little published work on how to distribute forces in other types of brace configuration. Fortney and Thornton (2015; 2017) studied CBFs with V- and inverted V-type brace configurations. They discussed the behavior of frame beams through a phenomenon called the chevron effect and developed a mathematical method to, initially, determine an admissible distribution of forces acting on the beam-to-gusset interface and, secondly, determine the beam shear force and bending moment in the connection region. It is worth noting that the chevron effect model does not consider any stiffening effect of the gusset plate for the beam. This assumption relies on the results of several experimental tests revealing the out-of-plane bending of the gusset plate when the compression brace buckles (Zhang et al., 2011; Lumpkin et al., 2012; Okazaki et al., 2012; Hsiao et al., 2013; Salawdeh et al., 2017). The mentioned mathematical model will be discussed, and its performance will be studied and compared with finite element model results in the following sections.

MATHEMATICAL MODEL FOR CHEVRON EFFECT

The chevron effect is a phenomenon that affects the demands on the frame beam imparted by the distribution of forces at the beam/gusset plate interface. The influence of these interface forces on the beam demands was initially introduced by Fortney and Thornton (2015). The chevron effect is present in CBFs with V- and inverted V-type brace configurations where braces are connected to the frame beam with a gusset plate. The net vertical force (NVF) method is the common method used to analyze the frame beam in such braced frames. However, this method only considers the center line of the columns, beams, and braces and applies the summation of the vertical components of the brace forces to the work point (the point in which the center lines of the braces meet the centerline of the beam) as concentrated load. Fortney and Thornton (2015) developed a mathematical model assuming a uniform distribution of connection interface forces and moments to calculate the shear force and bending moment on the beam. They showed that the presence of a gusset plate influences the force distribution in the connection region and referred to this phenomenon as the chevron effect. Fortney and Thornton (2017) showed the chevron effect results in localized beam shear and moment demands on the frame beam that are not captured by the NVF. Figure 1 presents the chevron effect and the difference between the results of the mathematical model (CE method) and the NVF method for the case in which the tension and compression braces have equal forces (i.e., zero NVF).

The CE method proposed by Fortney and Thornton (2015;

2017) to analyze the beam in a chevron brace frame does not consider the gusset plate as a stiffening element for the beam. So the flexural deformation of the beam is not influenced by the gusset plates in the connection region. This assumption is based on the instability of the gusset plate at the free edges and the out-of-plane deformation of the gusset plate during brace buckling. The out-of-plane behavior of the gusset plate is observed in several studies. Zhang et al. (2011) studied nine inverted V-braces (W-shapes) and their gusset plate connections under inelastic cyclic loading to define the hysteretic behavior of the assemblies. As can be seen in Figure 2, even the presence of stiffener plates does not impede the out-of-plane response of the gusset plate [see Figure 2(b)].

While the cyclic and monotonic response of isolated components such as braces and gusset plates has been the subject of several studies, the seismic response of two three-story SCBFs was studied to investigate the performance of their structural components when the components work interactively (Lumpkin et al., 2012). The gusset plate rotation and out of plane buckling phenomenon was also observed in this study (see Figure 3). Similar observations are reported by other researchers (Okazaki et al., 2012; Hsiao et al., 2013; Salawdeh et al., 2017).

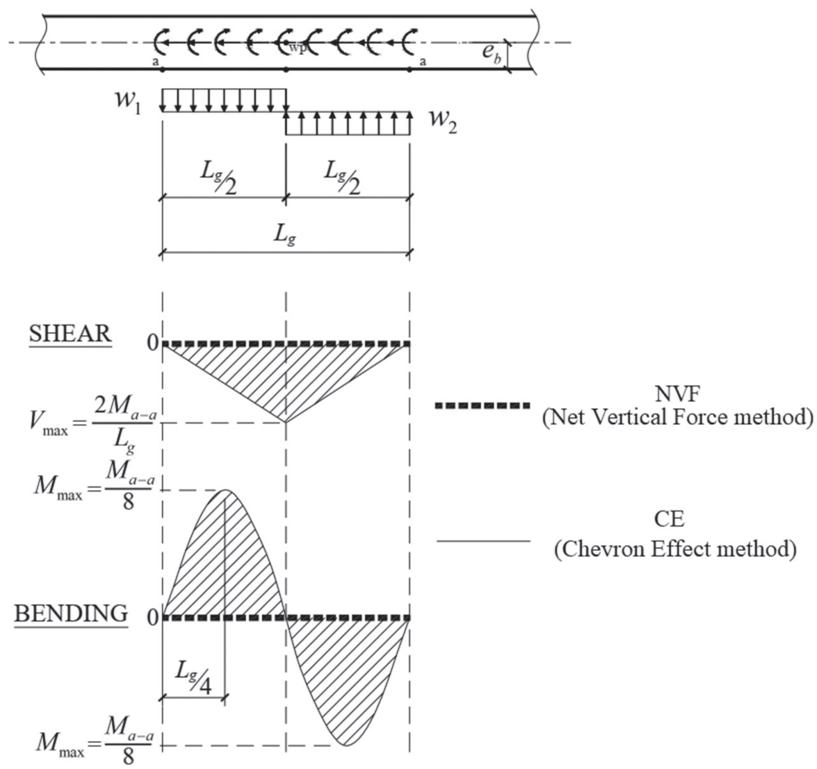
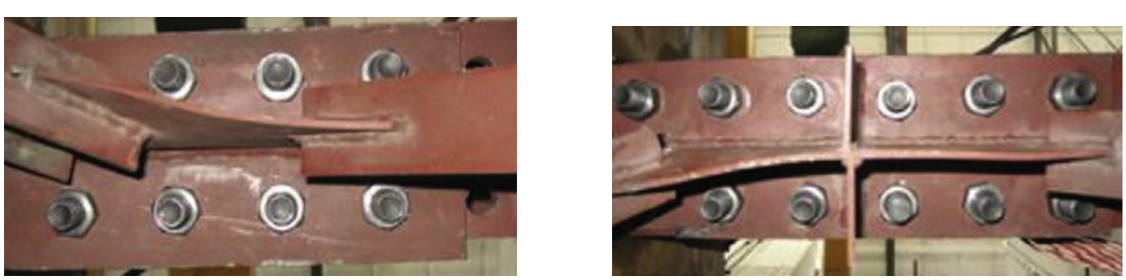


Fig. 1. The chevron effect—beam shear and moment (Fortney and Thornton, 2017).



(a) Without stiffener *(b) With stiffener*

Fig. 2. Deformation of gusset plates (Zhang et al., 2011).

TEST MATRIX

To evaluate the performance of the CE method, 20 gusset-beam assemblies are designed. Each assembly consists of a beam and either one or two gussets representing the two types of studied CBFs: a chevron (Vb) or X-bracing (Xb) configuration, respectively. The gusset is connected to the beam with two-sided fillet welds.

In order to provide enough freedom for the braces to buckle in real practice, a clearance must be provided between the end of the brace-to-gusset weld and the beam-to-gusset interface. Two values of clearances (2.0 in. and 8.0 in.) are considered in the models in order to evaluate the effect of the clearance distance on the stress distribution of the welded connection interface. The 2.0-in. clearance is assumed to represent a clearance that might be used when the braces frame to the bottom side of a beam. The 8.0-in. clearance is assumed to represent a clearance that might be used when a gusset frames to the top side of a beam (e.g., if one wanted the end of the brace to clear a floor slab). Models identified with an uppercase L, such as “M1 L”, are the models with the 8.0-in. clearance.

Three types of loads are considered in the design of the gussets and the beam-to-gusset welds: (1) equal tension and compression brace forces (Equation l); (2) mechanism load (Mec), which is the expected tensile strength of the tension brace and the expected buckling strength of the compression brace (in the case of X-bracing configuration, it will be the yield of two braces on a same line and buckling of the other two braces on the other line); and (3) the expected post-buckling strength of the compression brace (Pst) and the expected tensile strength of the tension brace. The brace forces are calculated using Equations 1–3 as required in AISC *Seismic Provisions* Section F2.6c (AISC, 2016).

$$P_{tension} = R_y F_y A_g \tag{1}$$

$$P_{compression} = 1.14 F_{cre} A_g \tag{2}$$

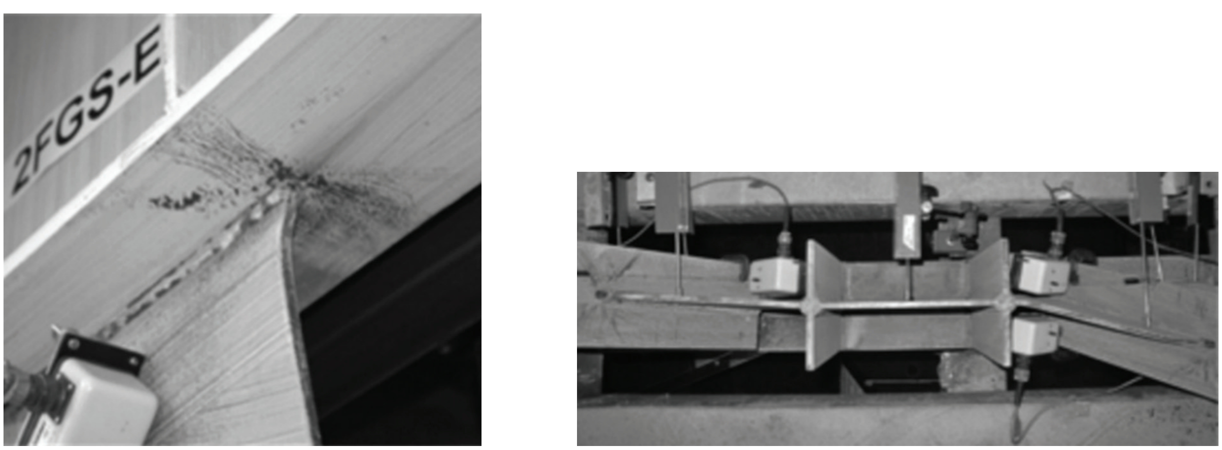
$$P_{post\text{-}buckling} = 0.3 P_{compression} \tag{3}$$

For sizing the beam, the maximum shear force and bending moment are obtained based on the CE method. The most economic (lightest) section is selected from Part 1 of the AISC *Steel Construction Manual* (AISC, 2017). Figure 4 shows the process of designing the beam using the CE method and compares it with the NVF method. The main difference between the two methods is that the NVF method is not dependent on the beam size and gusset geometry whereas the CE method is. Therefore, the CE method is an iterative process whereas the NVF method is not. The governing limit state for each beam used in each assembly is provided in Table 1.

Gusset thickness is determined based on eight limit states, including:

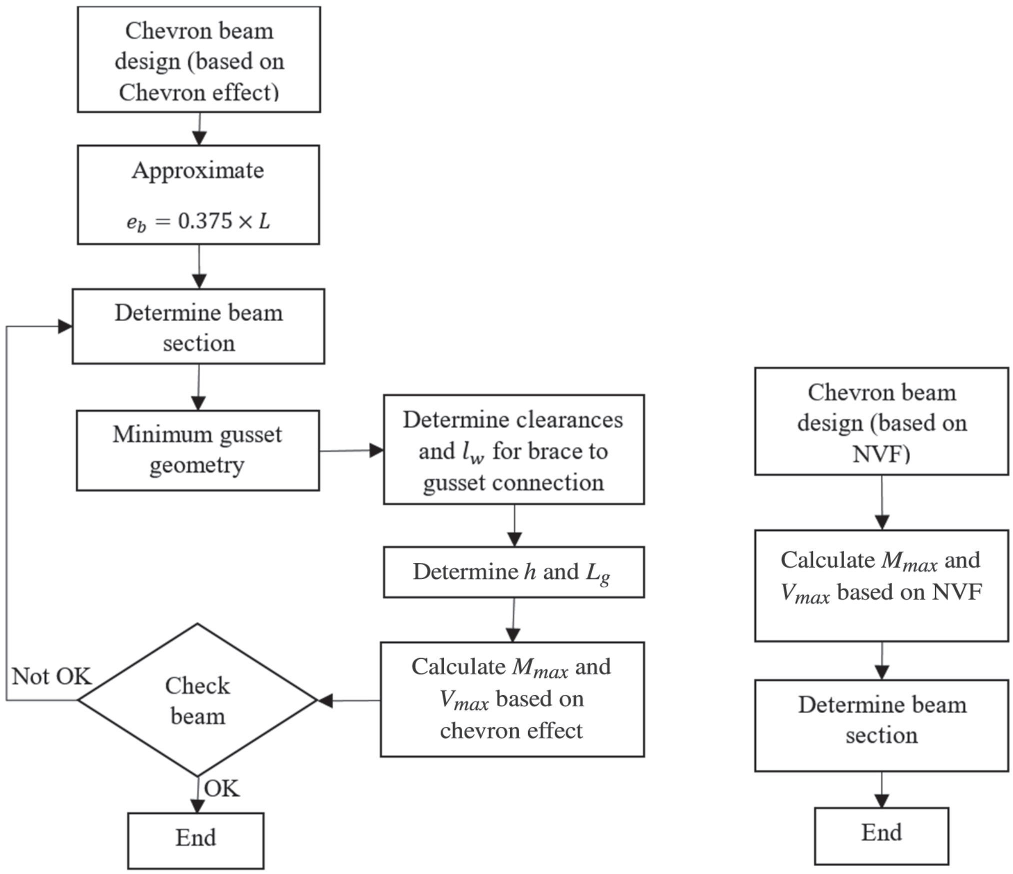
1. Tension along the brace line based on Whitmore section.
2. Compression along the brace line based on the Whitmore section.
3. Normal (axial) force to section a-a.
4. Shear force along section a-a.
5. Bending moment on section a-a.
6. Normal (axial) force to section b-b.
7. Shear force along section b-b
8. Bending moment on section b-b.

