Proceedings of the
Annual Stability Conference
Structural Stability Research Council
Pittsburgh, Pennsylvania, May 10-14, 2011

Buckling Behavior of Steel Truss with Torsional Bracing

Rangsan Wongjeeraphat[1], Todd A. Helwig[2]

Abstract
This paper outlines the results of a research investigation focused on the bracing behavior of steel trusses. The study included both laboratory tests and parametric finite element studies. The laboratory tests were conducted on a twin truss system with spans ranging from 48 feet to 72 feet. Both lateral and torsional bracing were considered in the laboratory tests, which provided valuable data for validating the models used in the parametric finite element analyses. The parameters that are being investigated in the FEA studies include both the truss geometry as well as the bracing details. There are a number of factors that have an impact on the effectiveness of torsional braces in truss systems. The bracing details that are being considered include the use of flexural members that restrain the top or bottom chord as well as the full depth cross frames. The buckling capacity of trusses with flexural member bracing are sensitive to the sizes of the truss verticals and diagonals due to cross sectional distortion. The connections between the web members and the chords are often idealized as simplified pinned connections. The truss with the simplified web has shown to provide a conservative estimate of the buckling capacity compared to the actual connection stiffness. The simplified web model was used to produce relatively simple expressions for the stiffness and strength requirements of the torsional braces that have been developed.

1. Introduction
Trusses can be complex systems in terms of the buckling behavior due to several unknown factors including the effects of the connection stiffness, orientation and alignment of the truss web members, and also the size and shape of the truss elements. The general buckling behavior of trusses has not been well studied in past investigations. Past investigations have focused on the bracing behavior of pony trusses, which generally consist of through trusses with no out-of-plane members framing into the top chord. The bracing of pony trusses are usually achieved by the floor beams that frame into the bottom chord at the joint locations. While there have been some studies on the bracing behavior of pony trusses, these studies are still relatively limited with regards to the general stiffness and strength requirements for the bracing. Therefore, there is still lack of information on the lateral and torsional bracing behavior for trusses.

[1] Graduate Research Assistant, University of Texas at Austin, <rangsan@mail.utexas.edu>
[2] Associate Professor, University of Texas at Austin, <thelwig@mail.utexas.edu>

The buckling capacity of structural systems is sensitive to several factors including the bracing details and layout. Many of the factors affecting trusses are similar to those affecting beam bracing that have been well studied. For bracing, the total stiffness is one of the factors compiled from a combination of several stiffness components. Those components include the brace stiffness, in-plane girder stiffness and the cross section stiffness. The total stiffness can be calculated by using the relationship for springs in series (Yura, 2001) as shown in the following expression:

$$\frac{1}{\beta_{tot}} = \frac{1}{\beta_{br}} + \frac{1}{\beta_g} + \frac{1}{\beta_{sec}} \tag{1}$$

Where β_{tot} is total brace stiffness, β_{br} is brace stiffness, β_g is in-plane brace stiffness and β_{sec} is cross section stiffness. In addition, connection flexibility, β_{con}, also has a significant impact on the bracing behavior and can be added to Eq. 1 (Quadrato, 2010).

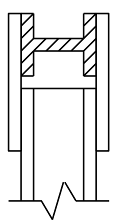
To effectively utilize the bracing in the structural system the stiffness of each component needs to be carefully considered since the total stiffness given by (1) will be smaller than the lowest stiffness component. The details of the truss cross section can be complicated due to the connection details and stiffening of portions of the cross section may be necessary to control local distortion. For beam bracing systems, the effects of web flexibility can be obtained using the equation similar to (1) suggested by Yura (2001), to combine the stiffness from each component throughout the depth of the beam. The details of the expressions related to cross sectional distortion in beams are summarized and shown in SSRC (2010).

As noted above, the effects of cross-sectional distortion in beams can be controlled by using web stiffeners to control the web flexibility. Torsional braces of beams usually consist of cross frames or diaphragms and cross-sectional distortion primarily occurs in regions of the web above or below the brace. Generally the portion of the web within the depth of a diaphragm brace is very stiff and does not distort. For cross frames, distortion of the region of the web within the depth of the brace does not affect the behavior. Distortion can also occur in columns if lateral braces frame into the web, or if torsional braces are used to improve the torsional buckling capacity. Web stiffeners also are effective at controlling the cross sectional flexibility for torsional bracing of column systems (Helwig and Yura, 1999).

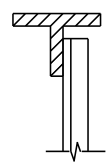
In trusses, connection flexibility can significantly affect the stiffness of the cross section and generally results in a reduction in the buckling capacity of truss system. Four frequently used types of truss chord-web connections were summarized by DeBlauw (2007) and are shown in Figure 1. The truss configurations include 1) Wide flanges for the chords and web members, 2) Tee sections for the chords and angles for the webs, 3) Angles for the webs and wide flange for the chords, and 4) Structural tubes for the chords and web members. The connections for system type (1) usually provide the highest chord to web rigidity which helps in improving the cross section stiffness and the truss buckling capacity. The connections for system type (2) and (3) are more flexible than type (1) but can still provide some stiffness to the system. System type (4) is often used for the aesthetic purposes. This paper mainly focuses on the case of the truss with the type (1) system configuration. Both laboratory tests and the FEA analyses were carried out. The parametric studies that were carried out actually provide meaningful conclusions for all of the different truss systems since the connection flexibility was varied from relatively flexible to relatively stiff.

