Practice-Accessible Methodology for Nonlinear Refined Analysis of Gusset Plate Connections of Steel Truss Bridges

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ABSTRACT

A new approach is proposed for estimating structural capacity of gusset plate connections of steel truss bridges. The approach involves nonlinear refined analysis of a truss model, consisting of a single truss made of shell elements for two gusset plates at a subject connection and frame elements for truss members. Connectors (bolts or rivets) are excluded from the proposed model in order to simplify the model, with the knowledge that their associated failure modes can be addressed by simplified design calculations. Only yielding and buckling failure modes of gusset plates are considered. The new approach is calibrated by comparison with laboratory test data from NCHRP Project 12-84. The primary intent of this paper's approach is to reduce the complexity of the refined analysis developed under this National Cooperation Highway Research Program project and make it more user-friendly to load rating engineers seeking accurate estimation of gusset plate capacity. Further, an imperfection sensitivity curve characterizing reduction in capacity as a function of lateral imperfection (gusset plate out-of-flatness) in gusset plate geometry is introduced as an added tool for the load rating engineer, providing more confidence in capacity estimation. Based on a limited case study, the proposed refined analysis provides an estimate of gusset plate capacity that is approximately equal to capacity calculated by the truncated Whitmore method of the AASHTO *Manual for Bridge Evaluation* (2018). In other words, the proposed approach validates the truncated Whitmore method as compared to the more conservative partial shear method of the AASHTO *Manual*.

Keywords: steel truss bridge, gusset plate connection, finite element analysis, load rating, inelastic compression buckling, nonlinear buckling.

BACKGROUND

The history of steel truss bridges goes back to the late 19th century when steel displaced wood and iron as the material of choice for truss bridge construction. However, the first significant experimental study on steel gusset plate connections was performed by Wyss (1926), followed by several successive substantial studies in the next decades (Sandel, 1950; Whitmore, 1952; Irvan, 1957; Hardin, 1958; Vasarhelyi, 1971). These research efforts indicated that the maximum tensile and compressive stresses in gusset plates develop at the end of the truss members. Whitmore conducted a set of experiments on Warren truss joints of a small-scale model. Based on the results, he proposed his effective width concept known as the Whitmore section, formed of a line through the end row of rivets/bolts intersected by two lines radiating outward at 30° angles from the first row of rivets/bolts. Thereafter, common design practice for gusset plates has consisted of (1) using beam theory over general sections and calculating the resultant moment and shear on a free-body diagram (for overall failure consideration); (2) adopting the Whitmore section method for tensile/compressive stresses at the end of truss members (for local buckling and yielding considerations); and (3) rivet/bolt shear, block shear, and hole bearing failures (for fasteners and gusset plate-fastener interaction considerations).

In 2007, the collapse of the I-35 bridge in Minneapolis led the bridge engineering community to reevaluate contemporary measures of safety and reliability of gusset plate connections of steel truss bridges. The investigations conducted by the National Transportation Safety Board (NTSB) indicated compression buckling in gusset plates as the initial cause for the subsequent bridge collapse (NTSB, 2008). In response to this disaster, the Federal Highway Administration (FHWA) issued recommendations for supplementary gusset plate load ratings for non-load-path-redundant steel truss bridges as well as new AASHTO procedures for design and load rating of gusset plates. However, there was not sufficient knowledge to address the concerns raised by the I-35 bridge collapse at the time. On that basis, NCHRP Project 12-84 (Ocel, 2013) was commissioned in 2008 (and completed in 2013) to develop the load and resistance factor design (LRFD) and load and resistance factor rating (LRFR) guidelines for riveted and bolted gusset plate connections.

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That research consisted of comprehensive experimental and analytical investigations to explore failure modes of gusset plates and reliability calibration of resistance factors for shear and buckling limit states.

Regarding gusset plate buckling resistance, National Cooperation Highway Research Program (NCHRP) research findings indicate a considerable difference in buckling failure modes between tight connections with chamfered diagonals and typical connections with nonchamfered members. In tight connections, buckling occurs after significant shear yielding along the chamfered edge. While in typical nonchamfered connections, the buckling occurs following a slight compressive yielding in the gusset plate region at the end of the compression diagonal member. Based on these findings, two alternative methods are proposed by the NCHRP research team for buckling resistance estimation: Method 1 is a twofold buckling resistance estimation including Whitmore section buckling along with a proposed partial shear yielding check. Method 2 is a revised Whitmore method known as the truncated Whitmore method. These two methods have been adopted by the AASHTO *Manual for Bridge Evaluations* (MBE) (AASHTO, 2018) for gusset plate load rating. However, the truncated Whitmore method is discussed in the MBE Commentary. Despite the calibration conducted in the NCHRP research, there is substantial uncertainty about gusset plate buckling parameters, including the buckling length and assumed effective length coefficient, *K*. The research findings also indicated a large discrepancy between actual magnitude and location of maximum stresses in gusset plates using beam theory. For shear yielding, this discrepancy is addressed by including a reduction factor. However, this factor is also associated with uncertainty. These uncertainties arise due to a vast variety of gusset plate geometries and, consequently, significant variation in boundary conditions of gusset plates in the local yielding/buckling region. Considering the safety margins applied for generalization of these parameters, the MBE prescriptive gusset plate capacity estimation approach involves significant conservativeness that may result in unnecessary repair recommendations. Alternatively, an authentic analytical simulation approach may provide a more realistic estimation of gusset plate yielding/buckling resistance.