Figure 5 shows the design parameters.



(a) *(b)*

Fig. 3. Mid-span gusset rotation due to brace buckling (Lumpkin et al., 2012).



(a) Design based on CE *(b) Design based on NVF*

Fig. 4. Comparison of chevron beam design methods.

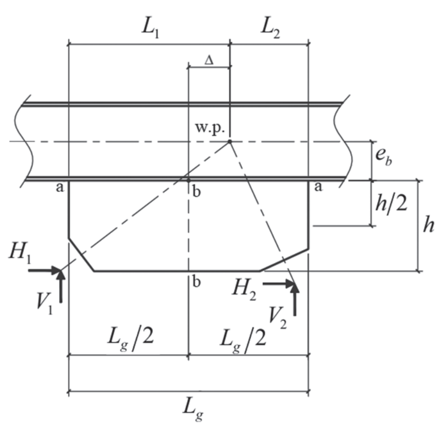


Fig. 5. Design parameters (Fortney and Thornton, 2015).

Table1. List of FEA Models with Design Details

No.	Model	Clearance (in.)	Load	Beam	Beam Limit State	t_{gusset} (in.)	Gusset Limit State	D (in.)	Geometry (in.)				Type
									H	a	b	Δ	
1	M1	2	Eql	W14×26	V	3/8	C	3/16	150	150	150	0	Vb
2	M1 L	8	Eql	W12×26	V	1/2	C	1/8	150	150	150	0	Vb
3	M2	2	Eql	W18×65	V	3/8	C	3/16	150	150	150	0	Xb
4	M2 L	8	Eql	W16×50	V	1/2	C	1/8	150	150	150	0	Xb
5	M3	2	Mec	W30×108	M	7/16	$A_{b\text{-}b}$	3/16	150	150	150	0	Vb
6	M3 L	8	Mec	W30×108	M	7/16	T	3/16	150	150	150	0	Vb
7	M3 Pst	2	Pst	W30×108	M	7/16	$A_{b\text{-}b}$	3/16	150	150	150	0	Vb
8	M3 L Pst	8	Pst	W30×108	M	7/16	T	3/16	150	150	150	0	Vb
9	M4	2	Mec	W24×84	V	7/16	T	3/16	150	150	150	0	Xb
10	M4 L	8	Mec	W21×62	V	7/16	T	3/16	150	150	150	0	Xb
11	M4 Pst	2	Pst	W24×84	V	7/16	T	3/16	150	150	150	0	Xb
12	M4 L Pst	8	Pst	W21×62	V	7/16	T	3/16	150	150	150	0	Xb
13	M5	2	Mec	W24×76	V	7/16	T	3/16	150	120	180	4	Xb
14	M5 L	8	Mec	W21×55	V	7/16	T	3/16	150	120	180	5	Xb
15	M5 Pst	2	Pst	W24×76	V	7/16	T	3/16	150	120	180	4	Xb
16	M5 L Pst	8	Pst	W21×55	V	7/16	T	3/16	150	120	180	5	Xb
17	M6	2	Mec	W24×84	V	7/16	T	1/4	180	144	156	1	Xb
18	M6 L	8	Mec	W21×62	V	1/2	T	1/4	180	144	156	2	Xb
19	M6 Pst	2	Pst	W24×84	V	7/16	T	1/4	180	144	156	1	Xb
20	M6 L Pst	8	Pst	W21×62	V	1/2	T	1/4	180	144	156	2	Xb

Eql: Equal tension and compression loading
Mec: Mechanism (buckling) loading
Pst: Post-buckling loading
$A_{b\text{-}b}$: Axial load on gusset section b-b
C: Compression
T: Tension
R: Rectangular
W: Whitmore
H: Story height
V: Shear
M: Moment (bending)
Vb: Chevron bracing configuration
Xb: X bracing configuration
a: Distance from work-point to left beam end
b: Distance from work-point to right beam end
Δ: Horizontal eccentricity
t_{gusset}: Gusset thickness

FINITE ELEMENT MODELS:

Abaqus 6.13 (Simulia, 2013) is employed to model and analyze the gusset-beam assemblies (listed in Table 1) and evaluate beam shear force and bending moment. An eight-node linear brick element is used for all parts in the models. An elastic perfectly plastic material model is used for all parts: ASTM A992 for the beams ($F_y = 50$ ksi), ASTM A529 Gr. 50 for the gussets ($F_y = 50$ ksi), and E70 ($F_u = 70$ ksi) for the welds. Two-sided fillet welds are used to connect the gusset(s) to the beam. Different mesh sizes were studied for each of the parts to find the appropriate mesh size. The mesh size study is summarized in Figure 6. The selected mesh sizes are 1 in. for the beam, 0.3 in. for the gusset, and 0.2 in. for the weld materials.

ANALYSIS RESULTS

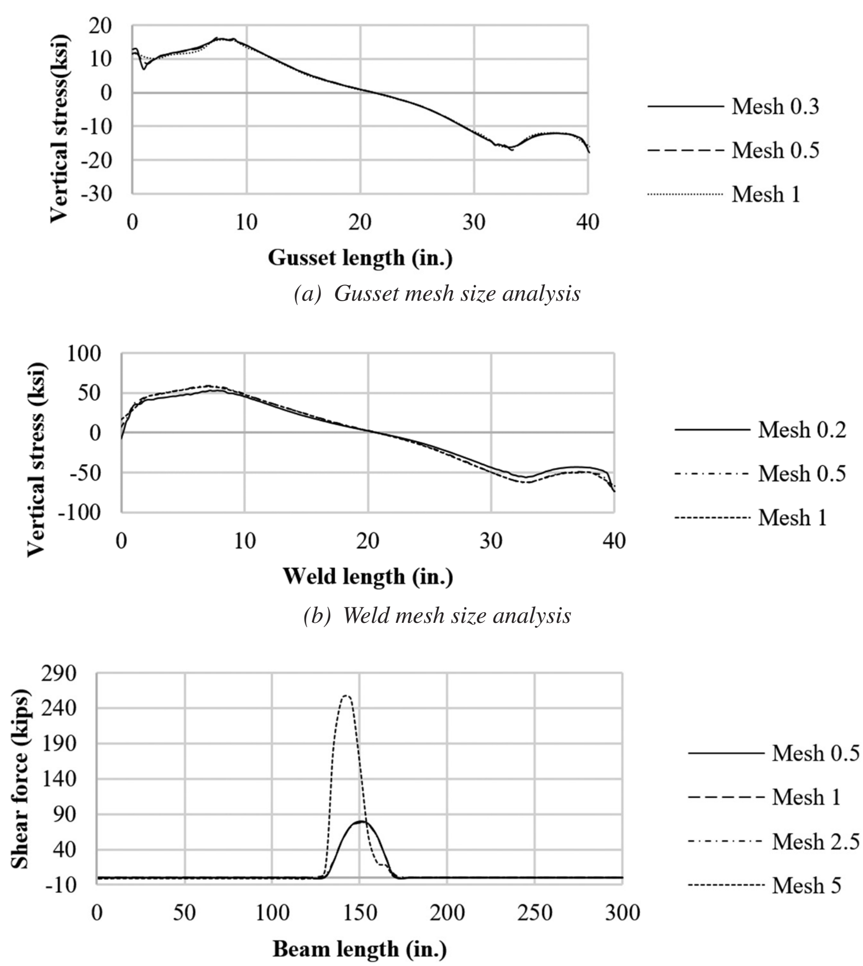
The shear force and bending moment diagrams of the beams were extracted from the finite element analysis (FEA) and plotted to compare the prediction of the CE and NVF methods to that of the FEA. Because the maximum beam shear force and bending moment are the important values in designing the beam section, a summary of the maximum values is reported in Table 2. To discuss and compare the results in more detail, all models are categorized into four groups based on the summation of the braces' vertical forces and the number of gusset plates used in each model:

1. Group1: $\Sigma V_T = 0$; gusset plate on one side of the beam
2. Group 2: $\Sigma V_T = 0$; gusset plate on both sides of beam
3. Group 3: $\Sigma V_T \neq 0$; gusset plate on one side of beam
4. Group 4: $\Sigma V_T \neq 0$; gusset plates on both sides of beam

The results of the analyses for each group are presented in the following sections.

Group 1: Summation of Vertical Forces = Zero; Single Gusset Plate

Because the summation of vertical forces is zero in this group, the NVF method does not predict any shear force and bending moment on the beam. However, due to the presence of the gusset, there is a shear force and bending moment on the beam due to the chevron effect, which is also observed in the FEA results. As can be seen in Table 3, the CE method predicts the maximum beam shear force and bending moment with values close to the FEA values. On average, the maximum beam shear force derived by CE method is 20% (1.22) larger than that of the FEA, and the maximum beam moment is approximately equal the FEA value (0.96). The distribution of beam shear force and bending moment for all methods is plotted in Figure 7 for model M1.



(a) Gusset mesh size analysis

(b) Weld mesh size analysis

(c) Beam mesh size analysis

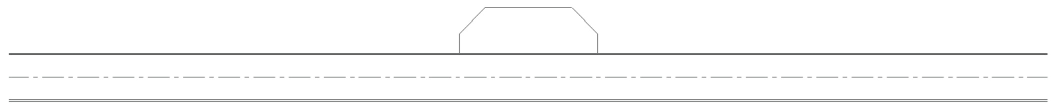
Fig. 6. Mesh size determination analysis for different parts.

Table 2. Summary of Results							
		CE		NVF		FEA	
No.	Model	V_{max} (kips)	M_{max} (kip-in.)	V_{max} (kips)	M_{max} (kip-in.)	V_{max} (kips)	M_{max} (kip-in.)
1	M1	93.4	244	0.00	0.00	79.4	256
2	M1 L	66.4	214	0.00	0.00	52.2	219
3	M2	226	651	0.00	0.00	183	228
4	M2 L	165	577	0.00	0.00	126	88.6
5	M3	169	8,130	56.7	8430	147	6,720
6	M3 L	144	7,920	56.7	8430	127	6,330
7	M3 Pst	132	13,600	99.1	14,800	123	11,800
8	M3 L Pst	112	13,300	99.1	14,800	113	11,200
9	M4	305	1,110	0.00	0.00	248	358
10	M4 L	227	964	0.00	0.00	181	107
11	M4 Pst	236	853	0.00	0.00	191	278
12	M4 L Pst	176	742	0.00	0.00	139	86.0
13	M5	313	5730	43.9	5,220	251	4,090
14	M5 L	224	5,520	43.9	5,220	196	4,090
15	M5 Pst	241	4,410	33.9	4,020	192	3,150
16	M5 L Pst	163	4,230	33.9	4,020	149	3,150
17	M6	309	1,870	6.81	980	259	813
18	M6 L	225	1,720	6.81	980	187	783
19	M6 Pst	238	1,440	5.24	754	200	624
20	M6 L Pst	173	1,330	5.24	754	143	603

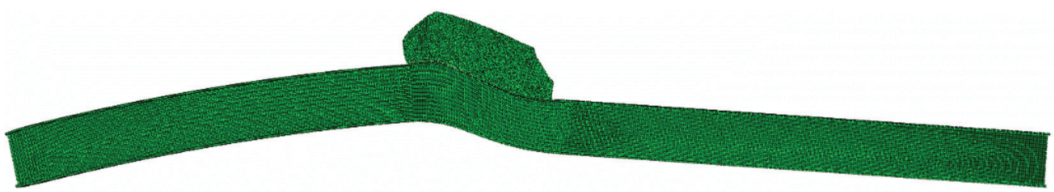
Table 3. Summation of Vertical Forces = Zero; Single Gusset Plate							
		CE/FEA		NVF/FEA		NVF/CE	
No.	Model	V_{max} (kips)	M_{max} (kip-in.)	V_{max} (kips)	M_{max} (kip-in.)	V_{max} (kips)	M_{max} (kip-in.)
1	M1	1.18	0.954	0.000	0.000	0.000	0.000
2	M1 L	1.27	0.977	0.000	0.000	0.000	0.000
Average		1.23	0.966	0.000	0.000	0.000	0.000

