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Buckling Strength of Axially Loaded Cold Formed Built-Up I-Sections

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ABSTRACT

This paper presents a numerical procedure using finite element analysis for the calculation of axial strength of cold formed steel built-up I-sections composed of two back-to-back channels. The material nonlinearity of the flat and corner portions of the section were incorporated in the model. The effects of initial local and overall geometric imperfections as well as the membrane residual stresses have been taken into consideration in the finite element model. The results of the nonlinear finite element analysis were compared with the available experimental results, and with the calculated theoretical buckling capacities based on the AISI design provisions. A parametric study was carried out using the developed finite element model to study the effects of member and cross-section geometries and imperfection values on the strength of cold-formed steel built-up I-columns. The column strengths predicted from the parametric study were compared with the design strengths calculated using the American Specification. The results of the parametric study showed that the design provisions specified in the American Specifications are generally conservative for long and medium length columns, but may give un-conservative estimates for some of the short columns.

1. Introduction

Cold-formed steel members are widely used in building construction, such as wall studs, floor joists, truss members and other structural applications. Cold-formed steel sections are usually formed in single C, Z, and hat sections. The cross sections of these members can be also formed by connecting two or more sections together, for examples, an I-section formed by connecting two channel sections back-to-back, and a box section formed by connecting two channel sections in the flanges.

Axially loaded cold formed members may fail by global, local and/or distortional buckling due to their high plate width-to-thickness ratio. Flexural buckling tends to occur in slender members due to global geometric imperfections, Fig. 1. As the slenderness ratio becomes smaller, geometric local imperfections cause the failure to become more localized as in a thin plate subjected to an in-plane membrane stress, resulting in a transition from global buckling to local and /or distortional buckling, Fig. 2.

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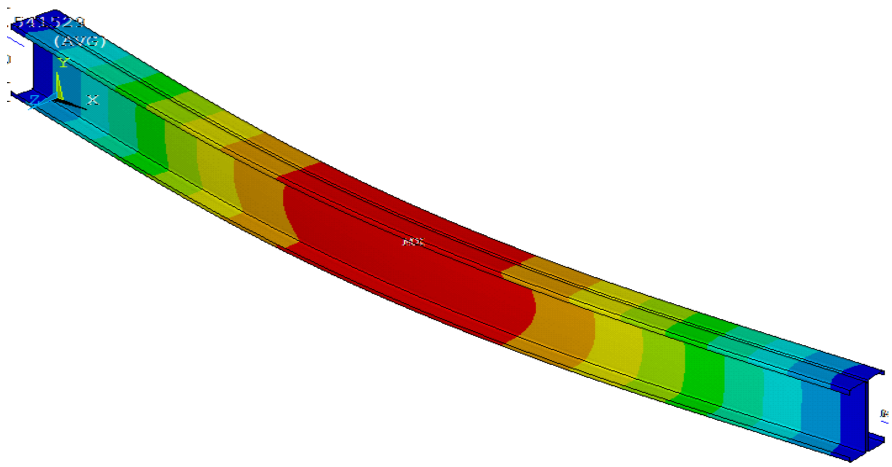

Figure 2 First Flexural Buckling Mode

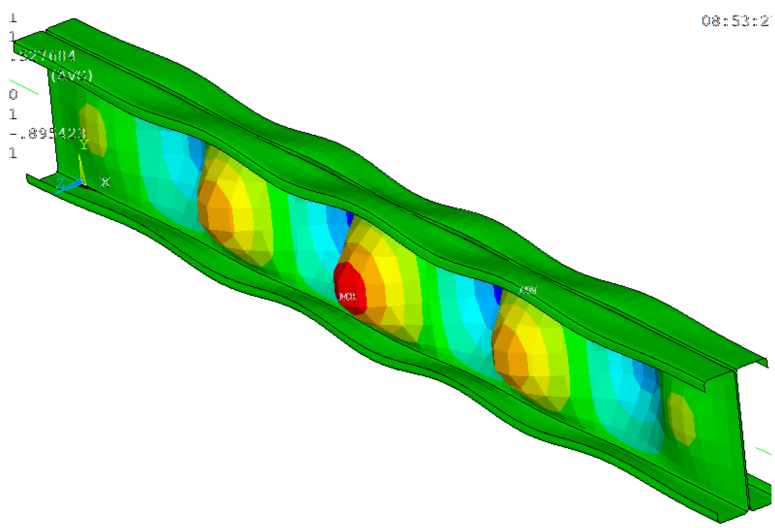

Figure 3 First Local Buckling Mode

The design provisions for cold-formed sections under pure compression are stated in Sections C4 of the 2007 North American Specification for the Design of Cold-Formed Steel Structural Members ; AISI (2007). The nominal axial strength P_n is taken as follows:

$$P_n = \text{ smaller of } (P_{ne} , P_{nd}) \qquad (1)$$

Where,

P_{ne} = nominal strength for yielding, flexural, flexural-torsional, and torsional buckling according to section C4.1,

P_{nd} = nominal distortional buckling strength according to section C4.2.

Note that flexural-torsional buckling does not occur in most cases of doubly-symmetric built-up members with a sufficient number of intermediate, symmetrical fasteners.

Similarly, distortional buckling may be present only for sections having exceptionally wide flanges as studied by Piyawat (2011).

Alternatively, Appendix 1 of the AISI Specification presents a different design procedure based on the direct strength method (DSM), Schafer (2008). According to this method the nominal axial strength P_n is taken as follows:

$$P_n = \text{minimum of } (P_{ne},\ P_{nl},\ P_{nd}) \tag{2}$$

Where,
P_{ne} = nominal strength for yielding, flexural, flexural-torsional, and torsional buckling according to section 1.2.1.1,
P_{nl} = nominal local buckling strength according to section 1.2.1.2.
P_{nd} = nominal distortional buckling strength according to section 1.2.1.3.

The direct strength method relies on the calculation of elastic buckling stresses from a rational elastic buckling analysis buckling analysis such as finite strip method, see Schafer (2010) , or using finite element methods. The DSM method is highly favorable than the traditional effective method because it does not require the calculation of the effective width for cross-section. In addition, the DSM uses realistic estimates of the local and global buckling loads based on consideration of the entire cross section rather than considering individual elements. However, the method has not been calibrated for built-up I-shaped members.