2. Laboratory Test Setup

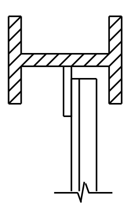
Two 72-ft span trusses with the type (1) system configuration that has W4x13 wide flange sections for the chords and webs were fabricated and tested. Torsional bracing was provided by two different size rectangular tube sections (HSS 3"x2½"x¼" and 5"x2½"x¼") with the end connection plates. The braces were positioned at either top or bottom chord with the number of bracing ranging from one to three. The loads were applied by hydraulic actuators that were mounted in gravity load simulators (GLSs) to insure the load remained vertical all the time. The GLS's connected to load beams that transferred the force to the two trusses. Additional details of the test setup are provided in Wongjeeraphat and Helwig (2010). Lateral stiffness tests were conducted on the truss system to provide validation data for the FEA models. The lateral stiffness tests consisted of applying lateral force at the third points on one chord to measure the deformations of the two chords. Tests were conducted with the unloaded chord unrestrained and restrained to vary the torsional deformations in the truss. Buckling tests were also used in the FEA model verification. The buckling tests were conducted with no bracing and also with torsional braces attached to the top chord at 20, 36 and 52 feet.



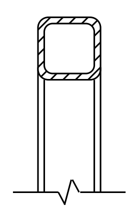
1) Wide flange to wide flange connection



2) Angle to tee connection



3) Double angles to wide flange connection



4) Structural tubes connection

Figure 1 Types of truss connection (DeBlauw, 2007)

3. FEA Model

The FEA model was developed by using the ANSYS® (2010) three dimensional FEA program. The elements used in the models were line elements, including the BEAM44 elements for the members with moment connections and the LINK8 elements for the members with only axial force. The connections at the chord to web regions were simulated by using a larger beam element that simulated the increase in stiffness from the gusset plates. Two types of the connection models were used in the analysis including connections with moment connections between the web members and the chords as well as simplified models with pinned connections for the web members. Figure 2 shows the enhanced detail for the truss model for the moment connections truss.

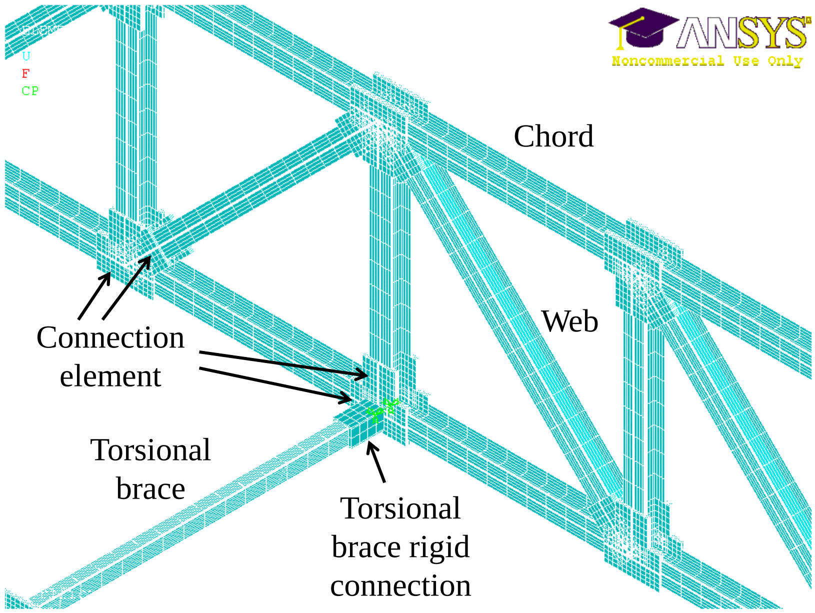


Figure 2 Enhanced FEA truss model with chord to web moment connection

A side by side comparison of the model to the actual structure at midspan of the truss with a torsional brace attached to is shown in Figure 3.