Group 2: Summation of Vertical Forces = Zero; Double Gusset Plates

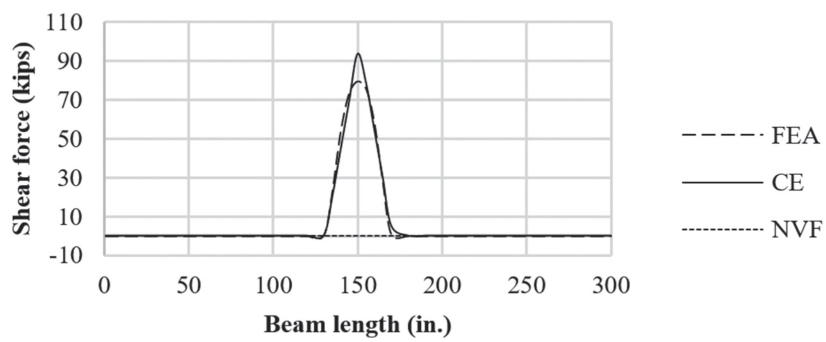
Similar to the first group's models, the summation of vertical forces is zero in this group, and thus the NVF does not predict any beam shear force and bending moment. As presented in Table 4, the CE method predicts a beam maximum shear force approximately 20% larger than that of the FEA. There is a big difference, however, between the maximum bending moment predicted by CE method versus the one in the FEA; the CE maximum bending moment is, on average, 5.5 times the FEA value. The difference is due to the presence of double gusset plates and the unavoidable suggestion of the FEA to stiffen the beam section with the gusset plates. It influences the beam flexural deformation, which leads to different beam bending moment values. The mentioned view point on the gusset plate(s) opposes the CE method that assumes (based on the discussed experimental observations) the gusset plate(s) does not stiffen up the beam section and thus does not have any impact on the magnitude of the beam bending moment. Regardless of the mentioned issue, the CE method can yet be a useful tool in the analysis and designing of such beams because the shear force is the governing limit state in selecting the beam section and the NVF method is not able to predict any shear force demand on the beam. The analysis results of model M2 is plotted in Figure 8 as an example.



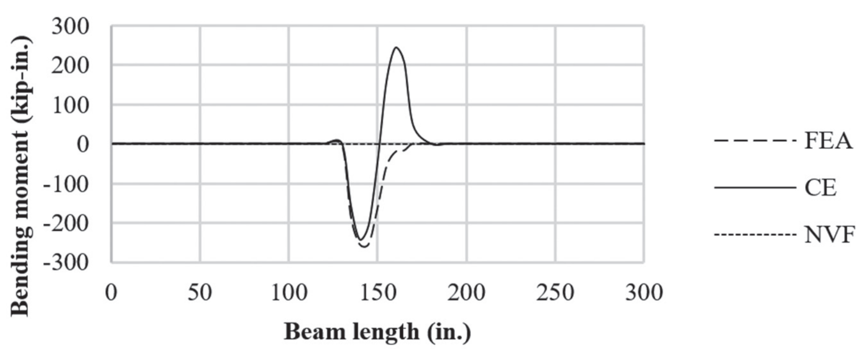
(a) Model geometry



(b) Deformed shape



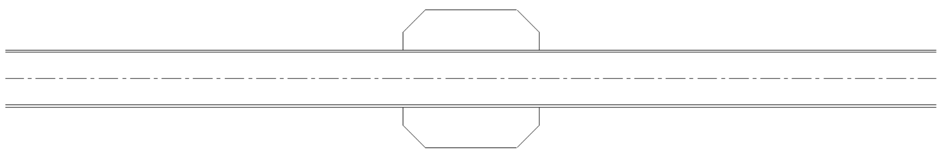
(c) Beam shear force diagram



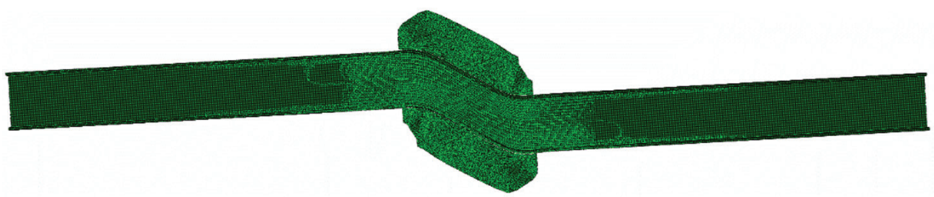
(d) Beam bending moment diagram

Fig. 7. Model M1 results.

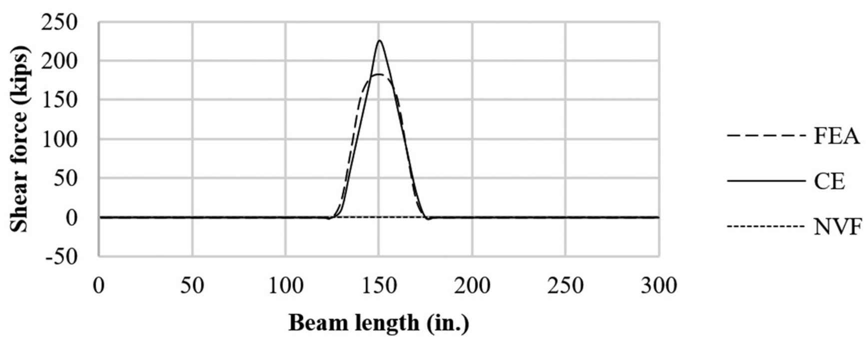
Table 4. Summation of Vertical Forces = Zero; Double Gusset Plates							
		CE/FEA		NVF/FEA		NVF/CE	
No.	Model	V_{max} (kips)	M_{max} (kip-in.)	V_{max} (kips)	M_{max} (kip-in.)	V_{max} (kips)	M_{max} (kip-in.)
3	M2	1.24	2.85	0.000	0.000	0.000	0.000
4	M2 L	1.31	6.51	0.000	0.000	0.000	0.000
9	M4	1.23	3.10	0.000	0.000	0.000	0.000
10	M4 L	1.26	9.02	0.000	0.000	0.000	0.000
11	M4 Pst	1.24	3.07	0.000	0.000	0.000	0.000
12	M4 L Pst	1.27	8.64	0.000	0.000	0.000	0.000
Average		1.26	5.53	0.000	0.000	0.000	0.000



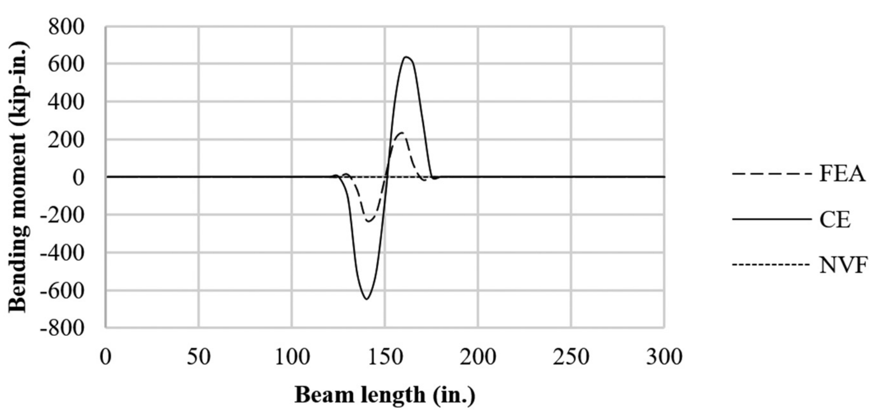
(a) Model geometry



(b) Deformed shape



(c) Beam shear force diagram



(d) Beam bending moment diagram

Fig. 8. Model M2 results.

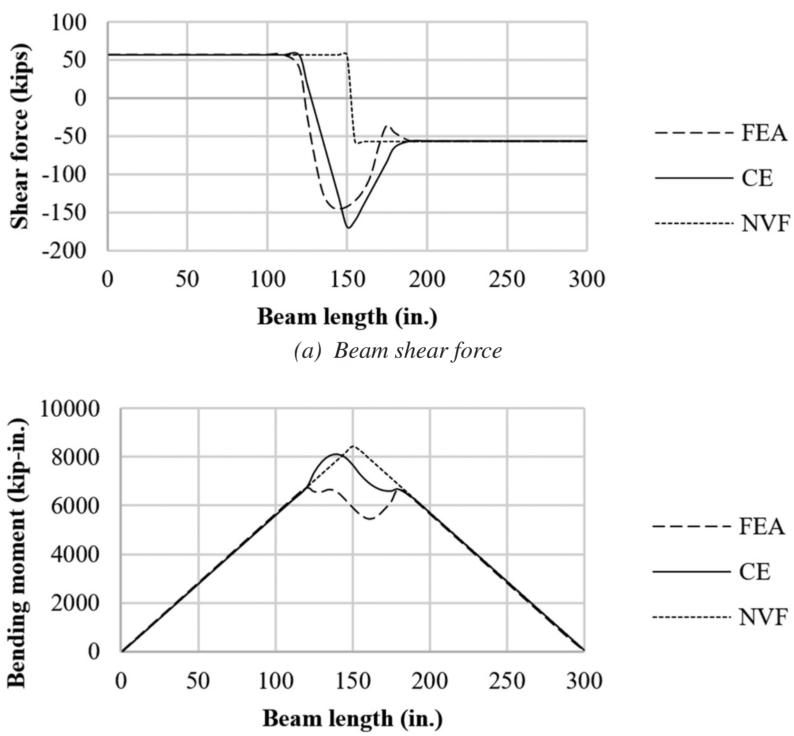
Table 5. Summation of Vertical Forces ≠ Zero; Single Gusset Plate							
		CE/FEA		NVF/FEA		NVF/CE	
No.	Model	V_{max} (kips)	M_{max} (kip-in.)	V_{max} (kips)	M_{max} (kip-in.)	V_{max} (kips)	M_{max} (kip-in.)
5	M3	1.16	1.21	0.387	1.26	0.335	1.04
6	M3 L	1.13	1.25	0.447	1.33	0.395	1.06
7	M3 Pst	1.07	1.16	0.803	1.25	0.752	1.08
8	M3 L Pst	0.992	1.19	0.875	1.32	0.881	1.11
Average		1.09	1.20	0.628	1.29	0.591	1.07

Group 3: Summation of Vertical Forces ≠ Zero; Single Gusset Plate

Based on the results in Table 5, the NVF method can predict the beam maximum bending moment in the models in which the summation of the vertical forces is not zero, but this method does not predict the beam maximum shear force. On the other hand, the CE method estimates a beam maximum shear force; approximately corresponding to the FEA (1.09) and bending moment 20% larger than FEA results. Thus, the CE method can be considered as a more versatile tool in analyzing chevron beams. The results of model M3 in this group is plotted in Figure 9.

Group 4: Summation of Vertical Forces ≠ Zero; Double Gusset Plates

Due to the presence of double gusset plates and the discussed issue regarding the stiffening influence of the gusset plates on the beam section, the CE method predicts the maximum beam moment on average 80% greater than that of the FEA. Because the beam maximum shear force is the determinative limit state in designing the X-bracing chevron beams (double gusset plate), and the CE method can estimate the maximum beam shear force with 20% difference in comparison to the FEA; the CE method can be used in analyzing such beams. Note that the NVF method does not



(a) Beam shear force

(b) Beam bending moment

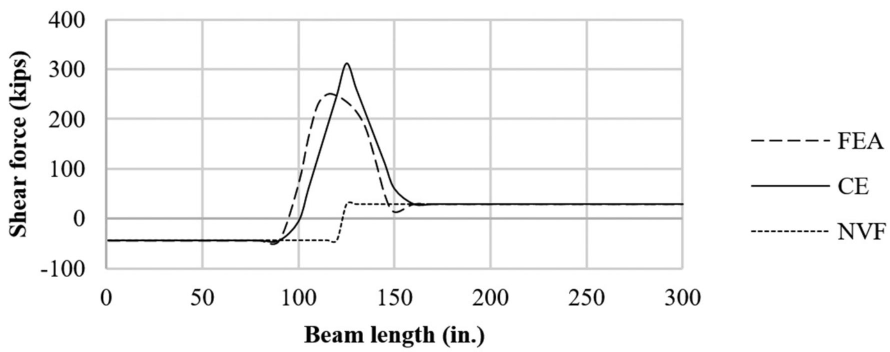
Fig. 9. Model M3 results.