Built up sections formed by connecting two components are subjected to shear-induced relative deformations between the combined components. Furthermore, built-up members may buckle globally either as one single component or as one combined section. For these reasons, additional specific design provisions for built-up compression members are stated in Section D1.2 of the AISI specifications.

If the buckling mode produces shear forces in the connectors between the members, the slenderness ration (KL/r) used to calculate the elastic buckling stress should be replaced with a modified value $(KL/r)_m$ calculated from:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} \tag{3}$$

where $(KL/r)_m$ is the overall modified slenderness ratio of the entire section about the built-up member axis; $(KL/r)_o$ is the overall (unmodified) slenderness ratio of the entire section about the built-up member axis; a is the longitudinal spacing between intermediate fastener or spot welds connecting the two components; and r_i is the minimum radius of gyration of the full unreduced cross-section of the individual component. Furthermore, Section D1.2 of AISI-2007 provides the requirements for the fastener or stitch weld strength and spacing. First, the ratio (a/r_i) is not to exceed $0.5(KL/r)_o$. Second, the member-end connectors or weld should have a certain length. Lastly, the intermediate fasteners or welds at any longitudinal member tie location should have a transmitting force of 0.025 of the nominal axial capacity in any direction.

The additional provisions stated in Section D1.2 are similar to those used in the AISC Specifications (2010). They are based on research conducted on hot rolled sections where the member axial strength is governed mainly by global buckling and rarely influenced by local buckling. Cold formed members, because of their high plate width-to-thickness ratios, may fail by local and/or distortional buckling at stress levels much lower than the corresponding global buckling stresses.

At the same time, limited test data are available on cold formed built up I-sections. Stone and LaBoube's (2005) tested some cold-formed, built-up I-sections constructed from steel studs. The members tested were cold formed C-channels intermediately connected back to back with screws to model a typical, cold-formed, I-shaped wall stud. Additional work of cold-formed built-up C-channels was conducted by Brueggen and Ramseyer (2003), Whittle (2007) and Biggs (2008) on smaller c-channels in open and closed-sections with intermediate welded stitch attachments. Their research on the built-up stub columns concluded that the AISI design method is conservative for compact members but often un-conservative for members with slender elements. Brueggen and Ramseyer (2003) recommended that additional tests be performed to determine the effects of length and location (double-or single-sided), spacing, and number of weld attachments on the behavior of welded built-up members. Piyawat (2011) studied the axial capacity of cold formed built-up I-columns with exceptionally wide flanges where distortional buckling may govern the design.

The previous review of current design provisions and available test results shows that there is a need to investigate the appropriateness of the current design rules for cold formed built-up I-shaped members.

The main objective of this paper is to develop a numerical procedure that can be used to calculate the axial capacity of cold-formed built-up I-section using finite element method. The finite element program ANSYS (2010) was used in the analysis. The results obtained from the numerical analysis were first compared with some available test results. A parametric study was then performed to investigate the effect of cross-section geometry and geometric imperfections on the strength of these sections. The results obtained from the parametric study were compared with the design strengths calculated using the AISI provisions.

2. Numerical Analysis

The finite element method has proven to be a very successful tool for calculating the post buckling capacity of cold formed steel members. The geometrical and material non-linear behavior present in such a case requires two types of analyses. The first type of analysis is an eigenvalue analysis that estimates the buckling modes and buckling frequencies as the solution to an eigenvalue problem. In this problem the material behavior is assumed to be elastic and the member is assumed to have perfect geometry. The lowest buckling modes predicted from the eigenvalue analysis are subsequently used to model the geometric imperfections. The second type of analysis is a nonlinear load–displacement analysis of the real member under the action of applied loads in the presence of initial geometrical imperfections, residual stresses and material nonlinearity. The ultimate loads and failure modes are determined from this analysis when it reaches a limit point located on its equilibrium bath; the corresponding load parameter value and deformed configuration provide the member ultimate strength and failure mode, respectively. The finite element program ANSYS was used in the present study to model the cold-formed steel columns as described in the next section.

2.1 Finite Element Model

The member was modeled using the 4-node finite strain shell element, shell 181, built in ANSYS element library. This element accounts for six degrees of freedom per node and allows for stress stiffening, large deformation, as well as material non-linearity. It is well suited for linear, large rotation, and /or large strain nonlinear applications. In order to choose the finite element mesh that provides accurate results with minimum computational time, convergence studies were conducted. It is found that the mesh size of 25 mm×10 mm (length by width) provides adequate accuracy and minimum computational time in modeling the flat portions, while a finer mesh was used at the corners as shown in Fig. 3.

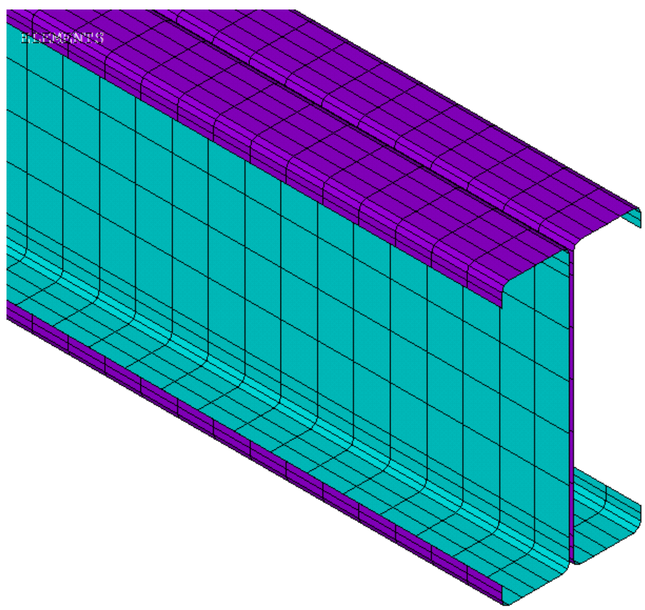

Figure 3 Finite Element Model

The material behavior provided by ANSYS allows for a multi-linear stress–strain curve to be used. The first part of the multi-linear curve represents the elastic part up to the proportional limit stress with a known Young's modulus, taken equal to 203000 MPa in the presents study, and Poisson's ratio, taken as 0.3. The slope of the plastic part of the multi-linear curve was assumed at 5 %. Von-Mises yield criteria with isotropic hardening was used.