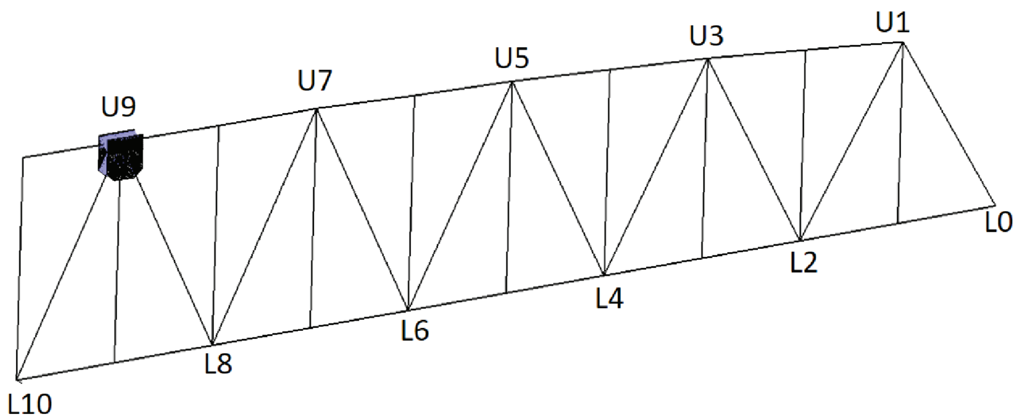
The AASHTO MBE allows employing refined finite element simulation to determine nominal resistance of gusset plate connections, particularly when the MBE prescriptive capacity estimation approach results in unacceptable load rating based on compression yielding or buckling criteria. The MBE Commentary refers to NCHRP research modeling attributes as a reliable approach that was able to predict gusset plate capacities with less than 10% error as compared to experimental testing results. Nevertheless, following strictly the same approach is not required, and simpler modeling is considered acceptable depending on the target failure model under investigation. The complexity of the NCHRP research modeling approach may hinder its implementation in common load rating practice. This paper proposes a more practical finite element modeling approach to estimate gusset plate capacity associated with yielding and buckling failure modes for typical double gusset plate connections of steel truss bridges.

PROPOSED NONLINEAR FINITE ELEMENT ANALYSIS APPROACH

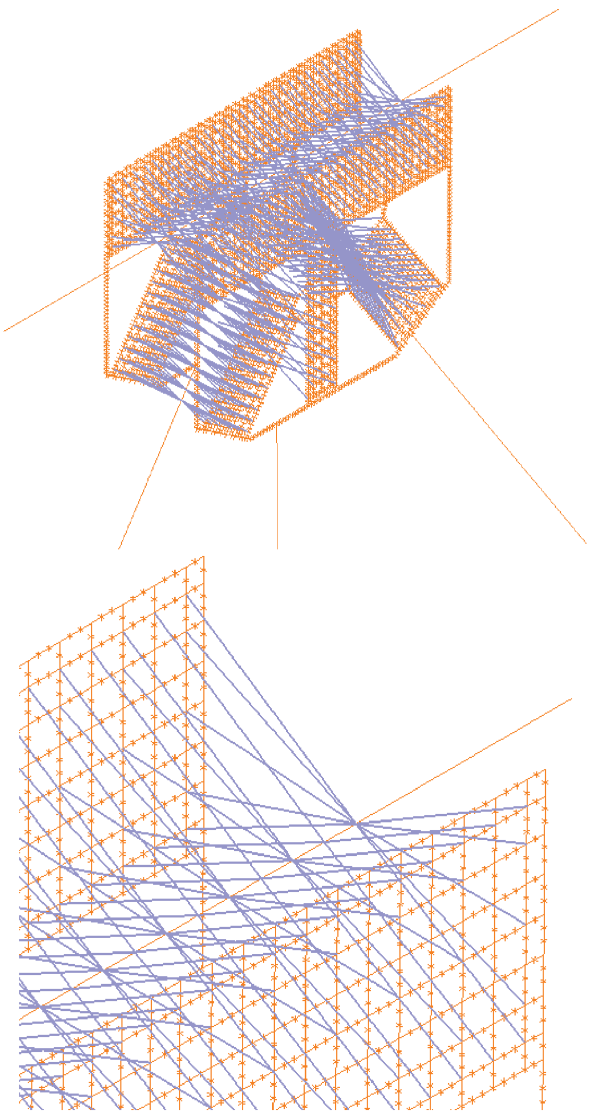
The main concept of the alternative nonlinear finite element method (FEM) modeling and analysis approach presented in this study is to simplify an ideal full comprehensive approach by excluding model attributes related to failure modes that can be confidently evaluated by the current MBE prescriptive calculations, as well as other attributes having negligible effects on capacity estimation of remainder failure modes. Failure modes related to fasteners and gusset plate-fastener interactions, including shear resistance of fasteners, block shear resistance and hole bearing resistance, can be well evaluated with the MBE equations. Also, the NCHRP research results indicated that modeling of fasteners holes is not required for yielding and buckling failure modes. Therefore, fasteners and fasteners holes are not included in this alternative FEM modeling approach. However, proper modeling of load transfer between gusset plates and truss members at fastener locations is considered as a necessary modeling attribute in this approach.

Analytical simulations conducted under the NCHRP research indicated the necessity of utilizing shell elements to accurately predict gusset plate connection failure modes and related resistances. Besides, geometry and material nonlinearities along with inclusion of initial geometric imperfections are acknowledged as other essential modeling attributes to create an authentic FEM model for gusset plate connections. Accordingly, the mentioned attributes are considered as the basis of the proposed FEM modeling approach for gusset plate yielding and buckling failure modes.

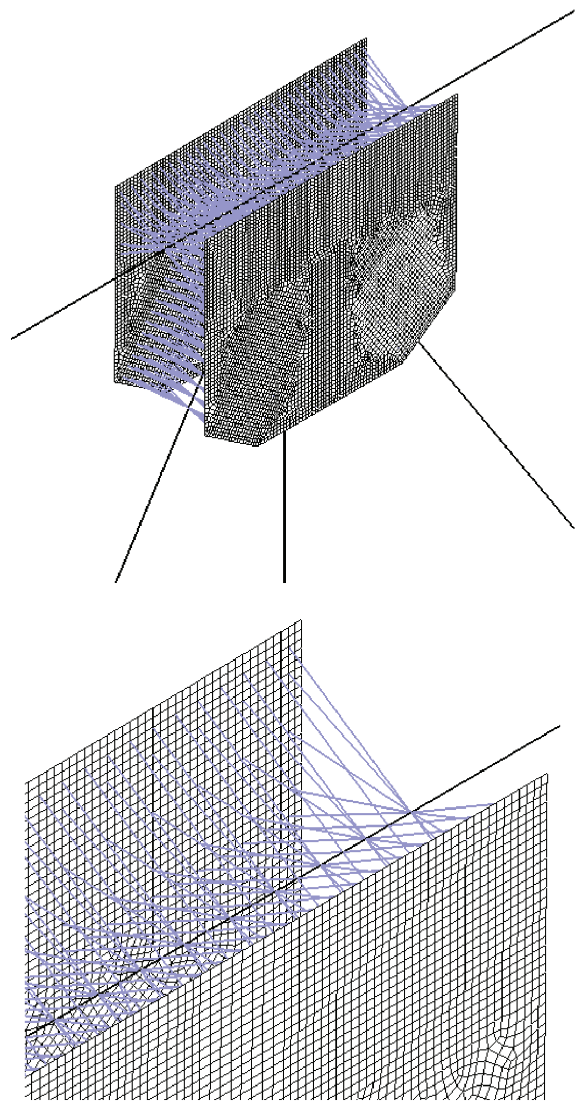
Figure 1 illustrates FEM simulation for a subject Warren truss with refined gusset plate modeling at a top joint for gusset plate capacity investigations. Details of this simulation and the performed nonlinear analysis are discussed next and demonstrate application of the proposed approach. As shown in Figure 1(a), the model includes a full truss with a refined gusset plate model at joint U9. The full truss is simulated via ADINA v9.3 (Adina, 2017) using truss elements, except at panels 9 and 10, where truss members are modeled with beam elements to accurately present the force and moments induced in the gusset plate connection. Instead of a full-truss model, a two-panel truss system including the