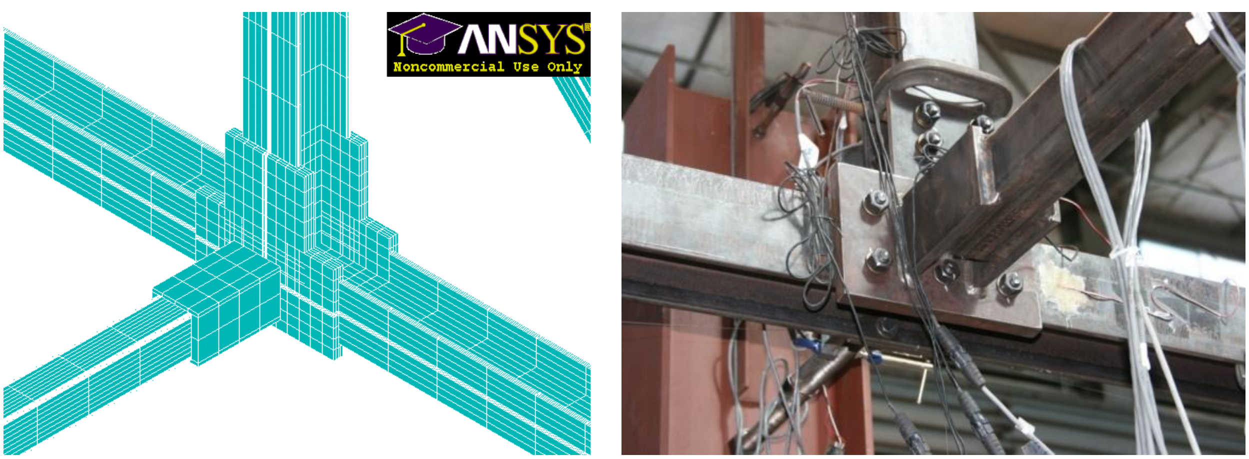


Figure 3 Comparison of the FEA model to actual truss at midspan bottom chord

For the simplified web connection model, all of the vertical and diagonal web elements were LINK8 elements, which only support axial forces. There were no connection elements connecting between the web and the chord in this case. The vertical web elements at both supports were the beam element with connection elements to stabilize the structural system. The simplified web model matches the idealized conditions that are often assumed for trusses and also provide reasonable truss systems with the connection type (2) configuration (tee chords with angle webs) and connection type (3) configuration (wide flange chords with angle webs), mentioned previously. In the parametric study, the areas of the web were set equal to the area of the web of elements used in the moment connection web model.

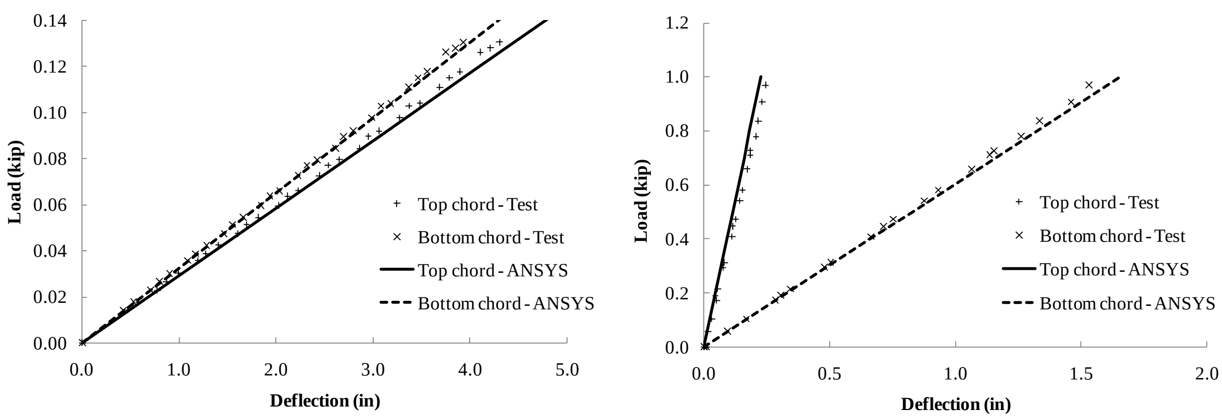
A disadvantage of using line elements to create the model was the inability to model individual gusset plates and connection chord elements that might experience significant distortion in the connections. The cross section distortion of individual gusset and connection chord elements can significantly affect the buckling capacity of the truss as reported in Wongjeeraphat and Helwig (2010).

4. FEA Model Verifications

The laboratory tests provided valuable data for validating the FEA models of the trusses so that parametric studies could be carried out. The data that was used for the verifications included the lateral stiffness tests, as well as the lateral and vertical deflections from the buckling tests. Experiments were conducted on the trusses with and without torsional bracing.

4.1 Truss Lateral Stiffness

The lateral stiffness tests provided valuable data for ensuring that the FEA models captured the lateral and torsional stiffness of the truss system. Experiments were conducted with lateral forces applied to one chord at the third points using a turnbuckle and a load cell to monitor the magnitude of the force. Tests were conducted with and without restraints on the unloaded chord. Figure 4 shows a comparison of the test and FEA data for the 72-ft span truss with bottom chord loading at the third points for the cases of (A) unrestrained and (B) with lateral restraint at top chord at the loaded locations. The results indicated that the FEA model has good agreement with the test results as the loads and lateral deflections were about the same for the trusses with and without lateral restraints.