2.2 Modeling Geometric Imperfections

Cold formed members always contain initial geometric imperfection during their fabrication either by cold rolling or press braking. Geometric imperfections may be classified into two categories; global imperfections along the member length and local imperfections of the cross section. Typically L/1000 is used as the magnitude and a global buckling mode shape, see Fig. 1, is used as the distribution shape to approximate global imperfections, where L = member length. The common approach in considering local cross-sectional imperfections is to use a portion of the thickness of the member as the magnitude and the local and distortional buckling mode shapes, see Fig. 2, as the distribution of these imperfections (Schafer and Pekoz (1998)). Examples of available imperfection measurements categorized into cross-sectional (local and distortional) and global (Bow or weak axis flexure, Camber or strong axis flexure and twist) are given in Zeinoddini and Schafer (2012) as shown in Table 1.

Table 1: Statistical summary of available data on imperfections (Zeinoddini and Schafer (2012))

	Local L (δ_0/t)	Distortional D (δ_0/t)	Bow G_1 (L/δ_0)	Camber G_2 (L/δ_0)	Twist G_3 (deg/m)
mean	0.47	1.03	2242	3477	0.36
st.dev.	0.62	0.97	3054	5643	0.23
25 %ile *	0.17	0.43	4755	6295	0.20
50 %ile	0.31	0.75	2909	4010	0.30
75 %ile	0.54	1.14	1659	2887	0.49
95 %ile	1.02	3.06	845	1472	0.85
99 %ile	3.87	4.46	753	1215	0.95

(*) %ile values are the probabilities that imperfection will be less than the table value

Different mode shape imperfections need to be combined in a proper way. In the traditional modal approach, imperfections are modeled as a linear combination of the first buckling modes using a suitable magnitude for each mode (can be chosen from Table 1, or traditionally chosen as 1/1000 of length or 10% of thickness). In this paper, 1/1000 of length and 10 % of thickness were used to compare test results with AISI results. Values at 25 %ile and 75 %ile, as recommended by Zeinoddini and Schafer (2012), were used in a sensitivity analysis in the parametric study.

2.3 Modeling of Residual Stresses

Cold formed members always contain residual stresses during their manufacturing process. Coiling, uncoiling, cold bending to shape, and straightening of the formed member lead to a complicated set of initial stresses and strains in the section. Residual stresses may be idealized as a summation of two types, flexural and membrane (Schafer (1998)). Some statistical results for membrane residual stresses are given in Schafer (1998). The data shows that membrane residual stresses exist primarily in corner regions and their values may reach about 8 % F_y at corners and about 4% F_y for flat parts. Opposing this effect, the yield stress F_y is increased at corner regions by about 15 % F_y due to cold work of forming as shown by Abdel-Rahman, (1997). On the other hand, measured flexural residual stresses show a large degree of variation. Statistics for flexural residual stress are reported in Schafer (1998) and Meon (2008). Considering these stresses in the finite element model complicates the analysis considerably as it requires defining the through thickness stresses for each layer. Furthermore, Meon (2008) suggested using kinematic hardening rule instead of isotropic hardening. As the main interest in this paper is to find the ultimate axial load capacity, the present analysis neglects the effect of flexural residual stresses. This assumption would not be correct when considering the deformation behavior and stress distribution across the section. In the present model the effect of membrane residual stresses of the stated representative values on the axial buckling capacity of cold formed built-up members was studied by using different values of the yield strength for corner regions and for flat regions. It was found that the effect on axial buckling strength was less than 1 %.

2.4 Boundary conditions and load application

Both member ends of the columns were modeled as hinged ends allowing bending rotation but prevented from translation and twist except for the displacement at the loaded end in the direction of the applied load. The nodes other than the two ends were free to translate and rotate in any directions. The displacements of the two components forming the cross section were coupled at the locations of the connecting screws. The load was applied as an axial concentrated load at the section centeroid at the loaded end.

2.5 Solution Methods

Elastic buckling analysis was carried out using Lanczos solver as recommended by the software. The nonlinear analysis was conducted using Newton-Raphson method with automatic load sub steps determined by the software.

3. Comparison with Test Results

The finite element model developed in the previous section was used to calculate the axial load capacity of 32 cold-formed columns tested by Stone and Laboube (2005). The values of geometrical global and local imperfection were taken as L/1000 and 0.1*t, respectively. The comparison of the ultimate loads (P_{test}/P_y and P_{FE}/P_y) obtained experimentally and numerically is shown in Table 2. The table contains also the results obtained by applying the current AISI provisions of sections C4.1 and D 1.2. Deviations between test results, finite element results and AISI results are shown in Table 3. The results are also plotted in Fig. 4 to to Fig.7 for the four cross sections tested.

Investigation of these results shows that considerable discrepancies exist, but the following conclusions may be stated:

1- Finite element results deviate from the test results by an average of 10.6 % on the conservative side, while the results based on the AISI provisions deviate by an average of 27.1 % on the conservative side.
2- The deviations increase for sections with large plate depth to thickness ratio and decreases for members with small slenderness ratio.