(a) Full 3D truss model incorporating refined gusset plate connection modeling at joint U9



(b) Gusset plate connection modeling before element meshing



(c) Gusset plate connection modeling after element meshing

Fig. 1. An example of the proposed FEM modeling approach.

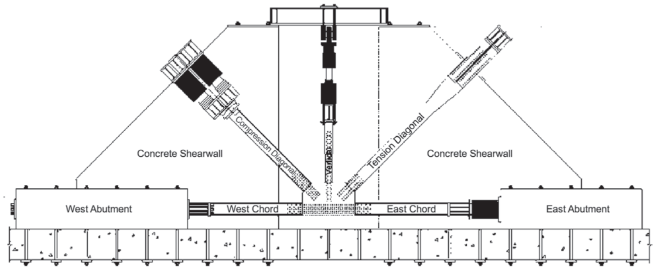
joint and two adjacent panels is acceptable for FEM simulation as recommended in the AASHTO MBE. However, boundary conditions and applied truss loads at the boundary joints shall be accurately set to guarantee the validity of the obtained results. In the NCHRP research modeling approach, the truss members attached to the gusset plates are modeled with a shell element for a distance of two member depths away from the gusset plate edge. This will significantly complicate the modeling of the truss members and gusset plate-to-truss member attachments. It will also significantly increase the time of modeling and analysis. Alternatively, in this proposed approach, the members are fully modeled with only beam elements. As shown in Figures 1(b) and 1(c), rigid links are provided to connect the truss members to the gusset plates at the fastener locations to precisely model the load transfer between them. In this approach, gusset plates are modeled using a combination of quadrilateral and triangular shell elements due to the gusset plate's complex geometry. However, quadrilateral elements constitute the majority of the elements. Utilizing triangular shell elements is relatively less favorable because it is associated with larger solution errors. Nonlinear material models should be used for gusset plate shell elements to capture the yielding, stress redistribution, and stress hardening.

To incorporate geometric instability (buckling) failure modes into the gusset plate FEM model, proper inclusion of initial imperfection (gusset plate out-of-flatness) along with a gradually increasing load need to be applied until termination of the analysis due to structural instability. According to the AASHTO MBE, the maximum initial imperfections should be limited to the smaller of 1/150 of the longest free edge length, 10% of the gap between the end of the compression member and the next adjoining member, or the thickness of the gusset plate. However, an adequate application of this imperfection to the model is not suggested by the MBE. In the NCHRP research, the initial imperfection was applied to both the gusset plate and the compression diagonal member. The imperfection was applied by a separate linear analysis imposing transverse pressure to the end of compression diagonal until reaching the desired out-of-flatness. This will generate an initial imperfection shape following the gusset plate's first buckling mode, which is a transverse sway mode. However, this approach may result in unrealistic locked-in stresses induced by the applied imperfection. Moreover, due to uncertainty about actual imperfection, applying a single initial imperfection without considering the gusset plate's buckling sensitivity to initial imperfection may result in an underestimation of the imperfection effect. Alternatively, in this proposed approach, the initial imperfection is included with a transverse load applied at the end of the compression diagonal beam element simultaneously increasing with truss loading steps. The buckling sensitivity is studied by applying a reasonable range of the mentioned imperfection simulating load. This will provide the load rating engineer with a full picture of the buckling resistance sensitivity and helps the engineer for more reliable gusset plate buckling resistance estimation.

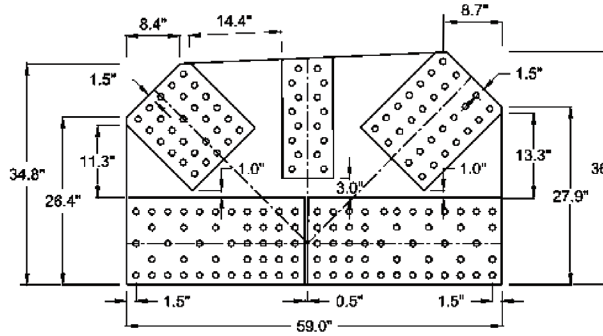
VALIDATION ANALYSES FOR THE PROPOSED MODELING APPROACH

A set of validation analyses was conducted to evaluate the accuracy and reliability of the proposed nonlinear FEM analysis approach. For this purpose, full-scale experimental tests conducted under the NCHRP research for specimens E1-WV-307SS were simulated utilizing the approach proposed in this study. The ADINA v9.3 finite element package was used to generate structural models and conduct gusset plate connection nonlinear analyses. Figure 2(a) shows the load frame used in the NCHRP tests to simulate the loading applied on a double-gusset plate connection at the lower joint of a Warren truss configuration with vertical truss members. As shown, the truss loads were applied through five independent jacking systems at the end of the truss members. The experimental testing included four E1-WV-307SS specimens, all with the same gusset plate geometry as shown in Figure 2(b) but different thicknesses, including 0.25 in., 0.3125 in., 0.4375 in., and 0.5 in. Reported test observations indicated gusset plate inelastic buckling at the compression diagonal as the failure mode for all four specimens. Accordingly, this specific set of specimens was selected for this validation to thoroughly investigate the validity of this analysis technique to capture localized initial yielding and consequent inelastic buckling of gusset plates at the compression diagonal.

