DISCUSSION

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

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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

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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

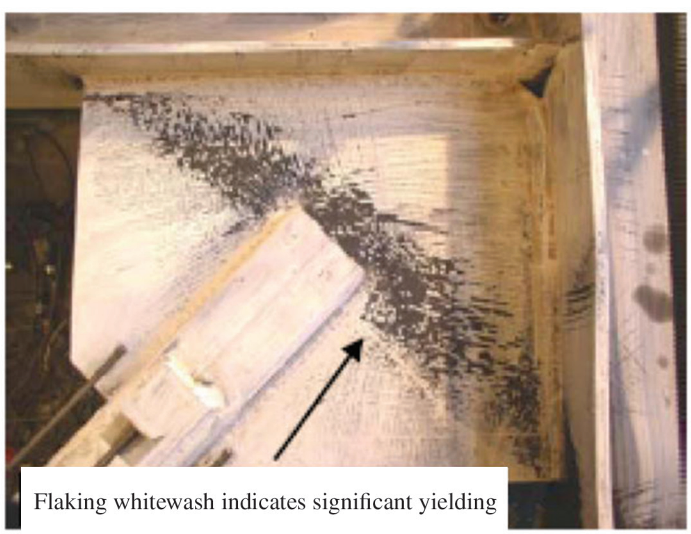
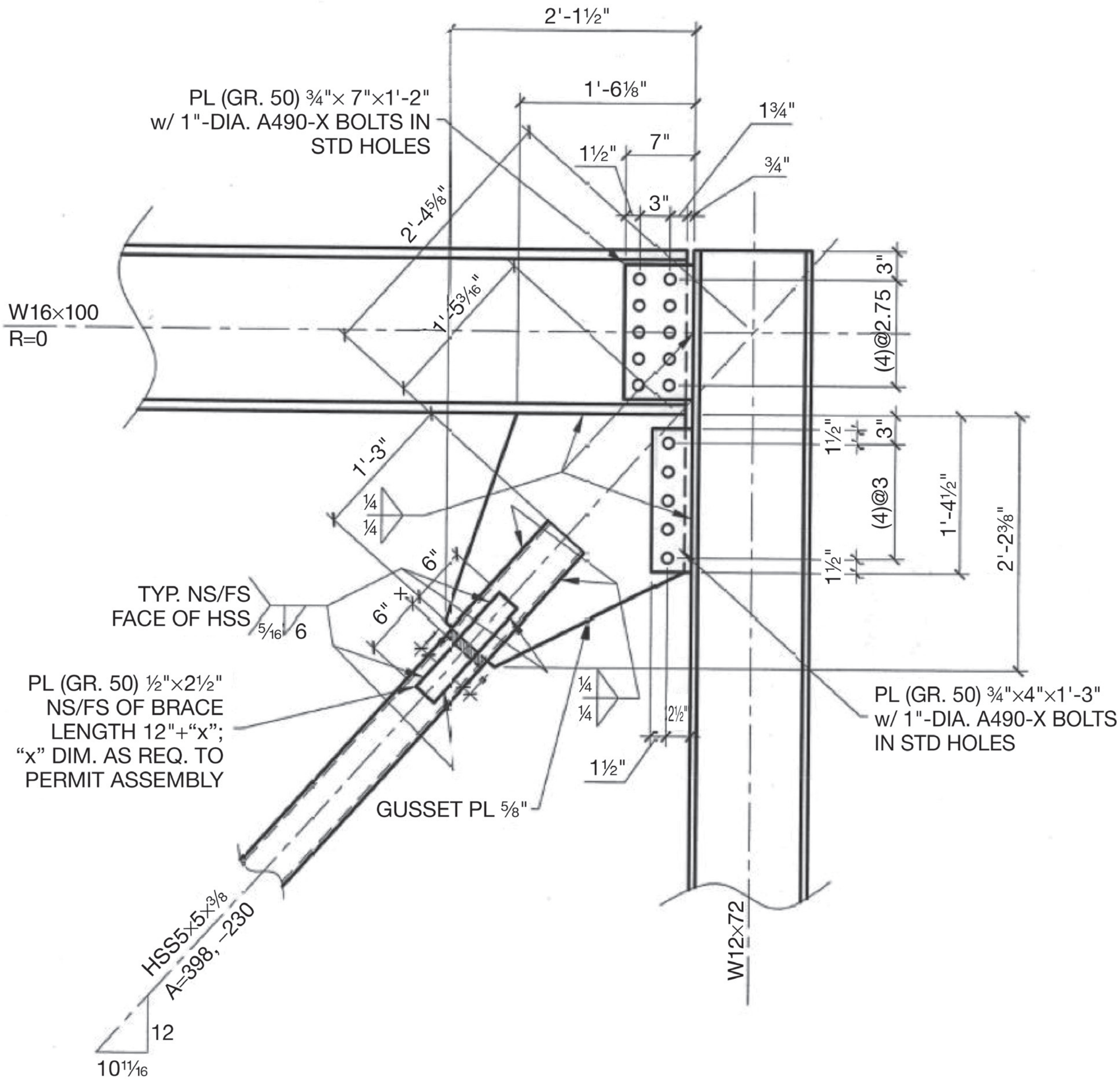
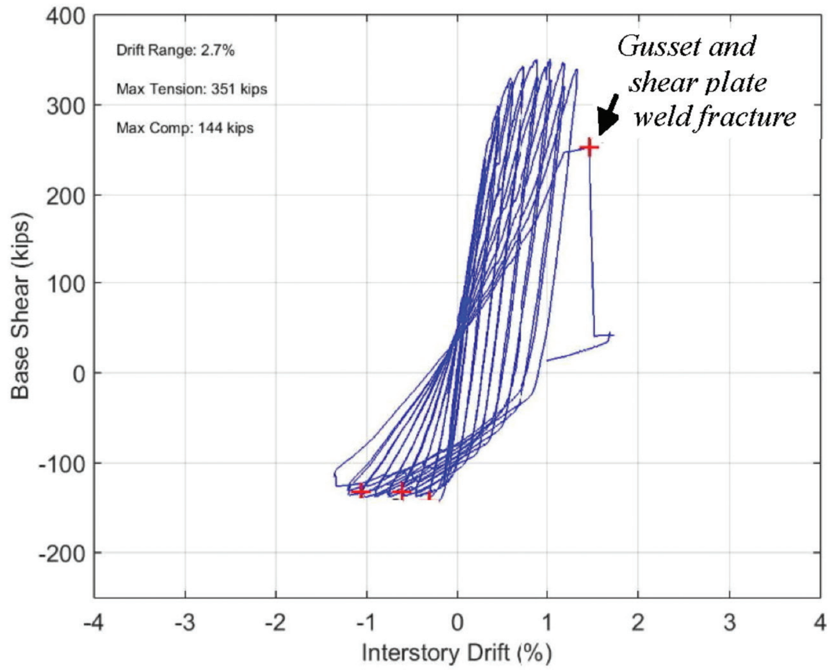