Table 6. Summation of Vertical Forces ≠ Zero; Double Gusset							
		CE/FEA		NVF/FEA		NVF/CE	
No.	Model	V_{max} (kips)	M_{max} (kip-in.)	V_{max} (kips)	M_{max} (kip-in.)	V_{max} (kips)	M_{max} (kip-in.)
13	M5	1.25	1.40	0.175	1.28	0.141	0.912
14	M5 L	1.15	1.35	0.225	1.28	0.196	0.947
15	M5 Pst	1.25	1.40	0.176	1.28	0.141	0.912
16	M5 L Pst	1.09	1.34	0.227	1.28	0.208	0.950
17	M6	1.19	2.30	0.026	1.21	0.022	0.524
18	M6 L	1.21	2.20	0.037	1.25	0.030	0.569
19	M6 Pst	1.19	2.31	0.026	1.21	0.022	0.524
20	M6 L Pst	1.21	2.20	0.037	1.25	0.030	0.569
Average		1.19	1.81	0.116	1.25	0.099	0.738

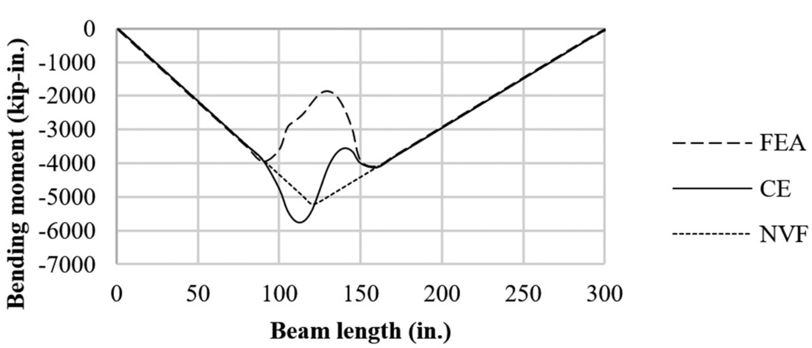
estimate the maximum beam shear force in such models (see the maximum shear ratios in Table 6). Model M5 results are plotted in Figure 10.

PERFORMANCE OF THE CE METHOD IN THE CASE OF FLAT BAR GUSSETS

As discussed earlier, the FEA suggests that the gusset plate(s) stiffen the beam influencing the flexural deformation of the beam and thus the beam bending moment values. In order to remove this influence, model M1 is reconsidered with one difference; the rectangular gusset plate is replaced with a couple of flat bar gusset plates. The new model's geometry and the analysis results are shown in Figure 11. According the beam shear force and bending moment results, the CE method correlates very well with the FEA when the stiffening influence of the gusset plate on the beam section is removed. As such, the assumption of uniformly distributed



(a) Beam shear force



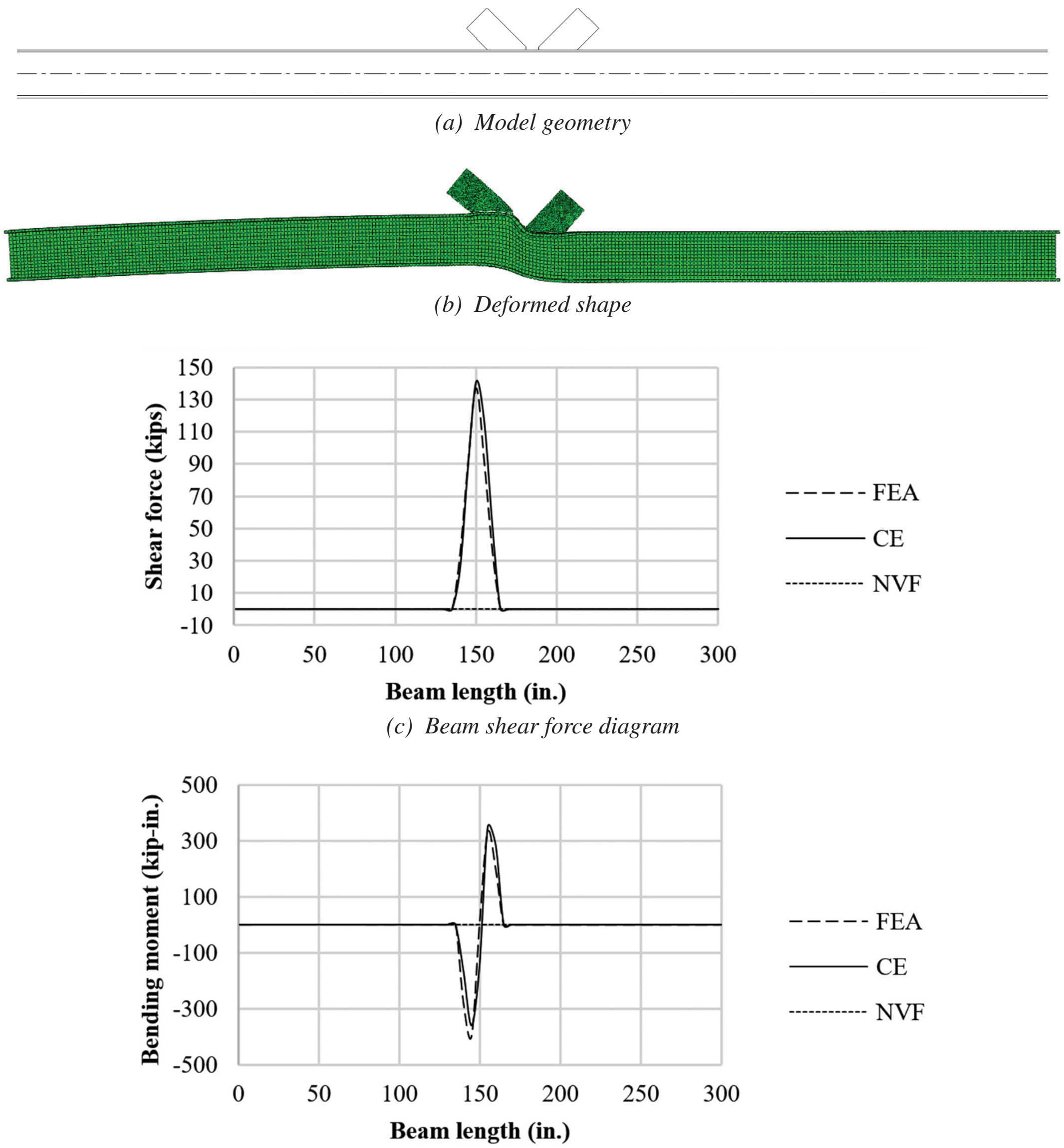
(b) Beam bending moment

Fig. 10. Model M5 results.

loads acting at the flat bar-beam interfaces is a valid assumption for flat bar gusset connections as well.

UNIFORM VERSUS NONUNIFORM STRESS DISTRIBUTION:

One of the common issues in brace connections is the uneven stress distribution across the gusset edge (Richard, 1986). In the case of a bracing connection welded to the flange of a member, which has significantly more rigidity than a connection to the web, the potential inability of the system to accommodate force distribution was observed (Hewitt and Thornton, 2004). The development of a peak stress induced at some point across the welded connection might cause the weld to fail at the point where the stress is concentrated, causing an unzipping of the weld and a progressive failure of the welded connection (Hewitt and Thornton, 2004). Such a phenomenon has been confirmed in experimental studies as well. See Figure 12 as an example (Lumpkin et al., 2012). To solve this issue, Richard (1986) considered the ratio of peak over average of the stress distribution at the edges of the studied gussets. In response to the Richard (1986) study, AISC introduced a weld ductility factor, equal to 1.4, that was intended to allow for adequate stress redistribution in the weld across the interface. That factor was subsequently reduced to 1.25 (Hewitt and Thornton, 2004).



(a) Model geometry

(b) Deformed shape

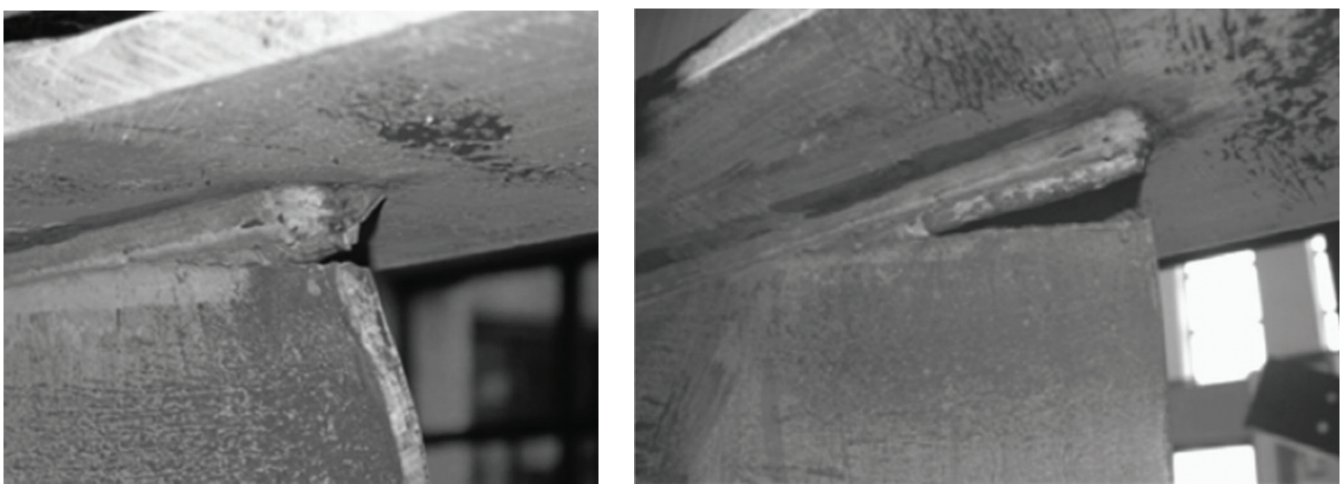
(c) Beam shear force diagram

(d) Beam bending moment diagram

Fig. 11. Model M1 with flat bar gusset plates results.

Stress concentration at the edges of the gusset(s) to the weld interface was observed in all finite element models studied in this research. Figure 13 shows a sample presenting the stress concentration at the left edge of the gusset-to-weld connection. In order to check the the stress distrubution along the gusset-to-weld connection region, a section cut is considered along the bottom of the gusset plate (above the fillet weld), and the vertical stress distribution at the section cut is studied. Finally, the peak over average ratio of the vertical stress distribution along the gusset to weld connection region is evaluated for all of the models.

Figure 14 shows the variation of peak to average ratio in 67 welded gusset connections. These values are also tabulated in Table 7. In Table 7, the ratios are given for the left and right half (compression and tension zones) of the gussets. The average of all ratios is 2.38, and the standard deviation is 0.63. These results show that the 1.4 ductility factor originally required for the corner gusset plates should be increased to a larger value (approximately 3) in the case of middle gusset plates used in chevron configuration. Another possible approach to solve this problem is to design the welds based on the shear strength of the gusset plate. Further studies with regard to the determination of the ductility factor for the case of chevron gusset plate connections are recommended by the authors.



(a) During the test (b) End of the test

Fig. 12. Weld tearing at corner gusset plate (Lumpkin et al., 2012).

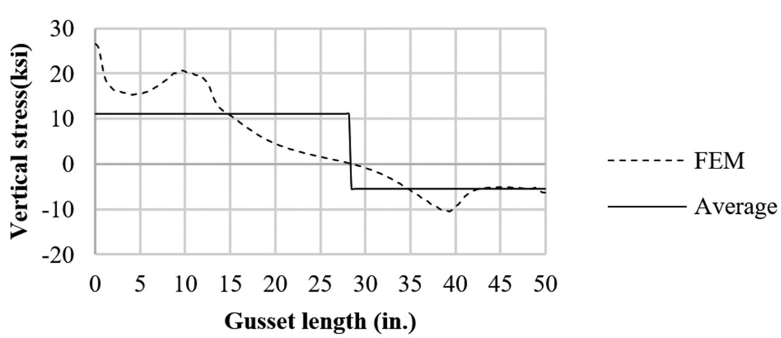


Fig. 13. Vertical stress distribution at gusset to weld interface, model M6.