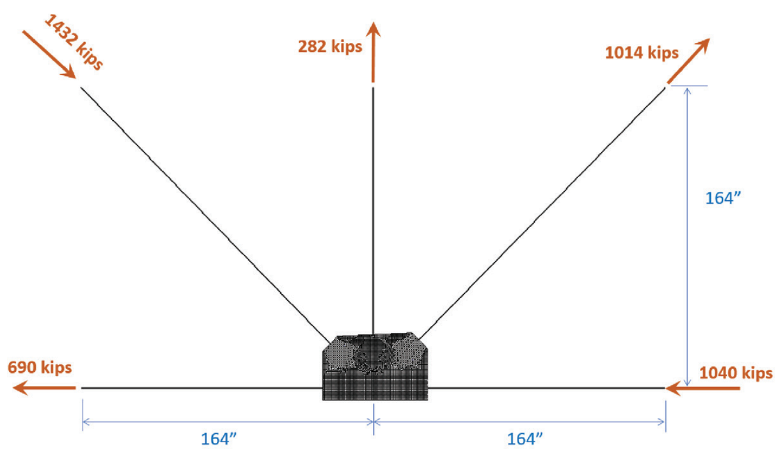
Figures 2(c) and 2(d) show the 3D FEM model developed for the validation analysis. The proposed modeling approach explained in the earlier section was utilized in this model. The model includes a shell element for gusset plates with nonlinear steel material model, elastic beam-element truss members (chords, diagonals, and the vertical), and rigid links connecting the truss members to the gusset plates at actual bolt locations. Figure 2(c) shows the truss loads applied at the end of the truss members. The loads are scaled up to twice the failure loads obtained for the 0.5-in. thickness. The larger loads were applied to ensure capturing simulation failure resistances higher than the reported test failure resistances. The loads were applied gradually with 1% increments along with software's automatic incrementation control (for further substepping at analysis diverging steps) to ensure capturing material and geometric nonlinearity until final failure. The loading step corresponding to the failure of analysis is presented as the applied load factor (ALF). ALF represents the fraction of total load at the last analysis step. The boundary conditions at the end of



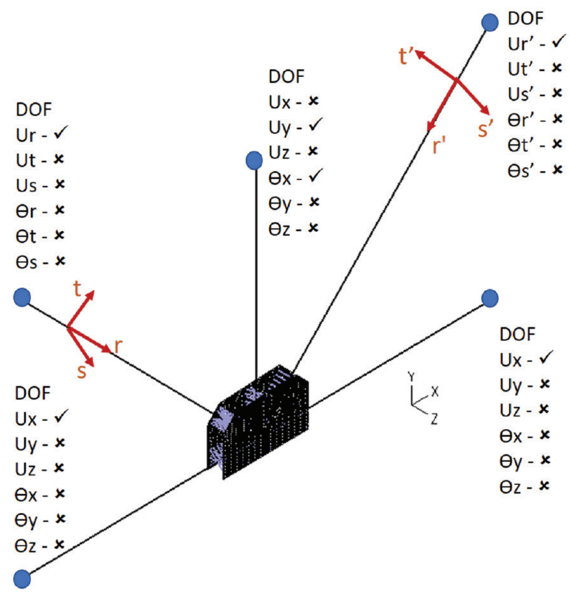
(a) NCHRP test load frame



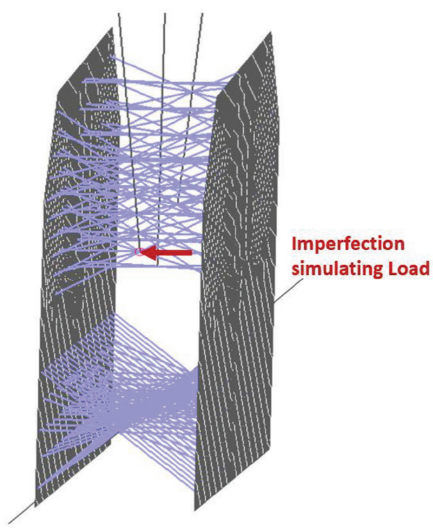
(b) Geometry of E1-WV-307SS specimens



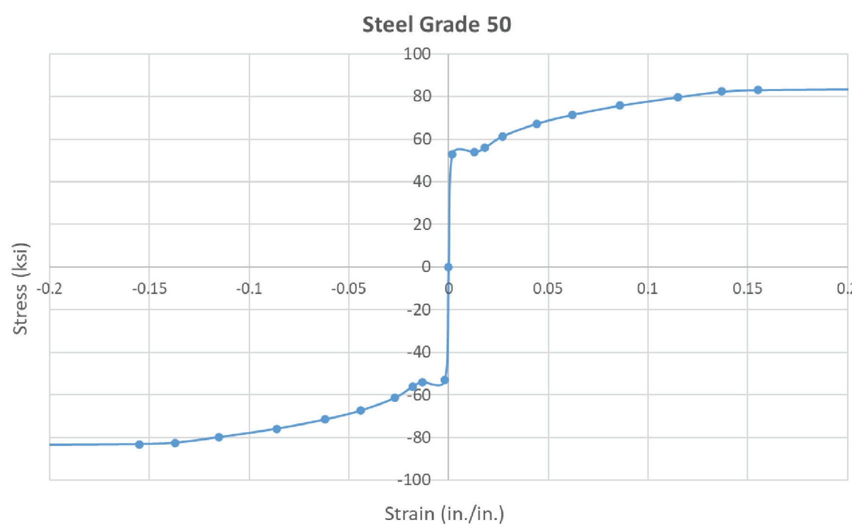
(c) Test simulation model, loads and dimensions



(d) Test simulation model, boundary conditions



(e) Imperfection simulating load



(f) Stress-strain relationship for Grade 50 steel

Fig. 2. E1-WV-307SS NCHRP test simulations.

truss members are shown in Figure 2(d) (U for translational, and θ for the rotational degrees of freedom). As shown, all members are only free for translational movement in their longitudinal directions, except the vertical member, which has freedom for out-of-truss plane rotation. These translational and rotational fixities correspond to the constraints provided at the jacking setups. As shown in Figure 2(d), secondary coordinate systems were defined for diagonal members to facilitate the applications of loads and boundary conditions for the diagonal members. As illustrated, these secondary coordinate systems have an axis along the diagonals (r and r'), a perpendicular axis within the truss plane (t and t'), and the third axis normal to the truss plane (s and s'). The bottom chord was modeled continuous over the gusset plate due to significant ample splicing plates provided in testing.