A B

Figure 4 Midspan lateral deflections of 72-ft regular truss with bottom chord lateral loading
(A) Without lateral restraint (B) With lateral restraint at top chord

4.2 Truss System with and without Torsional Bracing

The vertical and lateral deflections from the buckling tests also provided data for verifying the FEA model. Tests were conducted with and without intermediate torsional braces. The results are shown in Figure 5 and Figure 6 for the respective cases of truss with and without torsional bracing. Two graphs are shown in each figure including the comparisons of the lateral deflection (A) on the left and the vertical deflection (B) on the right. The FEA model showed good agreement with the test results in vertical deflections in both cases with and without torsional bracing. For the lateral deflection, the FEA model showed slightly higher loads (about 10%) at the same lateral deflection levels than the test results. Although efforts were made to improve the accuracy, changes to the model that matched the buckling behavior and still achieved good lateral and vertical stiffness were not identified.

The difference between the FEA model and test results could be explained for the case of the truss without intermediate bracing that the connection elements of the FEA were slightly stiffer than the connections of the actual truss due to the simplified connection model had the web of the connection elements extended to the virtual connection nodes. In the actual truss, the web and the flange of the web discontinued about 3-4 inches before reaching the virtual node. This led to the slightly higher out-of-plane stiffness of the truss. For the cases of truss with torsional bracing system, in addition to the difference of the connection model, the truss model was also unable to capture the distortion at the gusset plates and chords at the brace connections. This led to the higher buckling capacity in the FEA model. According to the model verification, it was also founded that the larger the brace resulted in the larger the difference between the FEA and the test results which was consistent to the effect mentioned above. However, the model was deemed sufficiently accurate to demonstrate the parametrical response of truss buckling and bracing behavior.

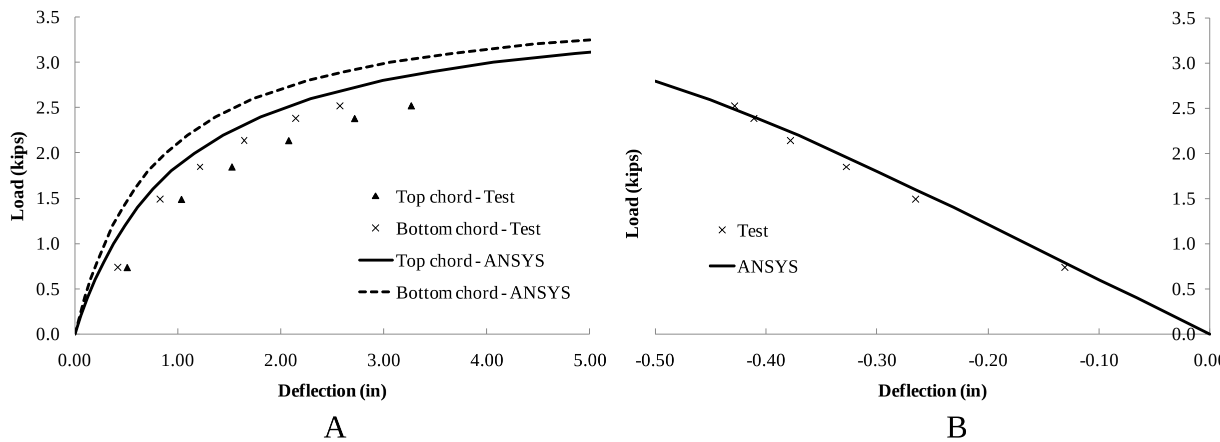


Figure 5 Buckling test of 72-ft span truss with top chord loading without intermediate bracing (A) Midspan lateral deflection (B) Midspan vertical deflection

5. Buckling Behavior of Truss Systems

5.1 Buckling Behavior of Truss with Different Types of Torsional Bracing

Previously reported laboratory test results (Wongjeeraphat and Helwig 2010) have demonstrated that cross section distortion of the chords and gusset plates at the points where the torsional braces framed into can significantly reduce the effectiveness of the bracing. This section focuses on the global distortion of the truss cross section by using the FEA results.