Table 2: Comparison with Test Results and AISI Results

Depth (mm)	Thickness mm	Screw spacing, a (mm)	P_{test}/P_y	P_{FE}/P_y	P_{AISI}/P_y
152.4	1.372	304.8	0.317	0.264	0.219
152.4	1.372	609.6	0.326	0.252	0.204
152.4	1.372	609.6	0.303	0.252	0.204
152.4	1.372	609.6	0.319	0.252	0.204
152.4	1.372	762.0	0.291	0.252	0.195
152.4	1.372	762.0	0.308	0.252	0.195
152.4	1.372	762.0	0.341	0.252	0.195
152.4	1.372	914.4	0.289	0.248	0.184
152.4	1.372	914.4	0.253	0.248	0.184
152.4	1.372	1016.0	0.312	0.247	0.177
152.4	1.372	1066.8	0.312	0.247	0.173
152.4	1.372	1066.8	0.314	0.247	0.173
92..1	1.155	304.8	0.444	0.447	0.421
92.1	1.155	304.8	0.538	0.447	0.421
92.1	1.155	609.6	0.412	0.432	0.392
92.1	1.155	609.6	0.411	0.432	0.392
92.1	1.155	914.4	0.382	0.42	0.348
92.1	1.155	914.4	0.450	0.42	0.348
92.1	0.880	304.8	0.657	0.549	0.497
92.1	0.880	304.8	0.579	0.549	0.497
92.1	0.880	304.8	0.421	0.549	0.497
92.1	0.880	304.8	0.521	0.549	0.497
92.1	0.880	609.6	0.564	0.528	0.474
92.1	0.880	609.6	0.650	0.528	0.474
92.1	0.880	914.4	0.564	0.523	0.437
92.1	0.880	914.4	0.631	0.523	0.437
152.4	0.841	304.8	0.310	0.327	0.256
152.4	0.841	304.8	0.362	0.327	0.256
152.4	0.841	609.6	0.415	0.319	0.239
152.4	0.841	609.6	0.351	0.319	0.239
152.4	0.841	914.4	0.303	0.304	0.215
152.4	0.841	914.4	0.335	0.304	0.215

Table 3: Deviations from Test and AISI Results

Depth (mm)	Thickness mm	Screw spacing, a (mm)	% Deviation Test vs FE	% Deviation Test vs AISI
152.4	1.372	304.8	16.7	30.8
152.4	1.372	609.6	22.8	37.3
152.4	1.372	609.6	16.8	32.5
152.4	1.372	609.6	21.1	36.0
152.4	1.372	762.0	13.5	33.2
152.4	1.372	762.0	18.2	36.8
152.4	1.372	762.0	26.1	42.9
152.4	1.372	914.4	14.4	36.3
152.4	1.372	914.4	2.2	27.2
152.4	1.372	1016.0	20.9	43.4
152.4	1.372	1066.8	20.9	44.6
152.4	1.372	1066.8	21.3	44.9
92..1	1.155	304.8	-0.6	5.4
92.1	1.155	304.8	16.9	21.8
92.1	1.155	609.6	-4.9	4.9
92.1	1.155	609.6	-5.1	4.7
92.1	1.155	914.4	-9.9	9.0
92.1	1.155	914.4	6.6	22.7
92.1	0.880	304.8	16.5	24.4
92.1	0.880	304.8	5.1	14.1
92.1	0.880	304.8	-30.3	-18.0
92.1	0.880	304.8	-5.5	4.5
92.1	0.880	609.6	6.3	15.9
92.1	0.880	609.6	18.8	27.1
92.1	0.880	914.4	7.2	22.4
92.1	0.880	914.4	17.2	30.8
152.4	0.841	304.8	-5.6	17.3
152.4	0.841	304.8	9.7	29.3
152.4	0.841	609.6	23.2	42.5
152.4	0.841	609.6	9.1	32.1
152.4	0.841	914.4	-0.2	29.3
152.4	0.841	914.4	9.2	35.9