As explained earlier, in this proposed analysis approach, the initial imperfection of gusset plate connections is applied via a single transverse load at the beginning of the compression diagonal member. The intent is simplicity and practicality of using this approach in common load rating practice. Figure 2(e) shows the imperfection simulating load in this modeling. This load simulates an initial imperfection following the connection's transverse sway buckling mode. Similarly, alternative loading may be utilized when other gusset plate buckling modes are of concern. Figure 2(f) shows the nonlinear steel Grade 50 material model (per the NCHRP research report) used for gusset plate shell elements.

For each of the four test simulations, nonlinear analyses were performed by incorporating different amounts of imperfection simulating loads. Each applied imperfection load is also presented by an equivalent out-of-plane imperfection eccentricity of the gusset plates at the compression diagonal. This imperfection eccentricity is comparable to the out-of-plane imperfection referred to in AASHTO MBE. The equivalent imperfection eccentricity for each applied imperfection simulating load was obtained by equating the out-of-plane bending moments at the joint center resulting from the imperfection simulating load with the equivalent imperfection eccentricity. Figure 3 shows the imperfection sensitivity curves for all four test simulations. Each imperfection sensitivity curve presents the rate for gusset plate capacity reduction for the increase in the magnitude of out-of-plane initial imperfection. As shown in Figure 3, for all four simulations, the imperfection sensitivity follows the same trend with a relatively sharper capacity reduction at the beginning for smaller amounts of initial imperfection, followed by an almost constant reduction rate for the larger amounts of initial imperfection.

For each test simulation, the calibrated nonlinear FEM analysis was recognized by identifying the initial imperfection, which resulted in about the same gusset plate capacity as reported for the experimental testing. Table 1 presents the calibrated FEM analyses for all four simulation cases. As shown, the calibrated imperfection loads increase for specimens with gusset plate thickness increases. A comparison between the gusset plate thicknesses and the corresponding

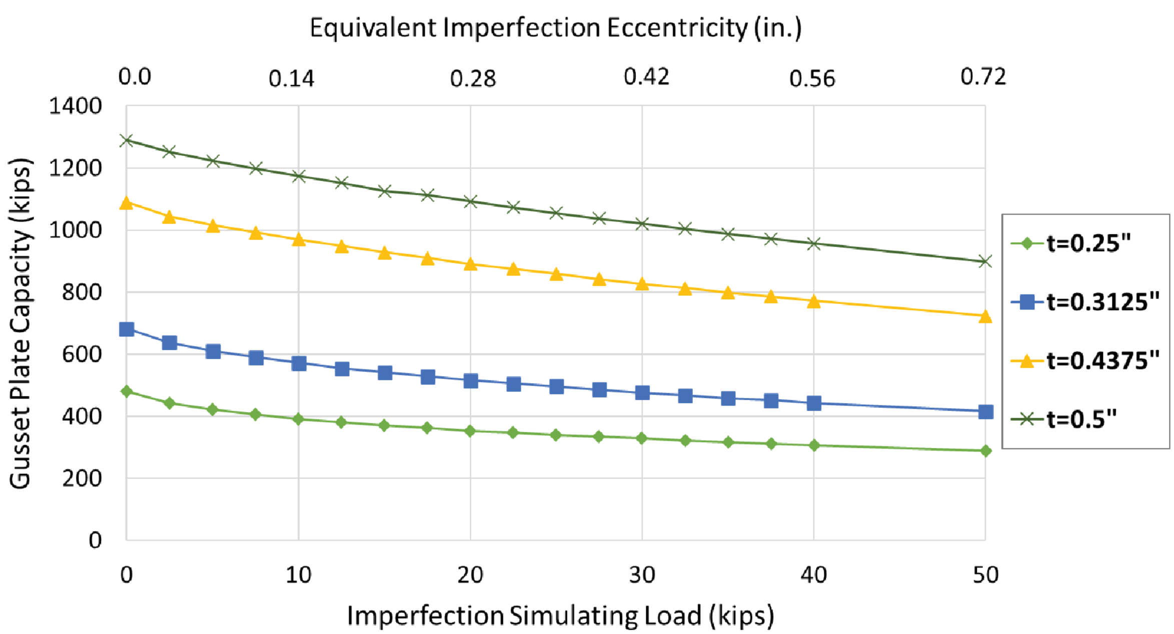


Fig. 3. Imperfection sensitivity curves for E1-WV-307SS test simulations.

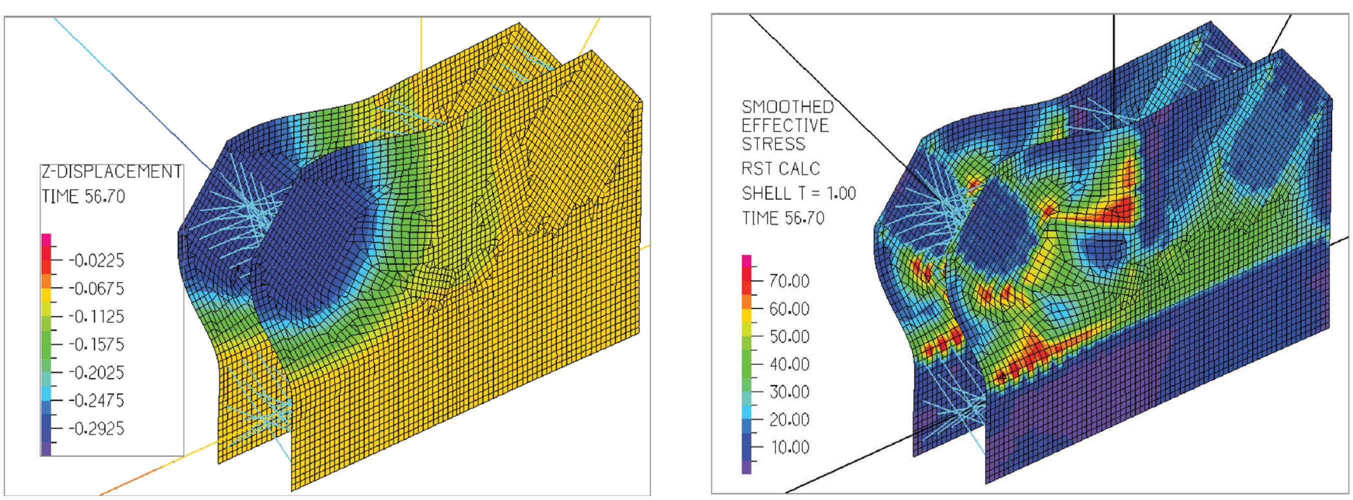
Table 1. Calibrated Gusset Plate Nonlinear FEM Analysis Results for E1-WV-307SS Test Simulations						
Simulations of NCHRP E1WV-307SS Specimens	Gusset Plate Thickness (in.)	Test Failure Resistance (kips)	Calibrated Imperfection Simulating Load (kips)	Calibrated Equivalent Imperfection Eccentricity (in.)	Applied Load Factor (ALF)	Simulation Resistance (kips)
	0.25 (¼)	380	12.5	0.180	0.266	381
	0.3125 (5⁄16)	530	17.5	0.250	0.369	529
	0.4375 (7⁄16)	817	32.5	0.460	0.567	812
	0.5 (½)	974	37.5	0.530	0.678	971