Table 7. Vertical Stress Distribution Ratios					
		Top Gusset		**Bottom Gusset**	
		Peak/Average		**Peak/Average**	
No.	**Model**	Left Half	Right Half	Left Half	Right Half
1	M1	1.74	1.89	—	—
2	M1 L	3.87	2.13	—	—
3	M2	1.96	2.01	1.95	1.93
4	M2 L	2.18	2.16	2.33	2.23
5	M3	2.10	2.61	—	—
6	M3 L	2.54	1.88	—	—
7	M3 Pst	1.99	1.79	—	—
8	M3 L Pst	2.36	1.41	—	—
9	M4	2.24	2.03	2.13	2.12
10	M4 L	2.03	2.27	2.19	2.44
11	M4 Pst	2.43	3.76	2.32	3.90
12	M4 L Pst	2.26	2.70	2.35	2.84
13	M5	2.18	2.80	2.32	2.73
14	M5 L	2.70	1.90	1.90	2.99
15	M5 Pst	2.68	2.26	2.34	3.36
16	M5 L Pst	3.22	1.65	1.88	2.49
17	M6	2.38	1.94	2.14	2.53
18	M6 L	2.58	1.34	1.71	3.42
19	M6 Pst	2.44	1.93	2.23	5.07
20	M6 L Pst	3.34	—	2.55	1.91

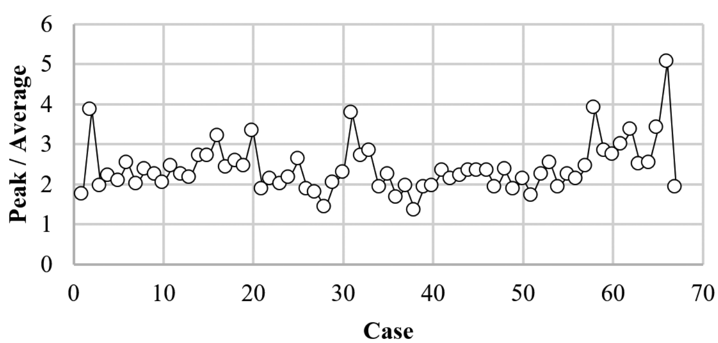


Fig. 14. Variation of peak over average vertical stress ratio in all assemblies.

CONCLUSION

The chevron effect is a phenomenon that occurs in the frame beams of CBFs with chevron V- and inverted V-bracing configurations due to the presence of gusset plate(s). This phenomenon was introduced in earlier literature, and a mathematical model was developed to generate an admissible force distribution in the chevron gusset connection to obtain beam shear force and bending moment with the inclusion of the chevron effect. The work reported in this paper considered 20 beam-gusset assemblies connected by fillet welds. To check the performance of the mathematical model (CE method), the authors used finite element analysis (FEA) to investigate the beam shear force and bending moment of all models.

The FEA results clearly revealed the presence of the chevron effect in the beam shear force and bending moment diagrams. The net vertical force (NVF) method, which has been commonly used in analyzing chevron beams, was not able to capture the chevron effect and could not estimate the maximum beam shear force accurately in all models, while the CE method could capture the chevron effect. On average, the maximum beam shear force estimated by CE method is approximately 20% higher of that in the FEA results. Thus, the CE method can always estimate the beam shear force with an acceptable conservatism.

Due to the out-of-plane behavior of the gusset plate, the CE method does not consider the gusset plate(s) as a stiffening element for the beam section. However, the FEA stiffens the beam section by the gusset plate(s), which influences the beam flexural deformation and thus the value of bending moment. The mentioned issue caused the difference between the CE method and the finite element model bending moment results, specifically for models with double gusset plates. On average, the ratio between the CE maximum bending moment and that of the FEA analysis is 2.4. However, because the maximum beam shear force is the governing limit state in cases with X-bracing configurations (double gusset plates), the CE method reasonably predicts the maximum beam shear force in such models and can be a useful tool in designing all types of chevron brace frame beams. If one can reasonably ensure that the gusset will experience no out-of-plane deformations, a rational analysis of the beam that includes the gusset stiffening effects may be reasonable.

The stress concentrations at the ends of the gusset-to-beam welds, caused local failure at the weld terminations. To avoid this local failure, the ratio of peak over average of the vertical stress distribution along the gusset to weld connection interface is calculated and studied. The results showed the ductility factor calculated for corner gussets should be reevaluated in the case of middle gusset plates. Until a more detailed study is performed to evaluate the interface stresses, the authors recommend providing an interface weld that develops the shear strength of the gusset plate.

IN MEMORY OF PATRICK FORTNEY

The impact of losing Dr. Patrick Fortney during the course of this research is beyond words. Kind, knowledgeable, dedicated, and thoughtful are the words that are being echoed by his friends and colleagues when remembering and describing him. His knowledge in structural engineering—next to his experience in steel structure design, construction, and fabrication—made him the pioneering engineer to identify the chevron effect in the behavior of chevron beams in concentrically braced frames. Although his loss has left a great void in the steel industry, as well as his friends' and colleagues' hearts, his scientific findings and unique professional contributions will be remembered vividly forever.

SYMBOLS

A_g	Gross cross-section area of beam, in.^{2}
$A_{b\text{-}b}$	Normal (axial) force to section b-b limit state
C	Compression along the brace line limit state
Eql	Equal tension and compression brace loads on the assembly
F_{cr}	Critical stress, ksi
F_u	Specified minimum tensile strength, ksi
F_y	Specified minimum yield stress, ksi
H_1	Horizontal component of force in brace 1, kips
H_2	Horizontal component of force in brace 2, kips
L_1	Horizontal distance from the left edge of the gusset to the work point, in.
L_2	Horizontal distance from the right edge of the gusset to the work point, in.
L_g	Contact length of the gusset to beam interface, in.
M	Beam bending moment limit state
$M_{a\text{-}a}$	Moment acting at the gusset to beam interface, kip-in.
M_{max}	Beam maximum bending moment, kip-in.
Mec	Mechanism (buckling) loading on the assembly
$P_{compression}$	Brace compression strength, kips
$P_{post\text{-}buckling}$	Post-buckling strength, kips

$P_{tension}$	Brace tensile strength, kips
Pst	Post-buckling loading on the assembly
T	Tension along the brace line limit state
R_y	Ratio of the expected yield stress to the specified minimum yield stress
V	Beam shear force limit state
V_1	Vertical component of force in brace 1, kips
V_2	Vertical component of force in brace 2, kips
V_{max}	Maximum beam shear force, kips
Vb	V-bracing configuration
Xb	X-bracing configuration
a	Distance from left beam support to location of work point, in.
b	Distance from work point to right beam support, in.
e_b	Perpendicular distance from the gusset interface to the gravity axis of the frame beam, in.
h	Vertical distance of the gusset, in.
t_{gusset}	Gusset plate thickness
w_1	Net uniformly distributed transverse load on left half of gusset, kip/in.
w_2	Net uniformly distributed transverse load on right half of gusset, kip/in.
w.p.	Brace work point
Δ	Horizontal misalignment between the work point and the centroid of the gusset-to-beam interface, in.
ΣV_T	Summation of vertical components of brace forces, kips

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DISCUSSION

Investigation on the Performance of a Mathematical Model to Analyze Concentrically Braced Frame Beams with V-Type Bracing Configurations

Paper by ALIREZA ASGARI HADAD and PATRICK J. FORTNEY
(2020, 2nd Quarter)

Discussion by CHARLES W. ROEDER, DAWN E. LEHMAN, QIYANG TAN, JEFFREY W. BERMAN, and ANDREW D. SEN

The paper "Investigation on the Performance of a Mathematical Model to Analyze Concentrically Braced Frame Beams with V-Type Bracing Configurations" (Hadad and Fortney, 2020) addresses a difficult problem that is not well understood by engineers. While that paper does not specifically limit its application to seismic design, the work was based upon the AISC *Seismic Provisions* and chevron-braced frame seismic behavior experiments. The authors of this discussion, hereafter referred to as the responders for clarity, have also studied chevron-configured concentrically braced frames. That research forms the basis of this discussion. In particular, this discussion aims to clarify the AISC *Seismic Provisions* and prior research results, which the responders believe is misinterpreted in the paper in question. Further, in the series of continuum finite element analyses on a beam with a plate discussed in the paper, it is of note that the model's boundary conditions, loading regime, and results are simplifications of those for beams in chevron-configured or multistory X-braced frames under seismic loading. This discussion addresses applicability of the paper to seismic design and does not apply to any other application.

There are several aspects of the original paper that need to be clarified. First, the findings of some of the cited research is misstated. For example, the paper states that the yielding-beam mechanism is inferior to the "nominally elastic" chevron-beam concept, but this is not correct. Research by the responders (Sen et al., 2016) shows that chevron-braced frames with yielding beams provide better distribution of inelastic deformation over the building height than those with "nominally elastic" beams, and they develop larger inelastic deformations prior to brace fracture. The expected brace forces for seismic design (i.e., Equations 1, 2, and 3 in the paper in question) are used as the basis of the analyses as specified in the AISC *Seismic Provisions*, but it is important to recognize that these are not the forces necessary to resist the design seismic lateral force for the system; instead, they are an idealized set of demands used in the capacity-based design process.

In seismic design of braced frames, the braces are initially designed to resist reduced forces [forces corresponding to the design basis earthquake reduced by the seismic reduction factor, R, as determined by ASCE/SEI 7, *Minimum Design Loads and Other Associated Criteria for Buildings and Other Structures* (ASCE, 2016)]. These reduced seismic design forces are expected to correspond to demands resulting from small, frequent earthquakes, and the structure is expected to remain elastic for that seismic hazard level. For special concentrically braced frames (SCBFs), collapse resistance of the structure is achieved by capacity-based design and detailing for ductility to ensure the system can sustain the cyclic inelastic deformation demands during large earthquakes. In such events, the braces are expected to buckle in compression, yield in tension, and sustain deterioration of their post-buckling compressive resistance at large inelastic deformations. Adjacent members, including the beams, columns, and gusset plates, are designed using capacity-based methods to ensure the braces develop their resistance and inelastic deformation capacity. The capacity-based design concept is not intended to guarantee that members or connections remain elastic during an earthquake; they are simply intended to ensure that the ductile element (the brace in this case) can develop its full deformation capacity by designing the adjacent members to sustain their maximum demands. A primary issue with the

Charles W. Roeder, Professor, Department of Civil and Environmental Engineering, University of Washington, Seattle, Wash. Email: croeder@uw.edu (corresponding)

Dawn E. Lehman, Professor, Department of Civil and Environmental Engineering, University of Washington, Seattle, Wash. Email: delehman@uw.edu

Qiyang Tan, Visiting Graduate Student, Department of Civil and Environmental Engineering, University of Washington, and PhD Candidate, Harbin Institute of Technology, Harbin, China. Email: qiyang_tan@163.com

Jeffrey W. Berman, Professor, Department of Civil and Environmental Engineering, University of Washington, Seattle, Wash. Email: jwberman@uw.edu

Andrew D. Sen, Assistant Professor, Department of Civil, Construction and Environmental Engineering, Marquette University, Milwaukee, Wis. Email: adsen@uw.edu

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stress evaluations proposed in the paper is that they will not necessarily achieve this goal.

The main objective of capacity-based design in SCBFs is to develop the inelastic deformation of the brace rather than to design for a set of fictitious forces. The gusset plates are designed for the expected brace capacities because they are directly connected to the brace. However, when braces are oriented for out-of-plane buckling, the gusset plates deform inelastically due to brace buckling and, as a consequence, sustain significant inelastic deformation demands, as shown in Figure 1. The actual local stresses developed in gusset plates after brace buckling are distributed unevenly and are much larger than those used in gusset-plate design calculations; as the authors recognize, this has been well established even in the absence of brace buckling (Richard, 1986). However, gusset plates that are very conservatively designed will still develop significant yielding because of this out-of-plane brace deformation demand (Lehman et al., 2008). Gusset plates sustain these demands because of the inherent ductility of steel, but welds joining gusset plates to beams and columns are vulnerable to tearing and fracture prior to brace fracture. Therefore, such welds should be designed to develop the plastic capacity of the gusset plate, not the expected brace forces, because inelastic action in the gusset plate is required to accommodate the deformation demands on the system. The User Note in AISC *Seismic Provisions* Section F2.6c.4 (AISC, 2016) is intended to address this goal. While the gusset plates should be designed to develop the expected brace forces, it is unwise to overdesign the gusset plate because that will require larger and more expensive welds on the gusset plate-to-beam or plate-to-column interface. The paper suggests that the welds joining a midspan gusset to the chevron beam should be designed for the in-plane shear, bending moment, and axial force from the gusset plate, but research funded by AISC has shown that this can lead to premature weld fracture, as illustrated in Figure 2 (Swatosh, 2015). Weld fracture should be prevented in seismic design because it prevents the braces and the SCBF from developing their full inelastic capacity. Moreover, weld fracture in some gusset plate connection configurations can also compromise the vertical load-resisting system, as shown in Figure 2.