Fig. 1. Inelastic deformation of a gusset plate under seismic deformation.



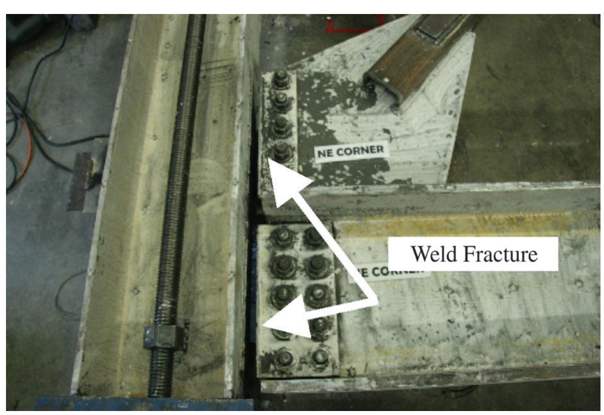
(a) Gusset-plate connection based on expected brace forces

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

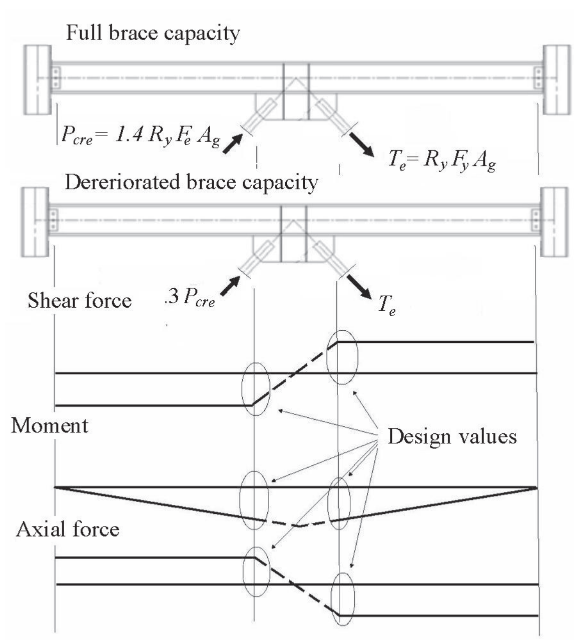
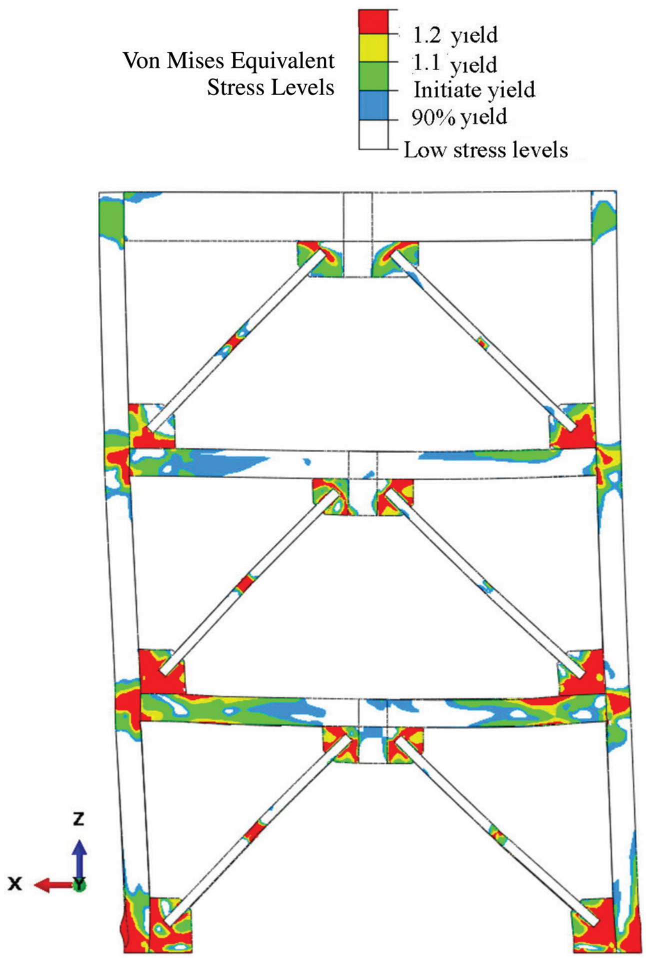
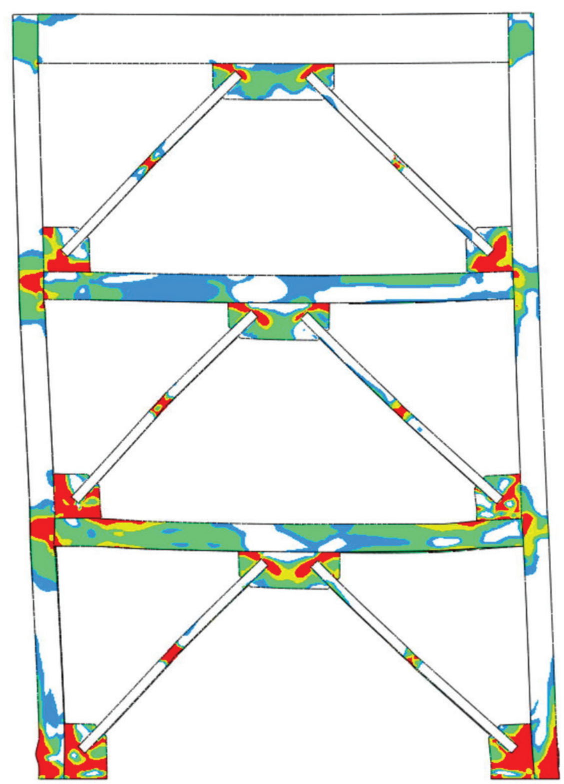