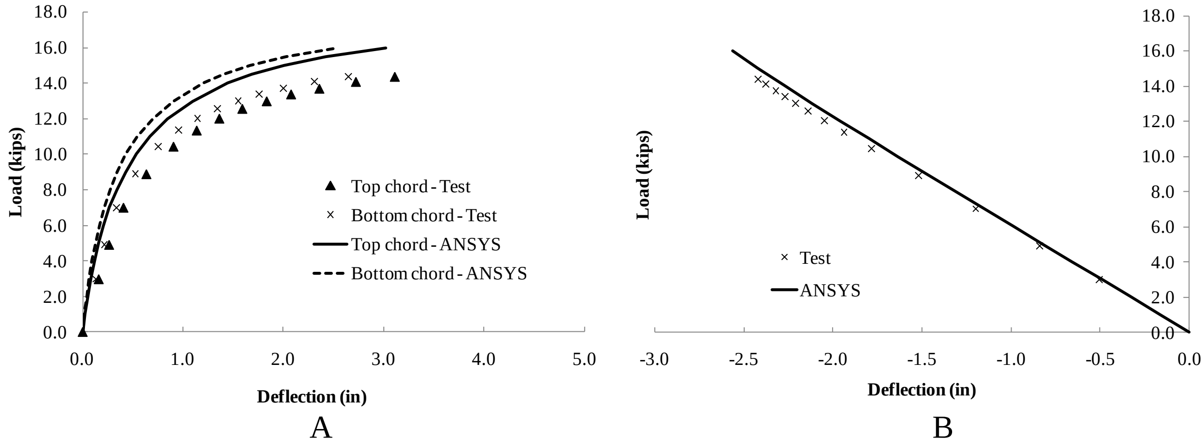


Figure 6 Buckling test of truss with 3 small torsional braces at top chord with top chord loading (A) Midspan lateral deflection (B) Midspan vertical deflection

The parametric study was conducted on trusses with a single torsional brace located at midspan. Although studies were conducted with multiple intermediate braces, the initial focus is on systems with a single intermediate brace to provide a starting point for the development of more general solutions. Some of the variables that were studied include the brace configurations, brace stiffness, and web details (moment connections with chords versus pinned connections). A variety of sizes were used for the top chord and the web members to vary the relative stiffness of the chord to the webs. The torsional braces were modeled as both full depth cross frames and beam elements that framed into one of the chords. Since full depth cross frames are not generally sensitive to cross sectional, the cases with beam elements were modeled using a rigid web vertical to eliminate the cross sectional distortion. The rigid web was created by the using a very large material modulus of elasticity of the web element and all six degree of freedoms of the web element were connected to the chord element at both ends.

Figure 7 shows the results of the FEA analysis of the truss with W8x24 chord with W3x8 web. One note is the W3x8 section is not actually available in the market but was generated for the analysis purpose to create a relatively flexible web member compared to the chord size. The results showed that the truss with torsional bracing with rigid vertical web at the brace connection provided the highest buckling capacity and served as the upper bound limit. The reason the top chord with the rigid web stayed in the half-sine curve is likely due to the warping restraint provided by the rigid web element. The buckling capacity of the truss with full depth cross frame closely followed the truss with the rigid web and then buckled between the brace point and the buckling capacity remained constant throughout the stiffness range. These two types of brace connection provided one similar condition which was the cross section distortion was prevented at the braced point. In practice, the web at the braced point is not rigid; therefore, the increased capacity relative to the cross frame system is not practical.

The two curves labeled TC (top chord bracing) and BC (bottom chord bracing) in Figure 7 are for the flexural brace with moment connections (regular web) between the web members and the chords; however the same size web members were used throughout (no rigid verticals). Therefore these members demonstrate the impact of cross sectional distortion of the web members. Because the effects of cross sectional distortion, β_{sec}, cause β_{tot} from Eq. 1 to be significantly reduced compared to the full depth cross frame or the case with the rigid webs. As a result, for a given brace stiffness the buckling capacity is reduced compared to the systems without cross sectional distortion.

Between the top and bottom chord bracing cases, it seemed that the top chord bracing provided slightly better performance in improving the truss buckling capacity. The likely reason for the difference is demonstrated in Figure 8 that shows the difference in the web member configurations at midspan for top chord and bottom chord bracing. The top chord bracing (TC) have more web elements framed into at the brace location (one vertical and two diagonal members) compared to the bottom chord bracing (BC) which has only one vertical member. The additional elements at the top chord reduced the effects of cross sectional distortion compared to the bottom chord bracing.

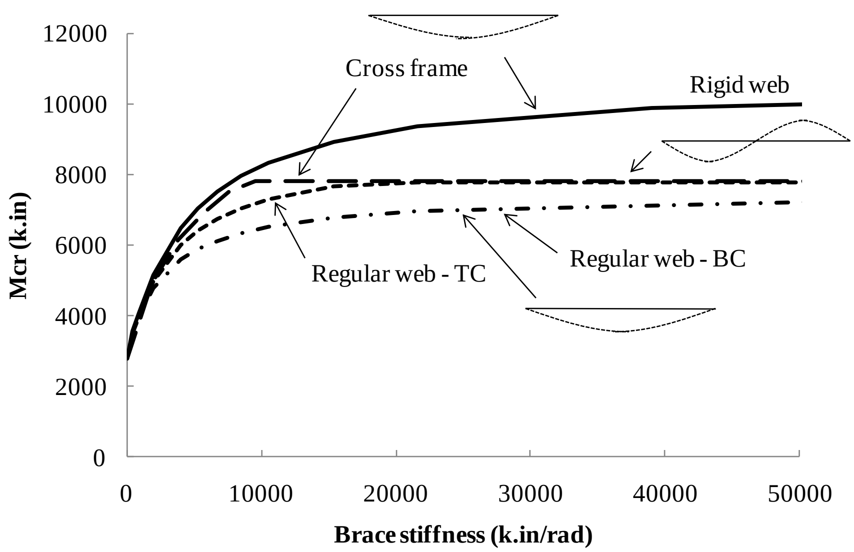


Figure 7 Buckling behavior of truss with different bracing conditions
(W8x24 chord W3x8 web truss)