A second issue with the research presented in the paper is the numerical simulation of the chevron beam. A finite element analysis of the beam and midspan plate was performed with the goals of improving the understanding of the behavior of beams in chevron-braced frames and supporting their proposed stress distribution at the gusset plate-to-chevron beam weld. There are several issues with the approach. First, the boundary conditions of the beam and other aspects of the modeling approach are not described, and these will influence the demands at that weld. It appears that the beam was simply supported, which is an idealized boundary condition and not representative of an actual SCBF. Second, a set of idealized point loads were applied to the gusset plate. However, the demand resulting from cyclic action on an SCBF does not result in a single set of forces, and as such, this approach is not sufficient to understand this complex stress state. Third, as a result of these two issues, the local demands are not properly simulated, but this is not recognized in the paper. Nevertheless, a detailed discussion of the local shear forces and bending moments of the beam in the midspan gusset-plate connection are provided, and it is asserted that these local effects

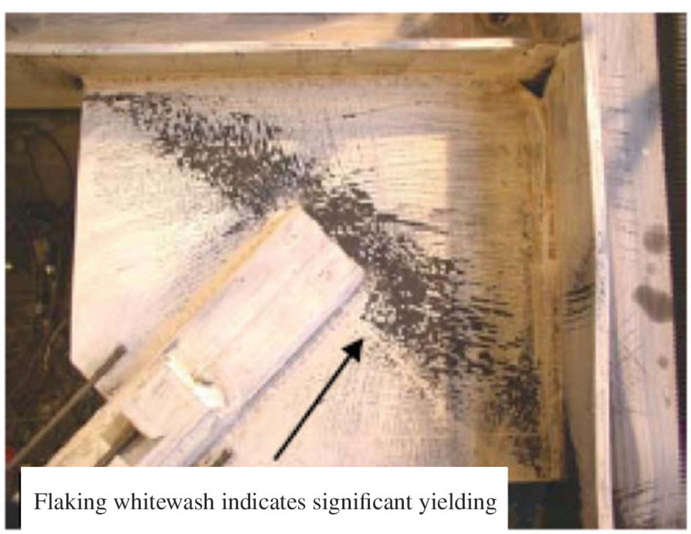
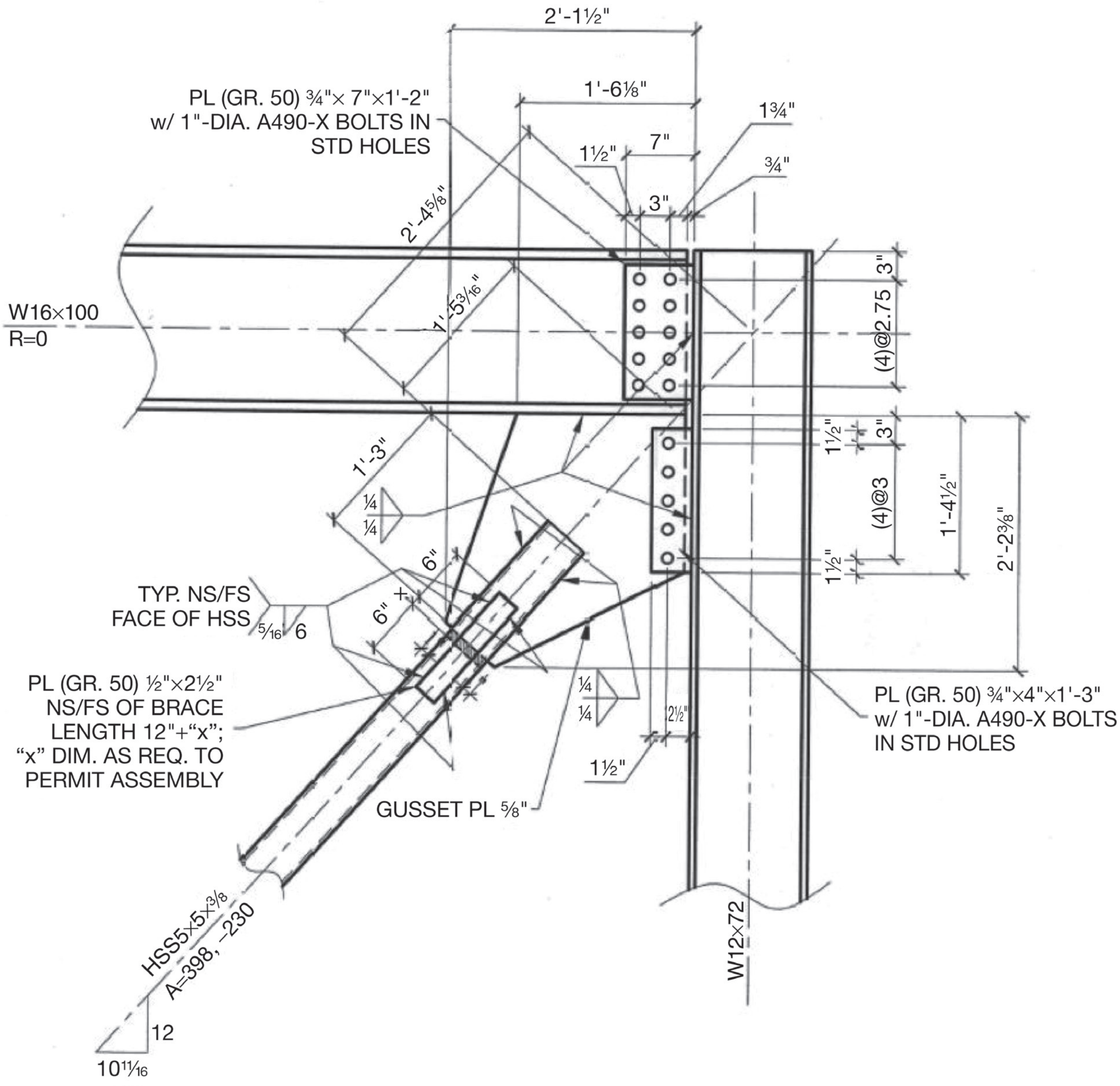
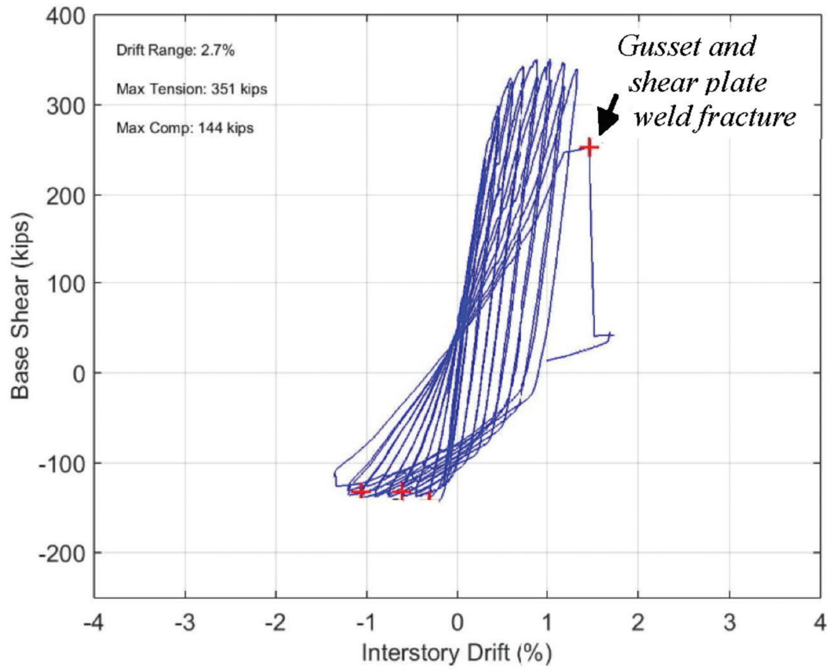

Flaking whitewash indicates significant yielding

Fig. 1. Inelastic deformation of a gusset plate under seismic deformation.



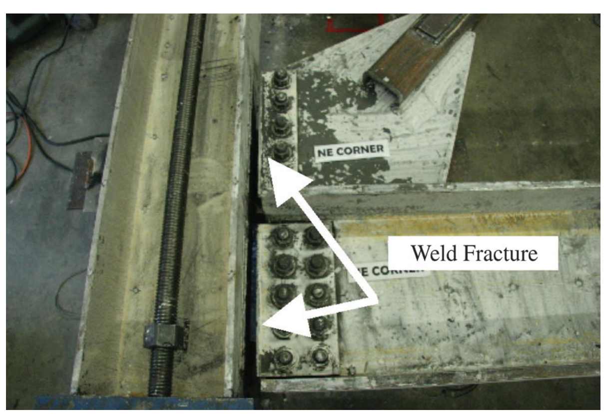
(a) Gusset-plate connection based on expected brace forces

Fig. 2(a). Fracture of gusset-plate welds designed by stress.



(b) Inelastic deformation of braced frames

(c) Buckled brace just prior to connection weld fracture



(d) Fractured weld

Fig. 2(b-d). Fracture of gusset-plate welds designed by stress.

control the design of the beam. This conclusion is incorrect because the simulation neglects the axial load, shear force, and bending moment demands in the beam outside the gusset area. These forces and moments are larger than the forces and moments in the gusset area as demonstrated in Figure 3, and they form the primary design considerations for the chevron beam.

Considering these three issues, there are several concerns with the paper's approach:

1. Using St. Venant's principle indicates that the local stresses in the beam in the vicinity of the gusset plate are not reliable when using this calculation method. The principle clearly states that an analysis will produce reasonable results at considerable distance from the load if the load is in equilibrium with the true condition but distributed differently; however, calculated stresses near or adjacent to the applied loads will be quite different under these conditions. Rather than use these reliable stresses away from the gusset plate, the analysis proposed in the paper focuses on local stresses in a disturbed region, and first principles indicates that this is not correct. In comparison, consider the analysis shown in Figure 4, which are the results from a nonlinear analysis of a full chevron-braced frame. The figure shows that the stresses and deformations in the beam when the full system is analyzed are very different from those described in the paper. The beams were designed for the forces of Equations 1, 2, and 3, but there is still significant yielding of the beam. Further, stresses are significantly higher in regions that are *not* adjacent to the midspan gusset plate; these higher stresses must be considered in design. They are neglected in the paper, however, and this is a primary concern to the responders.
2. The paper focuses on the stress-distribution transfer between the midspan gusset plate and the chevron beam; to achieve this, an idealized distribution of stresses is proposed, and these stresses are transferred to the beam as an external load. This approach does not represent the stress transfer because the gusset plate is attached to the beam by continuous welds (or bolts). Therefore, compatibility of the strain between the beam and gusset plate must be maintained at this interface, and these are truly internal stresses. The stresses and strains in the beam and gusset plate must be the same at this location, and the gusset plate and the beam join to resist the forces. The proposed idealization suggests that only the beam resists these forces; this is incorrect. The local effects predicted in the paper's proposed method do not represent the significant overall moment, shear, and axial load demands, and they do not consider that the chevron beam is strengthened significantly by the gusset plate in this region.
3. The axial load in the beam is neglected. A braced frame is often idealized as a truss. This idealization neglects the bending moment demands on the beams and column, but research by the responders indicates that it does