calibrated imperfection eccentricities presented in the table indicates their comparability with each other. In comparison to AASHTO MBE criteria for the magnitude of out-of-plane imperfection, this proposed method requires relatively larger imperfections due to excluding imperfection-induced locked-in stresses.

Figure 4 shows the calibrated nonlinear analysis results for the 0.4375-in. specimen at the failure stage (ALF = 0.567). Figure 4(a) shows the deformed gusset plates at the final analysis stage just before the analysis termination due to instability. The shown transverse displacement contours indicate sway buckling at the compression diagonal in conformity with the observed failure mode reported for the corresponding testing. Figure 4(b) illustrates the von Mises stress contours at the same final stage of the analysis. As shown, the gusset plates underwent significant yielding at the edge of the chord and vertical member at the compression diagonal side of the connection. This set of simulation analyses indicate the accuracy and reliability of the proposed nonlinear FEM analysis approach in the estimation of gusset plate capacity under inelastic buckling failure.

UTILIZING THE PROPOSED APPROACH FOR A CASE STUDY

The proposed FEM analysis approach was utilized for investigating the gusset plate connection capacity for a subject steel truss shown in Figure 5. This example demonstrates the benefit of using this analysis approach for load rating practice. The gusset plate capacity of joint U9 connection as calculated by the LRFR method of AASHTO MBE is controlled by partial shear failure; the factored capacity equals 1792 kips for the compression diagonal member. The truncated Whitmore method, which is an alternative method in the AASHTO MBE Commentary, resulted in a gusset plate factored capacity of 2574 kips. This significant discrepancy resulted in inventory HL-93 load rating factors of 0.42 and 1.44 for the partial shear and truncated Whitmore methods,



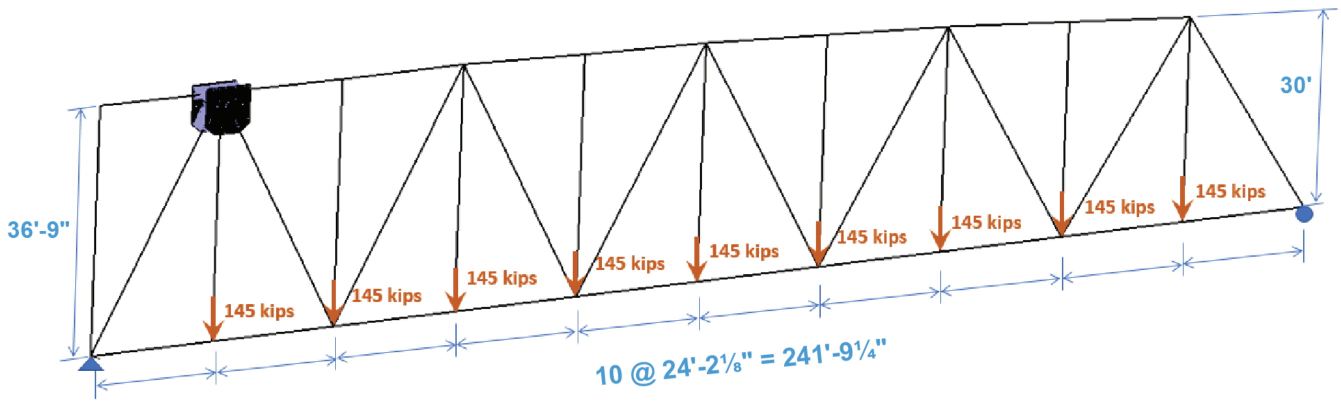
(a) Lateral deformation contours

(b) Von Mises stress contours

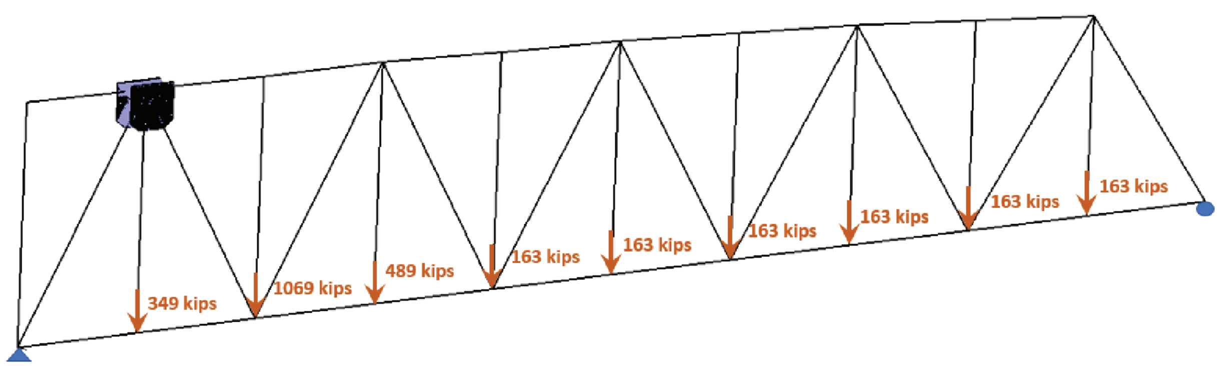
Fig. 4. Gusset plate FEM nonlinear analysis results for E1-WV-307SS test, 0.3475-in. specimen with 32.5-kip imperfection load at ALF = 0.567.

respectively. In the absence of utilizing a reliable and practical refined analysis method, a load rating engineer may conservatively recommend unnecessary retrofit for this gusset plate connection. With the aid of the proposed nonlinear FEM analysis approach, the capacity of the gusset plate connection was investigated in this study.