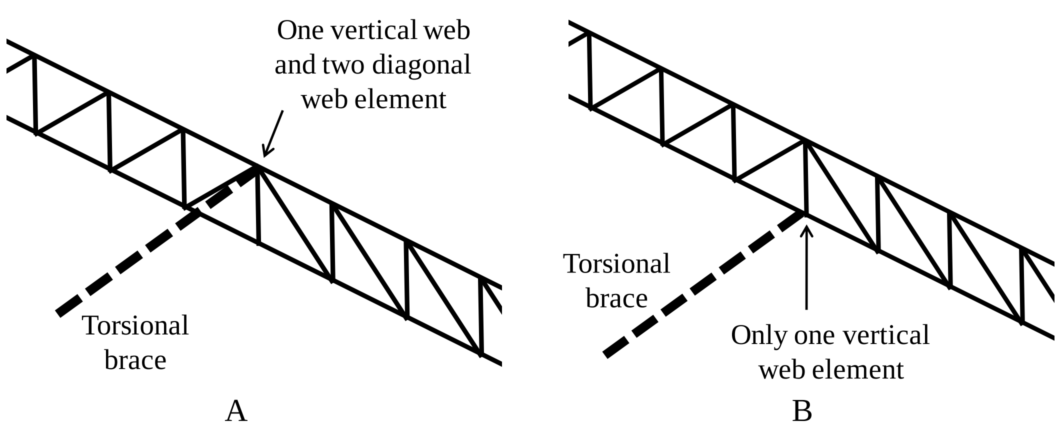


Figure 8 Midspan torsional brace connections (A) Top chord bracing (B) Bottom chord bracing

In the cases where the size of the web was large compared to the chord such as the case with the W12x50 chord and W12x26 web, the buckling behavior of the truss tended to be closer to the results from the truss with the rigid web as shown in Figure 9.

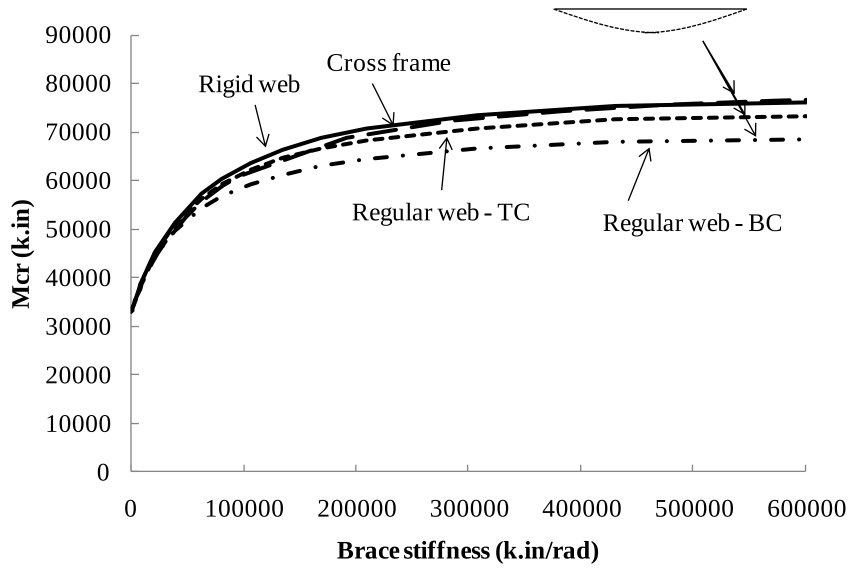


Figure 9 Buckling behavior of truss with different bracing conditions
(W12x50 chord W12x26 web truss)

5.2 Buckling Behavior of Regular and Simplified Webs Truss without Intermediate Bracing

Results presented in the last section showed that the buckling capacity of the truss with regular web varied due to the effect of cross section distortion. Accounting for the variation in the truss behavior as a function of the connection details is a complicated problem. Since expressions are not available that reflect the effects of both the connection and cross section distortions of truss with torsional bracing, the approach taken in this study was to simplify the problem by focusing on the behavior of trusses with idealized pinned connections for the web members. The simplified web model was created by using web members that only resisted axial forces. Comparisons between the behavior of web members that were pin ended (simplified web) versus members with joint restraint (regular web) were made to obtain a measure of the effect on the buckling capacity. Figure 10 shows the results of the trusses subjected to uniform moments. The loads were applied using a force couple applied to the two truss chords at the ends of the trusses. The chord and web sizes are graphed on the X-axis versus the buckling capacities on the Y-axis. The buckling capacities of regular truss increased with the increase of the sizes of the truss web elements for each chord size while the buckling capacities of the truss with simplified web were constant throughout the web range with the same chord sizes. For the truss with small web sizes, the buckling capacity of the truss with the simplified web was slightly less than the truss with regular web since the available restrain was relatively small. Increasing the web sizes resulted in the larger the difference in buckling capacity between the regular web truss and the simplified web truss as the buckling capacity of the truss with regular web increased. Similar results were observed for trusses with uniformly distributed loads.