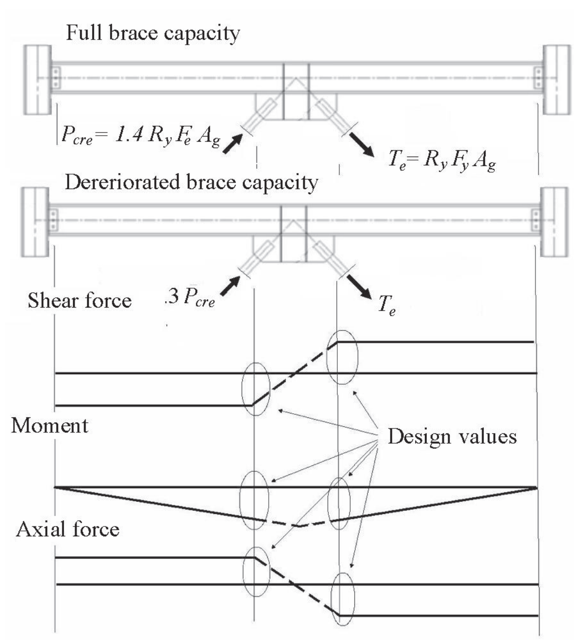
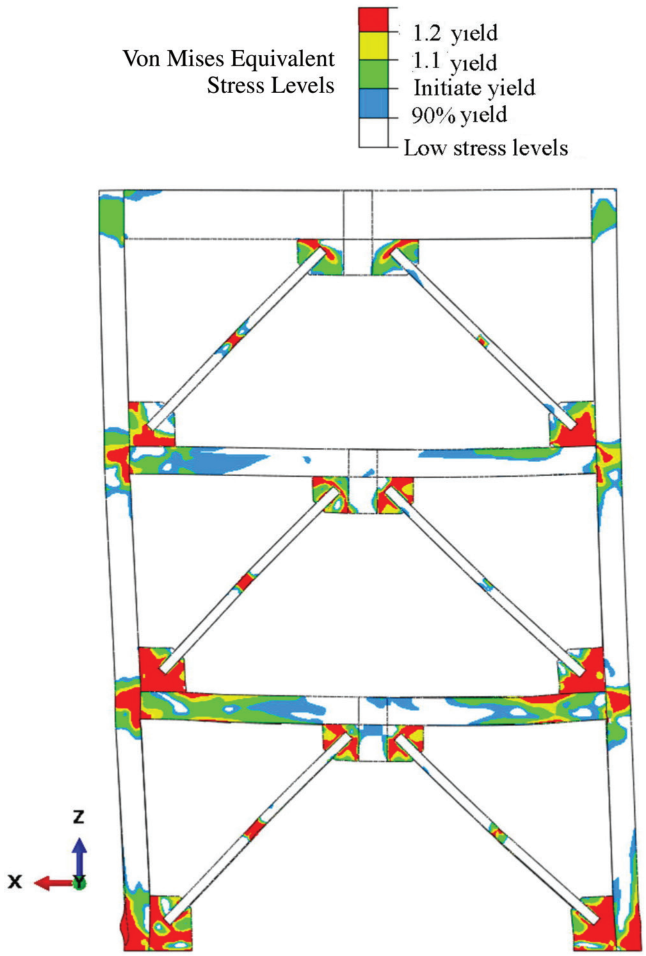
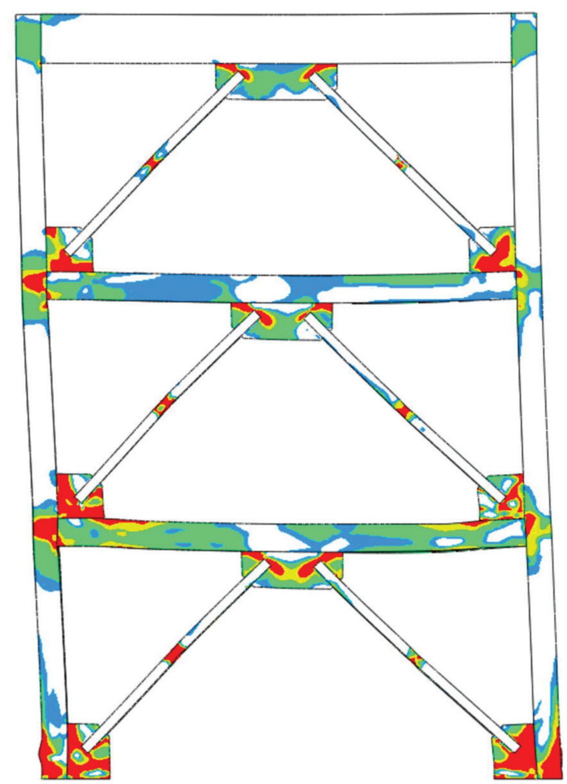


Fig. 3. Axial-load, shear-force, and bending-moment diagrams for idealized boundary condition for capacity-based design of chevron beams.



(a) Frame with chevron beam designed to current AISC Seismic Provisions *with* t_w/t_p *of 0.8 and with stiffeners*

Fig. 4(a). Nonlinear analyses of chevron-braced frames.



(b) Frame with chevron beam designed to current AISC Seismic Provisions *with* t_w/t_p *of 0.8 and with no stiffeners*

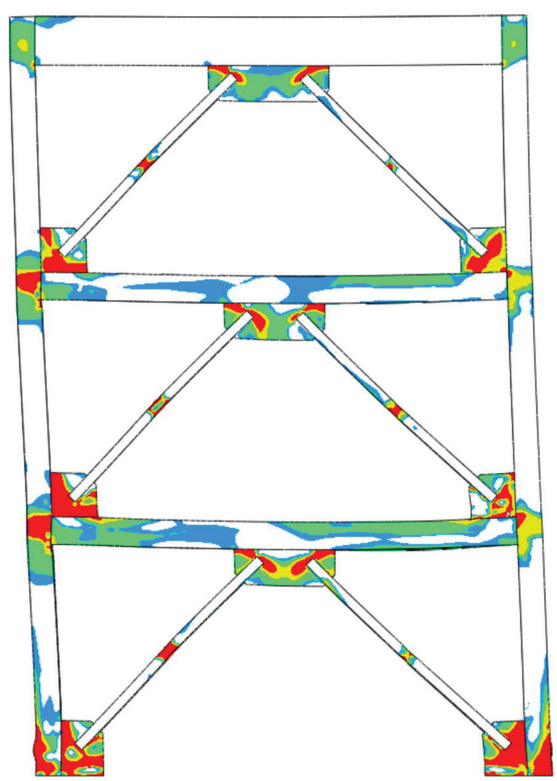
Fig. 4(b). Nonlinear analyses of chevron-braced frames.

provide a reliable estimate of the axial demands. As such, all primary members in SCBFs must be designed considering the axial forces. Neglecting these forces is not an acceptable approach for chevron-beam design. The chevron beam must be designed to develop the combined axial load and bending moment, not local stresses at the gusset plate.

4. In chevron-braced frames, the brace forces distributed over the region of the gusset plate and the extreme local stresses shown in the paper cannot occur. The stresses at the interface also include stresses induced by out-of-plane brace buckling deformations, which are not included in the proposed model. Further, the ends of the chevron beam may not be simply supported. There are often corner gusset plates for the bracing of the story above, and research has shown that these corner gusset plates will provide significant end restraint to the beam and develop significant end moments. This is clearly seen in the nonlinear analysis of Figure 4, but it is not considered in the paper. If there is no corner gusset plate, and the beam-to-column connection is a shear plate or similar connection, the end restraint and resulting moments are very small, as depicted in Figure 3. Figure 3 and the analytical results shown in Figure 4 indicate significant stress due to shear, bending, and axial force outside the gusset regions. Within the midspan gusset regions, the beams are more lightly stressed because the gusset plates stiffen and strengthen the beams in this location.

A BETTER WAY

Local beam forces result from post-buckling brace forces and the boundary conditions of the beam, and these local effects simply change the direction of shear, axial force, and bending moment vectors over the length of the midspan gusset plate. Figure 3 shows that the maximum moment, shear, and axial load are just outside the gusset plate, shown as dashed lines in Figure 3. Analysis of the frame shows the von Mises stresses resulting from combined moment, shear, and axial load are smaller in the region of the midspan gusset and largest just outside of the corner and midspan gusset plates (see Figure 4). Although these forces are affected by the local stresses argued in the paper, the only effect of these local stresses is to change direction and magnitude of these forces over the short length of the gusset plate, which is not the critical section of the beam. The actual shear and axial force must be largest in the beam section outside the



(c) Frame with chevron beam designed to current AISC Seismic Provisions *with* t_w/t_p *of 1.0 and with no stiffeners*

Fig. 4(c). Nonlinear analyses of chevron-braced frames.

gusset plates, shown by the solid lines. Note that Figure 3 assumes a pinned condition at the beam-to-column connection; if the stiffness of the corner gusset is included, then the demands and resulting stresses are also high just outside of the corner gusset plate, as shown in Figure 4. The moment will be slightly larger over the length of the midspan gusset, but the gusset plate adds considerable cross-sectional area and depth to the beam; thus, this larger moment is resisted by the combined capacity of the wide-flange section and gusset plate. The critical section is outside of the gusset plate—and, therefore, the bending moment at the edge of the gusset plate—needs to be checked in the beam design. It is important to note that the beam must be designed for the maximum combined axial, bending, and shear demands, which occur at the circled areas of the figure. This design requirement is not addressed in the paper.

While the responders disagree with results of the paper's stress-driven approach, they also recognize that additional checks are needed to evaluate the midspan gusset plate-to-beam connection in chevron-braced frames. The paper contends that seismic design requires a detailed stress calculation based on forces used as aids in capacity-based design. However, these calculated stresses are not accurate, and they will not capture the local stress demands that actually occur. As previously noted, the gusset-plate connection develops larger stresses than computed if the gusset plate is solely designed to develop the expected brace forces because out-of-plane brace buckling induces inelastic deformation demands on the gusset plate.

To accommodate these demands, gusset-plate welds should be sized to the plastic capacity of the gusset plate as stated in the User Note in AISC *Seismic Provisions* Section F2.6c.4. If the welds are designed to meet this user note, capacity-based design would logically require that the beam must also be capable of developing this resistance. For corner gusset plates where the beam is attached to the column as part of the beam-to-column connection, the beam flange could aid in developing this resistance. However, the beam flange is not restrained for midspan gusset plates, and hence the web of the chevron-beam must resist these demands. If the yield stress of the gusset plate and the beam are the same, this simply means that the thickness of the beam web, t_w, should be equal to or greater than the thickness of the gusset plate, t_{gp} (in multistory X-bracing, the thicker gusset plate controls). This is a very simple check, but it will ensure that stress and force demands required to develop the buckling capacity of the brace are resisted by the beam. Further, this again illustrates why it is so important that the gusset plate be large and strong enough to develop the brace capacity, but no larger or stronger than needed. If a larger plate is used, a thicker web is required.

This simple check may be difficult to satisfy in practice, however, because it is likely that the gusset plate will be thicker than the lightest beam needed to support chevron-beam forces. Figure 4 illustrates the impact of stiffeners and the gusset plate-to-beam web thickness ratio. The validity of this approach is illustrated in Figure 4 for frames subjected to large inelastic deformations. Figure 4(b) shows a chevron-beam web that is thinner than the gusset plate, and a significant amount of local yielding can be seen in the beam web above the midspan gusset plate. Figure 4(a) has the same beam and gusset plate, but stiffeners are applied to the beam web and midspan gusset plate. At the same inelastic deformation as in Figure 4(b), the beam web adjacent to the midspan gusset plate is quite clear of all yielding. Figure 4(c) has a beam with web thickness equal to the gusset-plate thickness and no beam-web or gusset-plate stiffeners. While the thinner beam web with stiffeners performed better, this frame with the thicker web and no stiffeners performed acceptably.

The results indicate the stiffeners can help achieve the desired response and a web thickness less than the gusset plate can be used; further research is being conducted by the responders to provide more guidance on this aspect of design and detailing. While adding stiffeners might be viewed as an undesirable expense, they serve multiple purposes here, which justify their cost. The bending moment in a chevron beam is largest at the midspan gusset plate, which means that bracing against lateral-torsional buckling is also required at this location. In addition, stiffeners can be used to attach transverse beams or elements used for lateral support and aid in transferring brace forces to the chevron beam.

An X-braced frame with top and bottom gusset plates is even more likely to require stiffeners with thin webs because stiffeners will facilitate force transfer between stories. The size of the stiffeners could be based on the size of the balance of the gusset-plate and beam-web thicknesses. A similar approach can be used for corner gusset plates, but this could be relaxed when the beam flange is attached to the column because this attachment stiffens the flange so that it also facilitates transfer of forces from the gusset plate to the beam or column.

CONCLUSIONS

The responders recognize that this is a difficult topic and more research is needed to define the optimal design procedure. However, the responders contend that the research presented herein justifies a simpler and more reliable method for evaluating beams and midspan gusset plates in seismic design of chevron or multi-story, X-braced frames versus that proposed in the paper. This research clearly suggests the following design method:

1. The brace is designed to develop the seismic design forces required by ASCE/SEI 7.

2. The expected tensile and compressive capacity of these selected braces are used to perform a capacity-based design of the gusset plates. The gusset plates should be strong enough to develop the expected resistance of the brace, but additional strength beyond this requirement is discouraged because overly strong gusset plates will require larger welds and adversely affect braced-frame performance.

3. The welds or bolts joining the gusset plate to the beams and the columns should be strong enough to develop the resistance of the gusset plate, ensuring they are able to accommodate the deformation demands of the gusset plate due to out-of-plane brace buckling.

4. The beams of chevron (V- and inverted V-configuration) and multi-story, X-braced frames should be capacity designed to develop the expected shear force, bending moment, and axial force in the beam from maximum elastic and plastic brace forces, including deterioration of the compressive capacity.

5. Because the midspan gusset plate stiffens and strengthens the beam where they are attached, the demands should be evaluated in the beam *adjacent* to the midspan gusset plate. No stress calculations for the beam in the midspan gusset plate region are recommended.