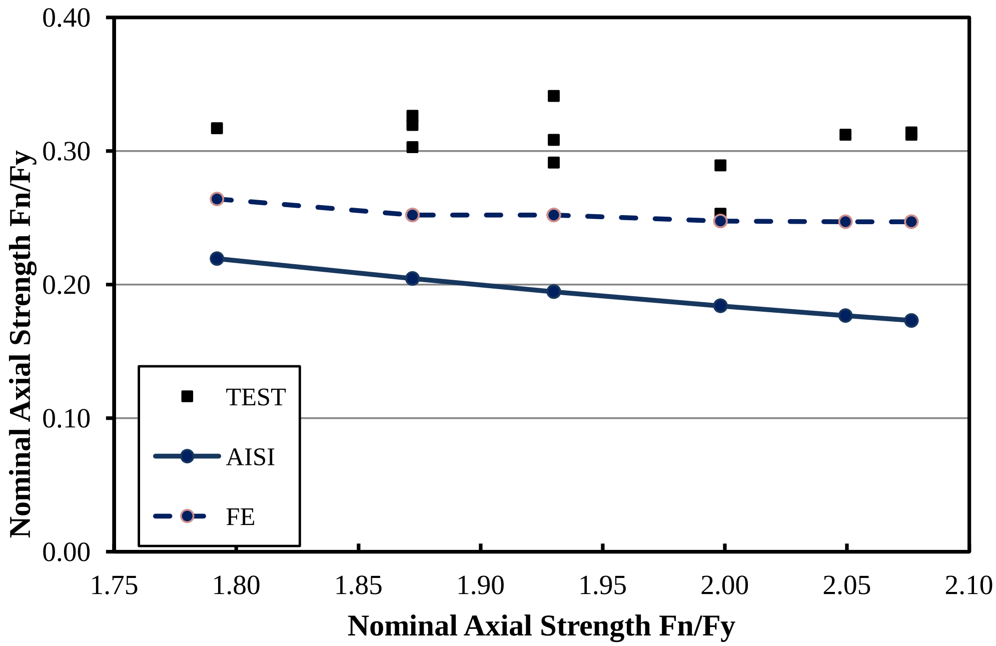

Figure 4 Comparison of Axial Strengths Results for Section152x1.372

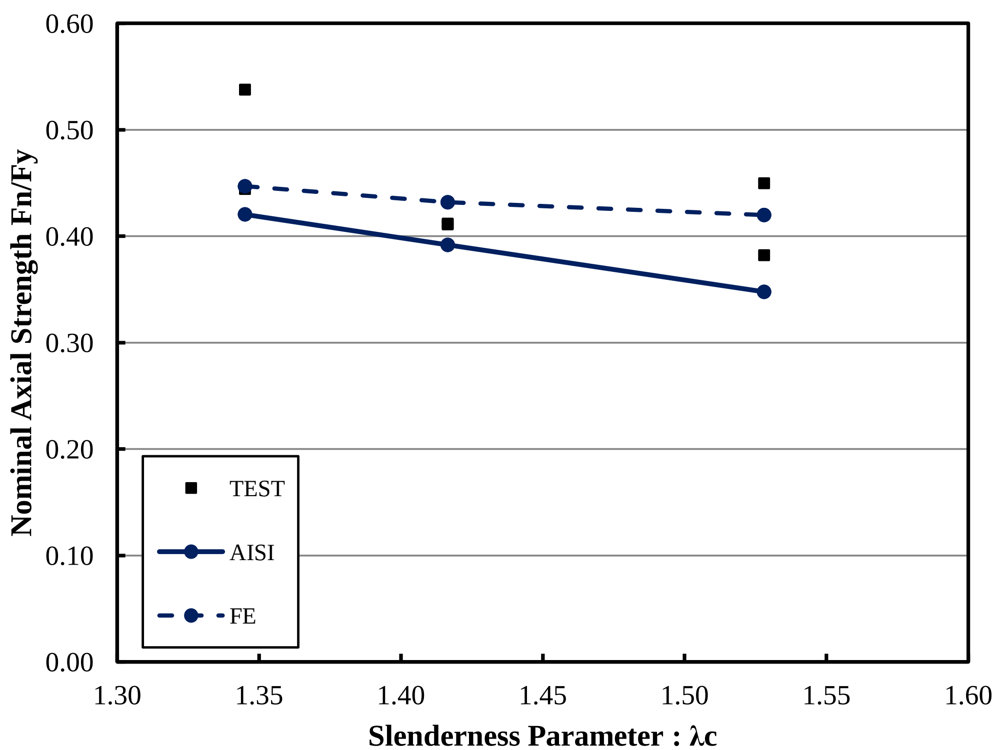

Figure 5 Comparison of Axial Strengths Results for Section 92x1.155

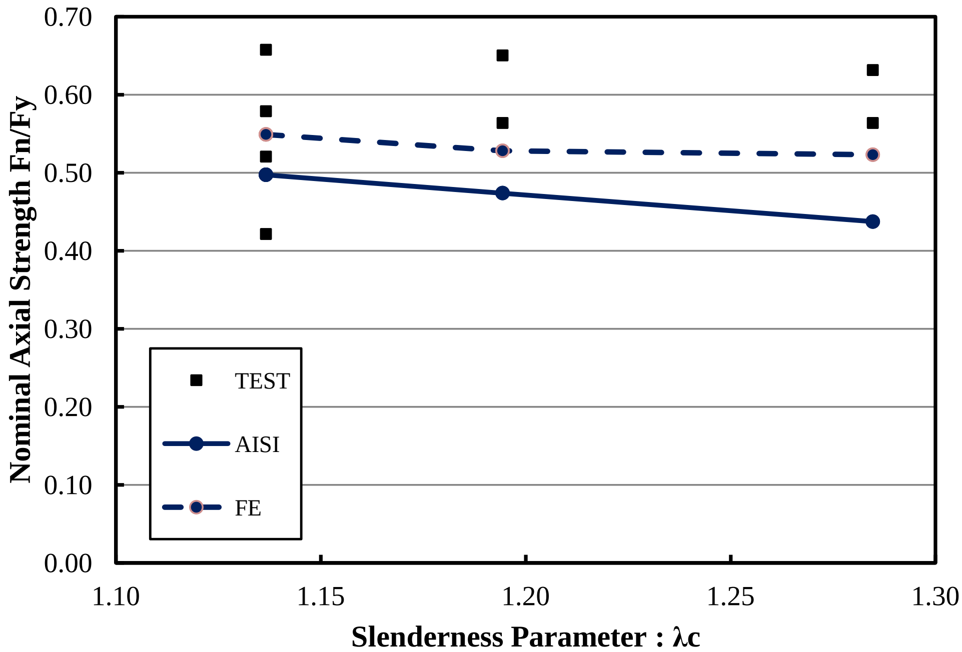

Figure 6   Comparison of Axial Strengths Results for Section 92x0.88

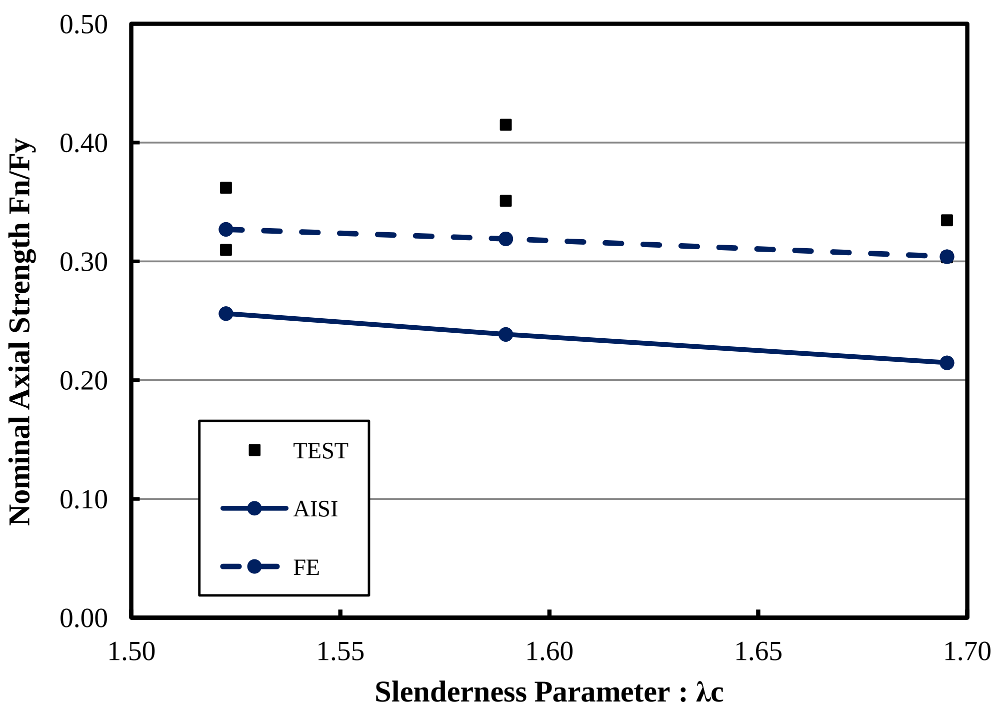

Figure 7 Comparison of Axial Strengths Results for Section 92x0.88

4. Parametric study

The comparison presented in the previous section was limited to the available test results which are representative of columns having relatively large overall slenderness ratios; λ_c between 1.1 and 2. In order to study the behavior over a wider range of cross-secions, the developed finite element model was used to conduct a parametric study to investigate the effect of the following design parameters:

1- Variation in the overall slenderness ratio λ_c.
2- Variation in the local width-to-thickness ratios of the web and the flange.
3- Variations in the amplitude of geometric imperfection.