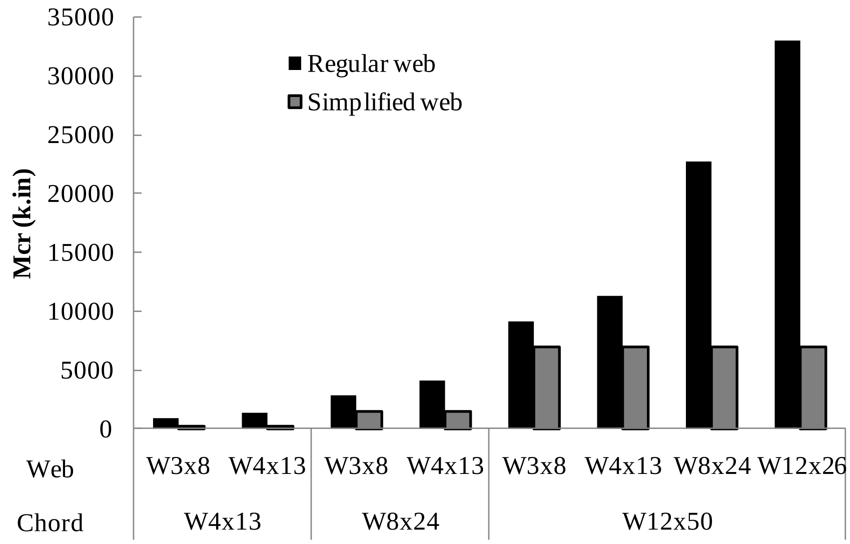


Figure 10 Buckling capacity of truss with uniform moment

The increase in the capacity of truss with regular web indicates that the moment connection of the web to chord connections can contribute significant to improving the buckling performance of the trusses. The buckling capacities were not affected by the area the webs in the simplified web truss since the buckling capacity remained constant with the change in web size. However, the sizes of the webs will likely contribute in the vertical stiffness. This indicated that neglecting the bending moment connection of the web in chord to web connections would be conservative in determining the buckling capacity of the truss. The level of conservatism is dependent on the relative sizes of the chords and webs.

The results of the eigenvalue buckling analysis of simplified web truss were normalized and compared with the ratio of the out-of-plane moment of inertia of the web to chord and shown in Figure 11. The trusses had a depth of 6 feet and spans of 48 and 96 ft. were considered. To discern between different span lengths, filled in markers were used for the 96 ft. spans compared to markers made from intersecting lines for the 48 ft. spans. The ratio of out-of-plane moments of inertia of the web to the compressive chord is graphed on the X-axis while the ratio of the critical moment of the simplified web truss to the regular web truss is graphed on the Y-axis. As expected, increasing the stiffness of the web resulted in larger reductions in more conservatism (smaller M_{cr_sim}/M_{cr_reg}). The reduction was more significant for trusses with larger span to depth ratios.

5.3 Buckling Behavior of Regular and Simplified Webs Trusses with Single Full Depth Cross Frame at Midspan

Both regular and simplified web truss models were used in the study to determine the buckling capacities of the trusses with a single cross frame at midspan. A graph similar to that shown in Figure 10 is shown in Figure 12. The chord and web sizes are indicated on the X-axis while the buckling capacities are indicated on the Y-axis. The behavior of truss with regular and simplified webs with single cross frame at midspan were similar to the truss without intermediate bracing in that the use of the simplified web was more conservative for increases in the size of the web members. Although the FEA results show that the solutions are more conservative using the simplified web, such an approach is consistent with the idealized models that are used for trusses. In addition such a solution will likely result in more simple solutions for predicting the buckling capacity of the trusses as well as the bracing requirements. Work is currently underway using results from the FEA parametric studies to develop buckling solutions and the stiffness and strength requirements for torsional and lateral bracing of trusses. Solutions are being considered for both the regular and simplified truss models.