6. The midspan connection should fulfill one of two requirements. If the thickness of the beam web is equal to or greater than the thickness of the gusset plate, no further design checks are required. Otherwise, when the thickness of the gusset plate exceeds the thickness of the beam web, vertical stiffeners to the beam web and the gusset plate should be employed.

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CLOSURE

Investigation on the Performance of a Mathematical Model to Analyze Concentrically Braced Frame Beams with V-Type Bracing Configurations

ALIREZA ASGARI HADAD and WILLIAM THORNTON

ABSTRACT

The discussion-paper published by Roeder et al. (2021), hereafter referred to as the responders for clarity, disputes the results published by Hadad and Fortney (2020) and the chevron effect (CE) analysis method published by Fortney and Thornton (2015, 2017), based on the responders' earlier research results and their understanding of the behavior of chevron beams. This paper provides the reply to the disputed topics.

WEAK BEAM VERSUS STRONG BEAM MECHANISMS

According to the literature, there are two failure mechanisms in chevron-braced bays: (1) a weak beam mechanism in which the buckling of the compression brace causes an unbalanced force to be applied on the beam and subsequently the formation of plastic hinges in the beam and where weak beams lead to considerable plastification in columns, beams, and braces, irrespective of brace slenderness, and (2) a strong beam mechanism where buckling of the compression brace is followed by yielding of the tension brace, the beam being sufficiently strong to remain elastic despite the vertical unbalanced force applied to it by the braces at mid-span. Chevron-braced frames (CBFs) with strong beams and weak braces exhibit a more uniform distribution of yielding with height than frames with weak beams and strong braces.

The intention behind explaining the mentioned mechanisms in the paper was to show the importance of chevron beams in determining the behavior of V-bracing and X-bracing CBF. The paper does not consider any mechanism superior to the other one. Below is the exact statement from the paper:

"It is not evident from the data in the literature which collapse mechanism is preferable." (Hadad and Fortney, 2020; emphasis added)

Although the responders consider the weak beam mechanism superior to the other failure mechanism based on their earlier research (Sen et al., 2016), they can review the other literature (Shen et al., 2015) to see how the weak beam mechanism is challenged. D'Aniello et al. (2015) showed the importance of chevron beam strength and stiffness in determining the behavior of concentrically braced frames. Thus, it is necessary to understand the actual demand on chevron beams and design them accordingly before taking a side on the controversial discussion about the desirable behavior of chevron beams. Thus, the authors investigated the performance of the CE method in the paper.

THE CHEVRON EFFECT ANALYSIS METHOD

The chevron effect (CE) method provides simple, closed-form solutions for normal and shear forces in the beam at the gusset to beam interface. There was no available information on these forces on the beam section until the CE analysis method (Fortney and Thornton, 2015, 2017) was published. Designers require a method to design these connections for buildings in a reasonable time frame. The responders criticize the authors' approach but provide no alternate method. Figure 3 in the responders' paper (Roeder et al., 2021) showed a broken line in the chevron region, which is exactly that part of the beam where the CE method shows the actual shear force and bending moment distributions. What do the responders propose for this region? The responders should have calculated the beam shear force demand for the connections in the analyzed frame using the CE method. The question of whether a beam that is inadequate per the CE method nevertheless performs adequately

Alireza Asgari Hadad, PhD, Structural Engineer, GEI Consultants, Inc., Marquette, Mich., Email: aahadad@geiconsultants.com (corresponding)

William Thornton, PhD, PE, NAE, Corporate Consultant, Cives Engineering Corporation, Roswell, Ga. Email: bthornton@cives.com

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seems obvious but is not addressed by the responders' analyses. If the beams, selected using other criteria, have sufficient strength per the CE method, their adequate performance can be interpreted as supporting the CE method.

The finite element studies of Hadad and Fortney (2020) were intended to verify the reasonable accuracy of the simple closed-form solutions of the original two papers. That goal was achieved. The results of the paper showed the chevron effect can be reasonably analyzed with simple closed-form equations without the use of finite element analysis. With the addition of AISC *Seismic Provisions* Section F2.6c.4 (AISC, 2016), even a seismic special concentrically braced frame (SCBF) design will likely be adequate.

It is worth noting that in buildings with composite floor systems, very little axial force will be carried to the beam-to-column connections. That is the reason Fortney and Thornton (2015, 2017) focused on estimating the shear force and bending moment distribution in chevron beams. The responders' considered the results of a bare 2D frame model to show the importance of axial load in chevron beams. The responders' test model is probably not representative of actual design conditions.

Figure 3 in the responders' paper showed a beam with axial forces. These can be handled by the CE method because it looks at the whole beam, including the chevron zone. The design shears and moments in the chevron region may be bigger than the design values shown in Figure 3.

Additionally, the responders' design relies on the fixity of the beam-to-column connection. This fixity induces shear in the beam that counteracts the chevron effect in the elastic range of drift. For elastic systems, the brace force direction (i.e., tension or compression) correlates with the drift direction such that both reverse in unison, and the offsetting of the beam-shear effects may be reliable. This correlation between brace force direction and drift direction likely holds for an SCBF. The large peak forces associated with the initial buckling strength of the brace in compression may occur only once, or once in each direction. For both of these peak-force conditions, the shear from the beam fixity reduces the chevron-effect shear force within the connection. However, for buckling-restrained braced frames, such correlation between drift direction and brace-force direction no longer holds after yielding of the braces. As such, the beam shear from the moment-frame behavior could be additive to the chevron shear. Recommendation to ignore the chevron effect generally and not limited to the specific configuration and system studied by the responders is irresponsible and could lead to unanticipated and potentially undesirable mechanisms in systems the responders have not considered or analyzed. The responders' failure to examine, or even consider, the statics of the connection results in their erroneous conclusion that the chevron does not exist because, in the case they studied, it is offset by another effect.

The authors do believe the CE method is not the ultimate method in the analysis of chevron configured brace frames, and thus, they appreciate and encourage the discussions on the CE analysis method and the efforts to further improve its accuracy. However, the inclusion of the chevron effect phenomenon, as confirmed by other researchers (Sabelli and Saxey, 2021), is a necessity to understand the shear force and bending moment demands on chevron beams. Until more accurate and convenient methods are available, the CE method is encouraged to be used in the analysis and design of new structures.

FINITE ELEMENT ANALYSES TO INVESTIGATE THE CHEVRON EFFECT METHOD

To evaluate the accuracy of the CE analysis method, the authors studied the shear force and bending moment distributions along the whole length of the beams in a group of beam-gusset assemblies through finite element analysis. The beam length in Figures 7 to 11 of the original paper is 300 in. as presented in Table 1 (Hadad and Fortney, 2020). The results published in the paper show:

- The existence of chevron effect and the success of the CE analysis method in identifying the phenomenon through its simple closed form solutions.
- The reasonable accuracy of the CE analysis method in estimating the beam shear force and bending moment in chevron beams, specifically in the beam-to-gusset plate connection region, while it requires no finite element analysis and a relatively small amount of time to be performed.

Because the paper investigated the performance of the CE analysis method that was introduced earlier (Fortney and Thornton, 2015, 2017), the same modeling approach was used in the finite element models (i.e., simply supported beams with gusset plate(s) in the middle carrying the brace loads).

The responders believe the chevron beams in braced frames should not be considered as simply supported beams due to the relative rigidity in the connection region. They also consider the gusset to be part of the beam increasing its strength. It could be helpful if the responders reported forces across section cuts at one or more chevron connections at the time of maximum shear force, such as shown in Figure 1. Such section cuts would reveal whether the shear force outside the connection region is offsetting the chevron shear, whether the moment on Section 1 is less than that determined based on brace axial forces (due to brace resisting moment), whether the gusset reinforces the web (as the proposed by the responders), or whether the distribution of shear force between the two is better described by the CE method.

Also, a reanalysis of the frame presented in the responders' paper without moment connections at the roof beam, perhaps with a beam that does not meet the required shear strength as determined using the CE method, could improve the discussion. The beam at the roof level in the responders' analyses appears to be a heavy beam with moment connections. Are such measures required in order for a design that does not consider the chevron effect to perform well?

Considering all the assumptions in the responders' analysis model, do the responders have a simple way of analyzing braced frames based on these considerations or have a complex finite element computer program is required? Until such time as a simple closed-form solution calculation method is available, the authors believe that the CE analysis method, as verified by the paper, is adequate to produce an acceptable design very quickly.

DUCTILITY FACTOR AND STRESS DISTRIBUTIONS IN THE BEAM-TO-GUSSET PLATE CONNECTION INTERFACE

The ductility factor is considered in the literature (Richard, 1986; Hewitt and Thornton, 2004) as one possible solution to accommodate the stress concentrations in the design of welds connecting gusset plates to beams. The authors decided to examine the value of this factor based on the results obtained from the chevron beam-gusset assemblies studied in the research (Hadad and Fortney, 2019, 2020). To do so, the authors compared the actual stress distribution at gusset-to-weld interface versus the average of the stress values. None of the stress values were calculated based on CE method. The results clearly showed the factor needs to be changed.

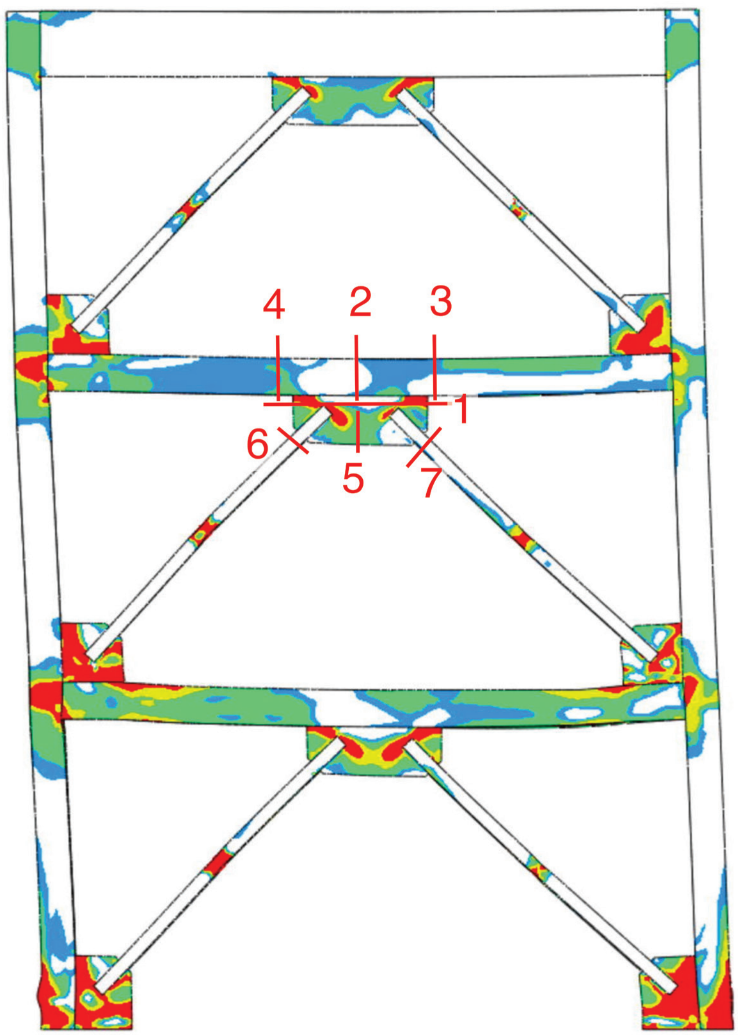

Fig. 1. Suggested sections that could be studied in the responders' model.

With respect to Figure 2 in the responders' discussion, it should be noted that this test was one of two tests that were identical except for the weld sizes. The weld sizes in the Figure 2 tests were sized for the gusset edge forces while the identical test had the weld increased by the AISC ductility factor of 1.25. In this second test the welds survived the fracture of the brace with no cracks. The requirement of AISC *Seismic Provisions* Section F2.6c.4 (AISC, 2016) is a direct result of the Figure 2 test, but did not explicitly involve a chevron gusset.

The solution suggested by the authors to use the gusset plate shear strength in designing the weld connecting the gusset plate to the chevron beam is obviously not the only possible solution. It is just a suggestion by the authors. Other researchers are highly encouraged to participate in this effort and suggest better solutions.

CONCLUSION

The chevron effect analysis method is the pioneering method in identifying the chevron effect in chevron configured concentrically braced frames. The method provides simple closed form solutions for structural designers to estimate the shear force and bending moment distributions along the chevron beam length without requiring them to run computationally expensive finite element analyses. The performed investigations showed the reasonable accuracy of the method.

The authors appreciate the attention of the responders, the reviewers' helpful comments, and other structural engineers and researchers for their feedbacks on the method since it has been published. Such comments help the authors in improving the chevron effect analysis method.

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