For the first design parameter, members having slenderness ratio λ_c between 0.5 and 2.5 were investigated. For the second design parameter, six typical SSMA cross sections with different web depths, flange widths, and thicknesses were used. These sections are 400S137-33, 400S137-68, 600S162-33, 600S162-97, 800S200-33, and 800S200-97. For the third design parameter, an imperfection sensitivity analysis was carried out by applying imperfection values corresponding to 25%ile and 75%ile to the finite element model. In all cases, the longitudinal screw spacing was taken equal to one third of the member length to satisfy section D1.2 provisions. A total of 60 cases were investigated. The axial strength predicted by the numerical model were compared with the corresponding design strength as calculated using AISI provisions of Sections C4 and D1.2. Figs. 8-13 show a comparison between the finite element strengths with the nominal design strengths obtained using AISI provisions.

It can be seen that the AISI specifications are generally conservative, except for short columns with λ_c around 0.5, where the AISI specifications overestimates the column strengths. This can be explained by the fact that the member behavior in these regions is governed by local buckling rather than overall buckling. The AISI provisions are based on research conducted on hot rolled sections where local buckling rarely governs the design. Additional numerical and experimental work is needed to study this point.

5- Conclusions

This paper presents a finite element procedure for calculating the axial buckling strength of cold-formed built up I-sections. The initial local and overall geometric imperfections, residual stresses, nonlinear material properties of flat and corner portions have been included in the finite element model. The comparison between the finite element results and the experimental investigation of 32 columns with different geometric dimensions showed that the current AISI design provisions are conservative for members medium and long members. A parametric study of 60 columns was performed using the finite element model to investigate the effect of major design parameters on the behavior. The results of the parametric study showed that the design rules specified in the American Specification are generally conservative for medium and long members but may overestimate the capacity for short members.

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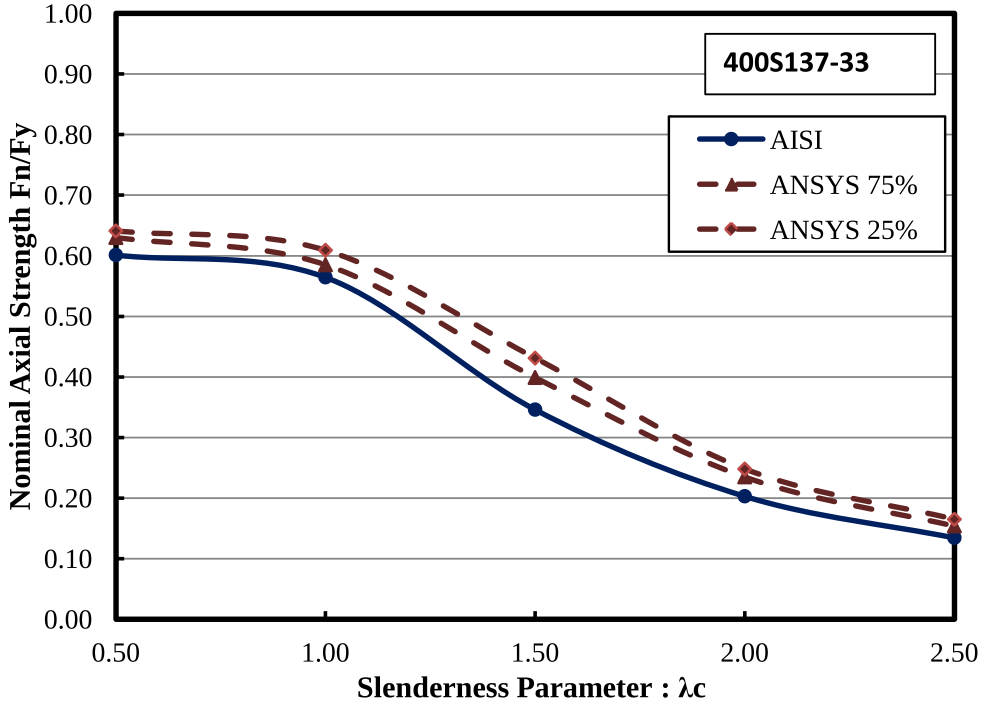