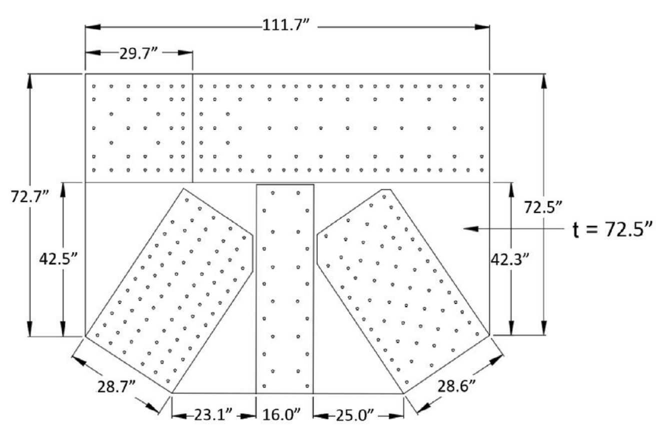
Figure 5 shows the geometry details of the studied steel truss along with a nonlinear material model. The FEM modeling details for this subject truss and refined modeling of joint U9 are shown in Figure 1 and discussed in an earlier section of this paper. Figure 5(a) shows the geometry and applied dead loads for the subject truss. Simulation analysis was conducted by first applying the dead load in a single-step analysis. The analysis continued by gradually applying



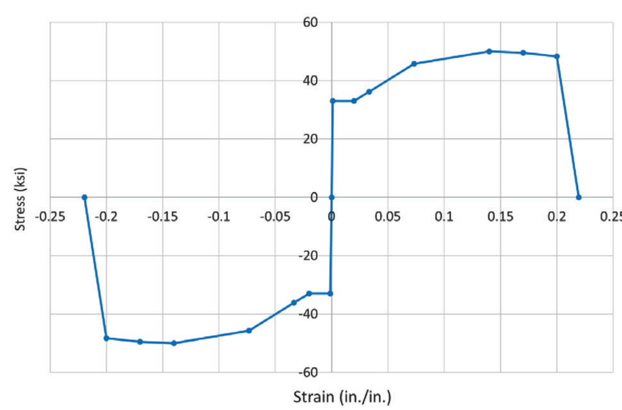
(a) Model geometry and dead loads



(b) Scaled-up live loads



(c) Details of gusset plates at joint U9



(d) Nonlinear material model for $F_y = 33$ *ksi carbon steel*

Fig. 5. Studied steel truss.

scaled-up HL-93 live load reactions shown in Figure 5(b) until analysis termination due to structural instability. The geometry of joint U9 gusset plates is shown in Figure 5(c). As shown, both diagonals are slightly chamfered at the connection. Late 1930s carbon steel with F_y = 33 ksi was used for truss members and gusset plates. The corresponding nonlinear material model used for gusset plates is shown in Figure 5(d). The top chord splice plates were not included in the model, and instead, a continuous top chord was considered in this simulation. This was reasonable because the chords are supplied with splice plates on all four faces, which significantly limits the contribution of gusset plates in taking splicing forces. For other cases where a significant splicing action is expected, the modeling can be revised by discontinuing the chord and adding splice plates and rigid links and following the same approach used for gusset plate modeling.

Figure 6 shows an imperfection sensitivity curve for the subject gusset plate connection developed by conducting 3D nonlinear analyses for a range of imperfection loads and associated eccentricities. As shown, the gusset plate capacity reduction is relatively slow for an initial increase in imperfection load/eccentricity, but the curve becomes steeper for larger imperfections. For this subject gusset plate connection, the imperfection eccentricity was assumed equal to the gusset plate thickness. According to the imperfection sensitivity curve, this imperfection eccentricity corresponds to the gusset plate capacity of 2503 kips. Comparing the gusset plate capacity obtained from the refined analysis with partial shear and truncated Whitmore method capacities indicates that the refined analysis confirms the truncated Whitmore method's estimation. Accordingly, this also suggests no need for gusset plate retrofit. Figure 7 presents the nonlinear FEM refined analysis results, at the final stage before buckling failure, for the subject gusset plate connection with the imperfection eccentricity equals to the gusset plate thickness. Gusset plate transverse displacement contours are shown in Figure 7(a). As shown, buckling of gusset plates at the compression diagonal is the failure mode for the subject connection. Von Mises stress contours presented in Figure 7(b) indicate significant gusset plate yielding prior to final inelastic buckling. As shown, the gusset plates mostly yield along the horizontal section at the bottom edge of the top chord before failure.

SUMMARY AND CONCLUSIONS

A 3D nonlinear refined FEM analysis approach for gusset plate capacity estimation is proposed in this paper. The main intention of this approach is to provide a less complicated but still reliable refined FEM modeling and analysis technique to be utilized in common practice of gusset plate load rating. For this purpose, the proposed modeling excludes modeling details related to failure modes that can