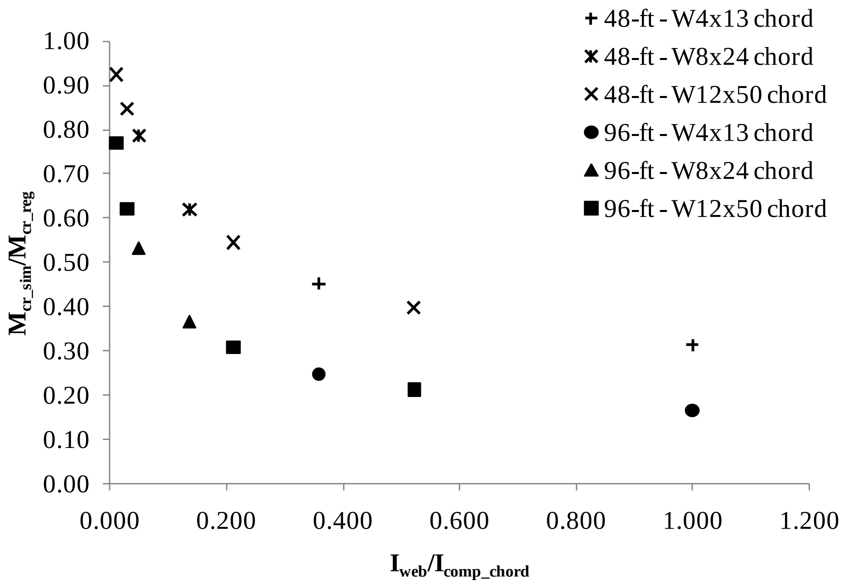


Figure 11 Reduction in buckling capacity of truss with simplified web

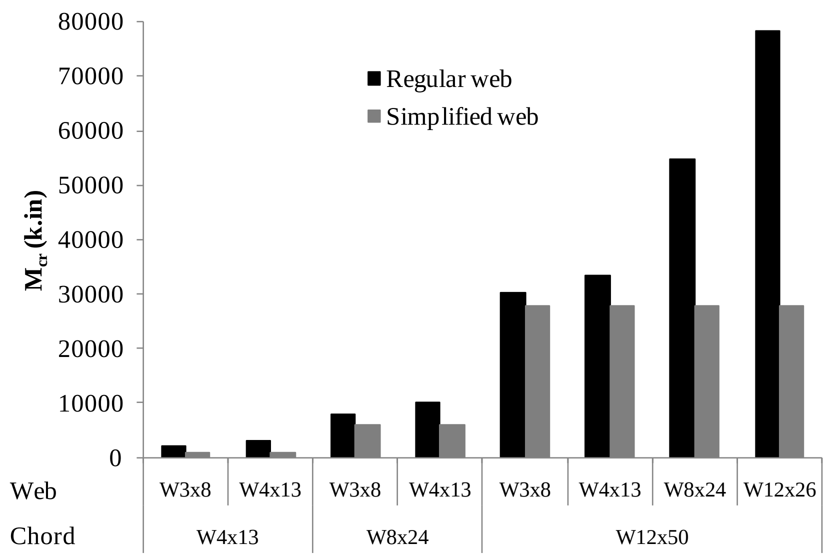


Figure 12 Buckling capacity of truss with single cross frame at midspan with uniform moment

6. Conclusions

Predicting the buckling behavior of trusses is complicated by the numerous factors that affect both the mode and capacity of the truss. Cross sectional distortion has a significant impact on the torsional bracing behavior of the truss. While full depth cross frames are not significantly impacted by cross sectional distortion, in some instances these braces are not practical and bracing is instead provided by flexural members that frame into one of the chords. In these instances, cross sectional distortion must be considered. FEA solutions demonstrated that the distortion was reduced at nodes where multiple web members framed into the joint. Additionally, although trusses are often idealized as pinned at the joints, in reality the web member will often transfer moments to the chords. The restraining moments can lead to significant increases in the buckling capacity of the truss relative to the simplified web models. The simplified web model is conservative with respect to the actual web conditions, with the level of conservatism sensitive to the relative stiffness of the web compared to the chords.

References

ANSYS Inc. "Elements Reference" *Release 12.0 Documentation for ANSYS.* Canonsburg, PA, 2010.

DeBlauw, R., Master report, The University of Texas at Austin, Austin, TX, 2007.

Helwig, T.A.; and Yura, J.A., “Torsional Bracing of Columns,” *ASCE Journal of Structural Engineering*, Vol. 125, No. 5, pp. 547-555, May, 1999.

Quadrato, C.E., *Stability of Skewed I-Shaped Girder Bridges Using Bent Plate Connections*, Ph.D. Dissertation, The University of Texas at Austin, 286 pp, May 2010.

Structural Stability Research Council (SSRC), *Guide to Stability Design Criteria for Metal Structures*, 5th ed., Ziemian R.D. ed., John Wiley & Son, New York, NY, USA, 2010.

Wongjeeraphat, R., and Helwig, T.A. (2010), "Bracing Behavior for Steel Trusses", *Proc. of the Annual Stability Conference*, Structural Stability Research Council (SSRC), Orlando, FL, pp.387-406, May, 2010.

Yura, J.A., “Fundamentals of Beam Bracing”, *Engineering Journal*, American Institute of Steel Construction, 1st Quarter, pp. 11-26, 2001.