Figure 8 Comparison of FE strengths with design strengths for section 400S137-33

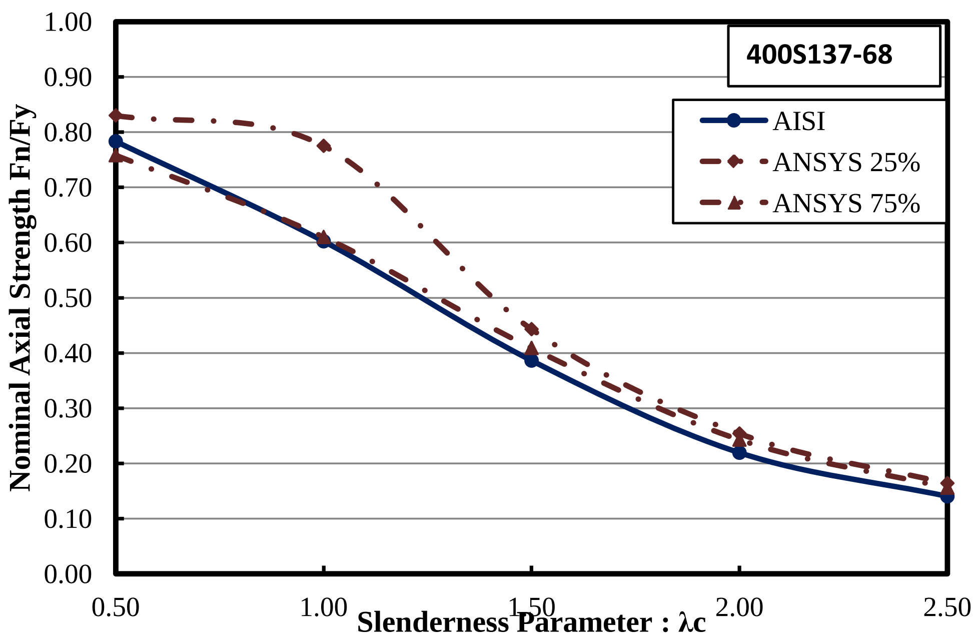

Figure 9 Comparison of FE strengths with design strengths for section 400S137-68

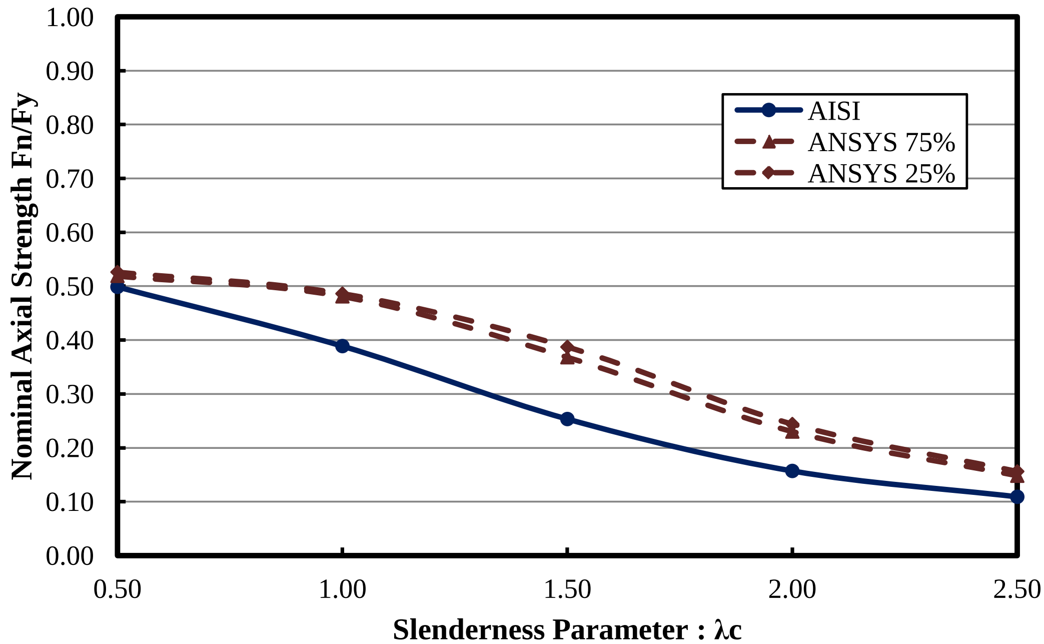

Figure 10 Comparison of FE strengths with design strengths for section 600S162-33

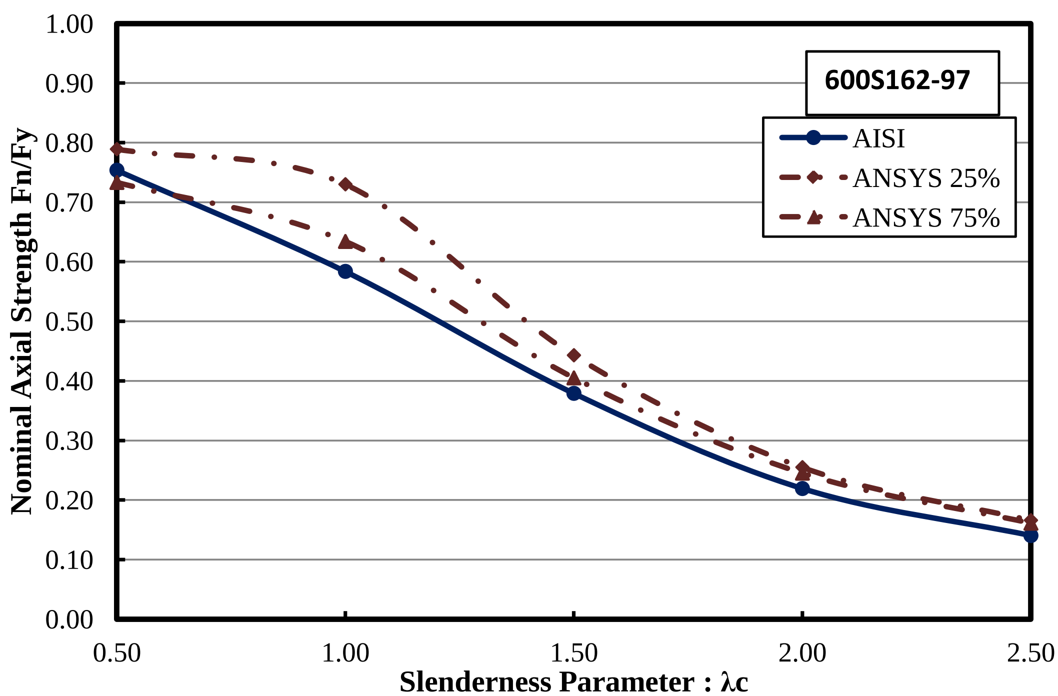

Figure 11 Comparison of FE strengths with design strengths for section 600S162-97