Fig. 3. Axial-load, shear-force, and bending-moment diagrams for idealized boundary condition for capacity-based design of chevron beams.



(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*

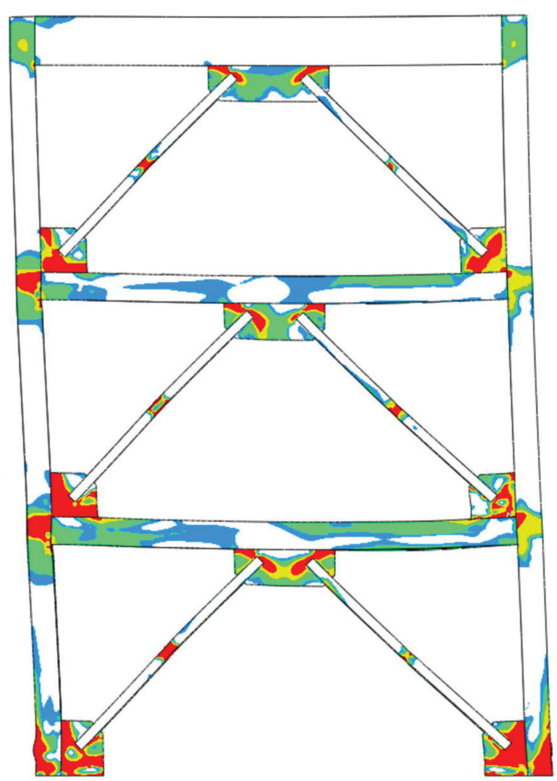
Fig. 4(b). Nonlinear analyses of chevron-braced frames.

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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