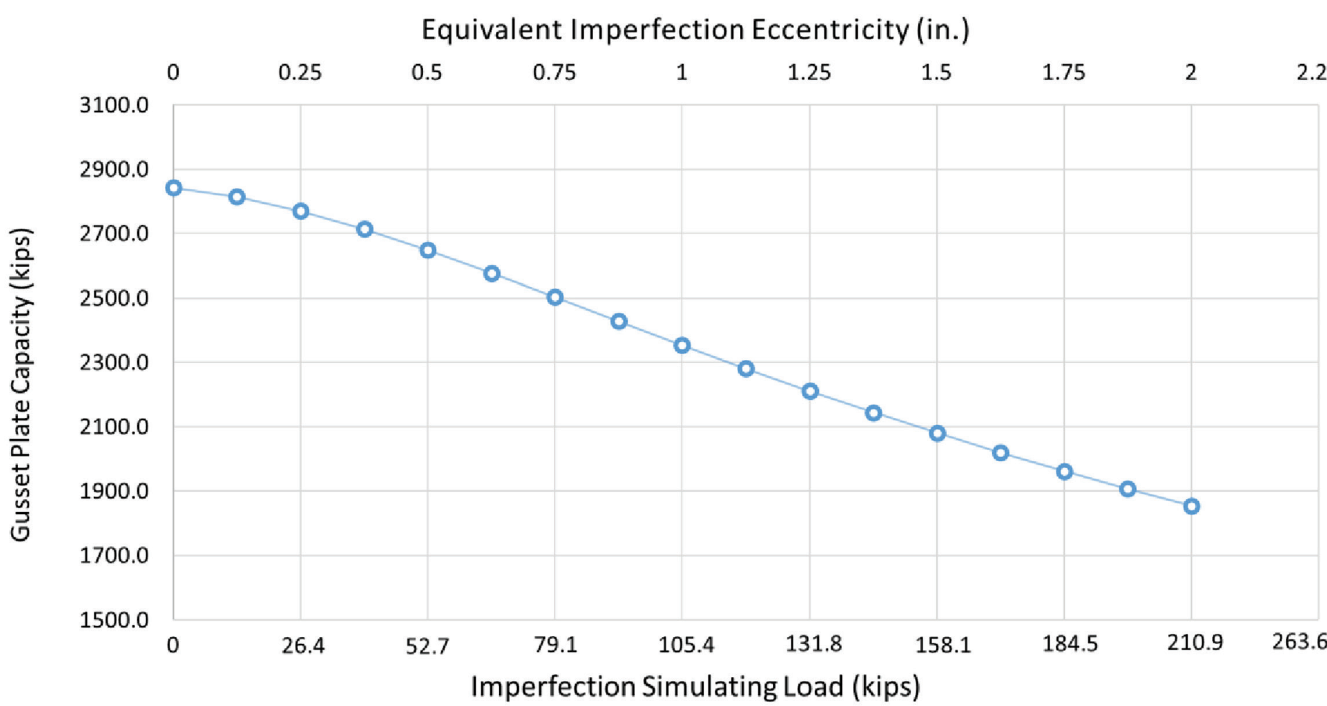


Fig. 6. Imperfection sensitivity curve for gusset plate connection at joint U9.

be confidently estimated with current design calculations. Accordingly, the proposed modeling deals with yielding and buckling failure modes of gusset plate connections that are associated with significant material, geometric, and boundary condition uncertainties. Incorporating initial imperfections to capture gusset plate connection buckling failure modes results in a significant complication in FEM modeling. In this proposed approach, the incorporation of initial imperfection is facilitated by applying an imperfection simulating force. Due to uncertainty about initial imperfection, this approach includes developing an imperfection sensitivity curve by investigating the rate of gusset plate capacity reduction for a reasonable range for the equivalent imperfection eccentricity. An imperfection sensitivity curve would provide a load rating engineer more confidence in estimating gusset plate capacity.

Reliability of the proposed refined analysis approach was investigated by numerically simulating a set of experimental tests on a gusset plate connection conducted under NCHRP Project 12-84. The validation analysis of this study demonstrated the reliability of the proposed approach in estimating gusset plate capacity at inelastic buckling failure as compared to actual tests. Calibration with test results indicated that a relatively larger out-of-plane imperfection eccentricity is required for this proposed approach as compared to the magnitude of imperfection criteria provided in the AASHTO MBE. This is due to not including imperfection locked-in stresses in this proposed approach. The proposed approach was utilized for a subject gusset plate case where the partial shear and truncated Whitmore method resulted in significantly different estimations for the gusset plate capacity and subsequent uncertainty about the need for gusset plate retrofit. The refined FEM analysis resulted in a capacity estimation comparable with the truncated Whitmore method estimation and confirmed the unnecessity of gusset plate retrofit. As indicated, engineers should be aware that the partial shear method may result in a significantly conservative load rating for gusset plates. For cases where the partial shear method load rating requires a retrofit, the truncated Whitmore method can be used as an alternative approach. Divergence of calculated load ratings between the two methods will roughly indicate whether using refined FEM analysis will be beneficial or not. Further research is needed to study if the truncated Whitmore method will always provide an estimate of the gusset plate capacity that is comparable to the refined FEM analysis method.

As demonstrated, gusset plate load rating practice will significantly benefit from utilizing more practical gusset plate refined FEM analysis approaches such as the one presented in this paper. Bridge owner agencies also will benefit by avoiding costs and efforts of unnecessary retrofits. Further investigations are required to understand the benefits and limitations of this proposed approach and develop guidelines for its application in load rating practice.

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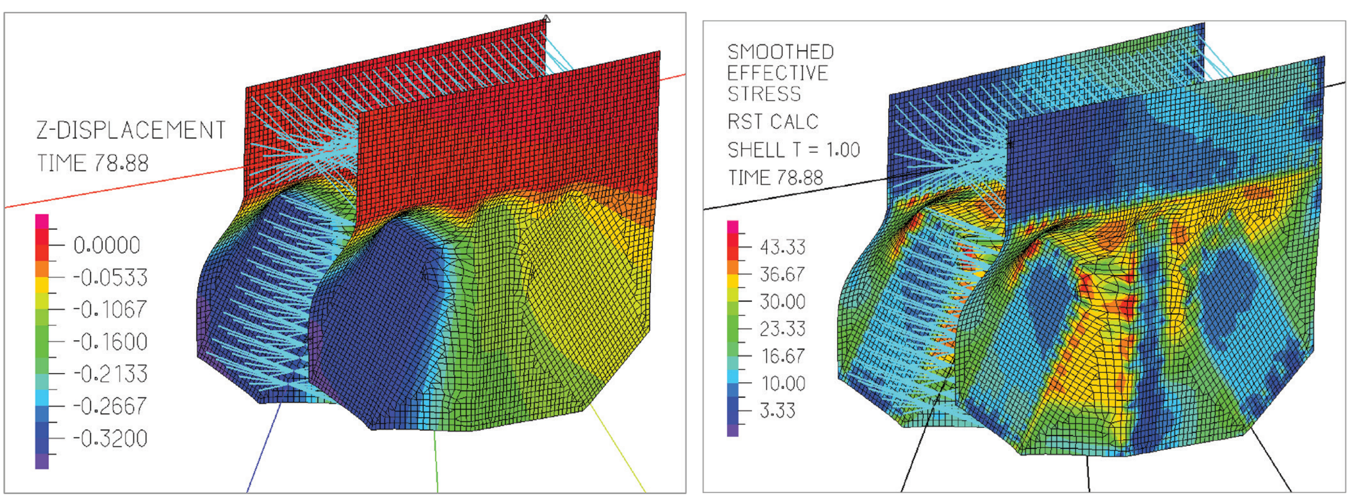

(a) Lateral deformation contour *(b) Von Mises stress contour*

Fig. 7. Gusset plate FEM nonlinear analysis results for gusset plate connection at joint U9 with 0.75-in. imperfection eccentricity.

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