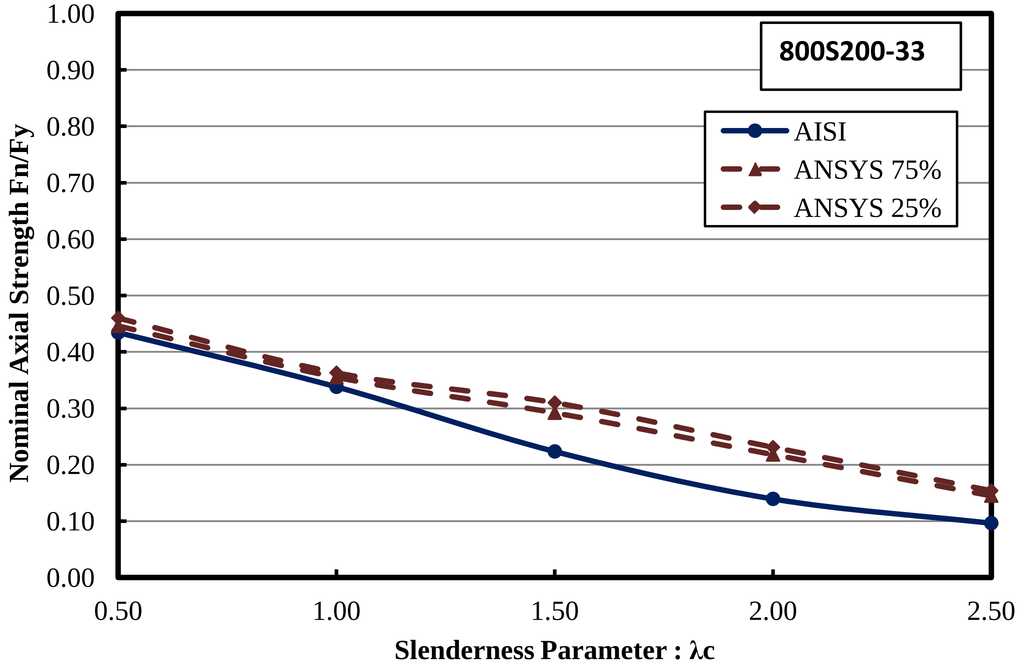

Figure 12 Comparison of FE strengths with design strengths for section 800S200-33

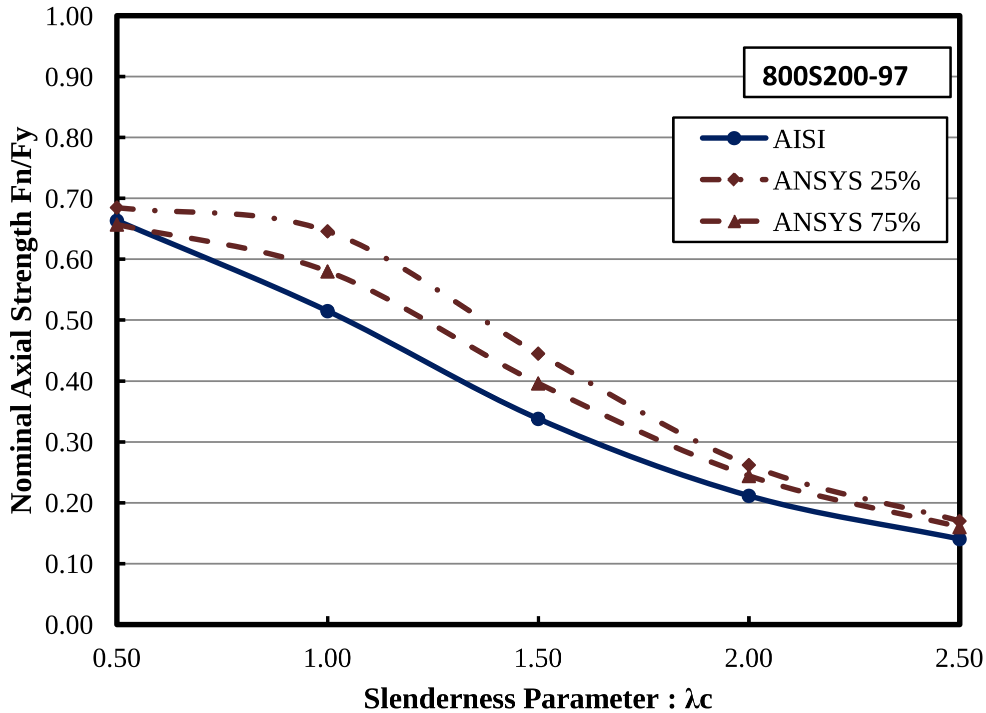

Figure 13   Comparison of FE strengths with design strengths for section 800S200-97

References

Abdel-Rahman, N. and Sivakumaranz, K. (1997): *Material Properties Models for Analysis of cold-Formed Steel Members*, ASCE, Journal of Structural Engineering, Vol. 123, No.9, September, 1997, 1134-1143.

AISC Design Specifications (2010), American Institute of Steel Construction.

American Iron and Steel Institute (AISI) (2007). *North American specification for the design of cold formed steel structural members*. Washington, DC.

Brueggen, B., and Ramseyer, C. (2003). *Cold-formed steel joist member buckling capacity testing*. Technical Report, University of Oklahoma, Norman, OK.

Biggs, K. A. (2008). "*Axial load capacity of cold-formed, built-up members*." MS thesis, University of Oklahoma, Norman, OK.

Moen, C. D., Igusa, T., Schafer, B.W. (2008): *Prediction of residual stresses and strains in cold-formed steel members*, Thin-Walled Structures, Volume 46 (2008), pp. 1274-1289.

Piyawat, K. (2011) "*A suggested design approach for built-up cold-formed columns based on research using nonlinear finite element method.*" Ph.D. thesis, University of Oklahoma, Norman, OK.

Schafer B.W., Pekoz T. (1998) "*Computational modeling of cold-formed steel: characterizing geometric imperfections and residual stresses*". Journal of Constructional Steel Research 1998;47(3):193–210.

Schafer B.W., (2008): "*Review: The Direct Strength Method of cold-formed steel member design*". Journal of Constructional Steel Research; 47(3):193–210.

Schafer, B.W., Li, Z., Moen, C.D. (2010), *Computational modeling of cold-formed steel*, Thin Walled Structures, Vol. 48, pp 752-762

Swanson Analysis systems Inc. (2011), ANSYS Technical Manual.

Whittle, J. (2007). "*Buckling capacities of axial loaded cold-formed built-up members*." MS thesis, University of Oklahoma, Norman, OK.

Whittle, J., and Ramseyer, C. (2009). "*Buckling capacities of axially loaded, cold-formed, built-up channels*." Thin-Walled Structures, 47(2), 190–201.

Zeinoddini, V., Schafer, B.W. (2012), *Simulation of geometric imperfections in cold-formed steel members*, Proceedings of the Annual Stability Conference, Structural Stability Research Council, Grapevine, Texas.