Toward the Simplified Design of Single-Angle Beam-Columns

C.J. EARLS and D. CHRISTIAN KEELOR

Single-angle compression members are attractive choices in design as a result of geometric attributes that seemingly lead to simple connection details. Common application areas for these types of members are in lattice towers and in trusses wherein the connection detail involves coupling of the single-angle compression members to another element such as a chord member in a truss or a corner member in a lattice tower. While design contexts such as these arise naturally in the preliminary design phase of a given structure, the designer soon learns that these seemly simple members and connections are problematic to treat within the context of design provisions contained in the AISC specifications (AISC, 1999, 2000, 2005).

CURRENT SPECIFICATION APPROACHES

Since the commonly encountered applications mentioned previously oftentimes result in the development of loading eccentricities on the order of 1.5 times the centroidal distance, $\bar{y}$ (as shown in Figure 1), end moments will develop. It has been the position of AISC (1999, 2000) that these types of members should be treated as beam-columns with end moments amplified to account for the relevant "second-order" moment effects; although this is being revisited in the most recent AISC *Specification for Structural Steel Buildings* (AISC, 2005). In practice, this design approach is quite tedious and complicated and results in capacity predictions that are not as accurate as one might hope; test loads from the laboratory are consistently reported to be two and three times the AISC predicted capacities (Temple and Sakla, 1998a, 1998b).

C.J. Earls is associate professor and William Kepler Whiteford Faculty Fellow, department of civil and environmental engineering, University of Pittsburgh, Pittsburgh, PA.

D. Christian Keelor is structural engineer, The Osborn Engineering Company, Cleveland, OH.

The American Society of Civil Engineers has adopted an alternate approach wherein the bending effects due to load eccentricity, as well as the restraint effects of adjacent members, are accounted for through the imposition of modified effective length factor equations developed from testing results obtained using full-scale truss and tower specimens (ASCE, 2000). Such an approach as this offers the designer a greatly simplified approach as compared with past AISC approaches (AISC, 1999, 2000).

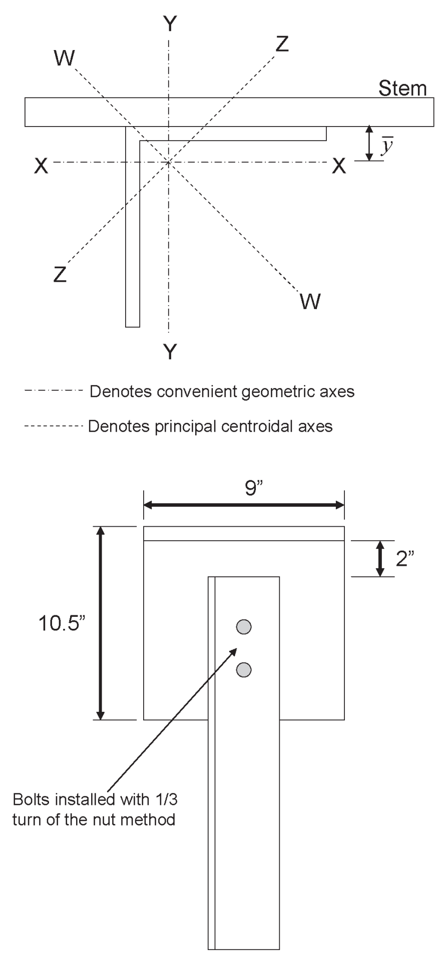

Fig. 1. Schematic of test specimens of Mengelkoch and Yura (2002).

PROBLEM STATEMENT

It is believed that the AISC specification provisions (AISC, 1999, 2000) for the design of single-angle columns eccentrically loaded through one leg are complicated and somewhat inaccurate (Lutz, Temple, and Sakla, 1996; Temple and Sakla, 1998a, 1998b; Mengelkoch and Yura, 2002). Mengelkoch and Yura (2002) have proposed an alternate and simplified design approach to the problem; modeled after the ASCE procedure contained in *Design of Latticed Steel Transmission Structures* (ASCE, 2000), wherein load eccentricity and connection end restraint are considered through the use of modified effective length factors. The approach of Mengelkoch and Yura has shown promise in terms of accuracy in strength prediction; as observed during comparison studies carried out on 10 laboratory specimens possessing two different angle sections and two different overall column slenderness values. The research outlined in the current paper seeks to add to this knowledge base of documented angle response through the consideration of 16 additional single-angle members treating four different column slenderness ratios, two different loading eccentricities, and three different end restraints. An overview of earlier research on this topic follows in the next section, but one point should be made up front. The current research treats loading eccentricity separately from restraint effects at the connection. In all of the earlier studies involving individual, isolated single-angle columns, the column end connections were made using WT sections either bolted or welded to a single-angle leg through the WT stem while the WT flanges were fully supported. As a result of this approach, any change in loading eccentricity is achieved by increasing the stem thickness of the WT which in turn increases the rotational restraint at the connection, since the WT is subsequently more robust. Using computational approaches, the present study overcomes this obstacle, treating eccentricity and end restraint separately. Such an approach as this provides new insight into this problem.

EARLIER WORK

Confusion regarding the proper application of the AISC single-angle column design approach, as put forward in Example 3-8 of the 2nd edition LRFD *Manual of Steel Construction*, led to an article by Lutz (1996) wherein clarifications regarding the proper consideration of moment sense and load eccentricity calculations were provided. While the paper by Lutz contributed to reducing the overall conservatism in the AISC single-angle column design approach (AISC, 1999, 2000), the approach still remains cumbersome and somewhat inaccurate (Mengelkoch and Yura, 2002).

Mengelkoch and Yura (2002) conducted an experimental research program wherein 10 laboratory specimens, possessing two different angle sections and two different overall column slenderness values, were subjected to eccentric loading in the presence of end restraint. The specimen geometry was such that the load was transmitted through only one leg of the angle cross-section via a bolted connection that attached the angle leg to a WT stem as shown in Figure 1. The experimental results were used to gauge the accuracy of predictive equations that were applicable to this angle member configuration. The predictive equations followed two primary themes: (1) so-called beam-column approaches and (2) modified effective length approaches. One outcome of the work of Mengelkoch and Yura (2002) was that the predictive equation that they proposed (using a modified effective length approach) tended to provide the most favorable agreement with the results from the experimental test program. The capacity equations provided by Mengelkoch and Yura (2002) were essentially those proposed by ASCE (2000), but they were modified slightly to recognize that while the use of the minimum principal radius of gyration, r_z, of the cross-section, in computing overall column slenderness, may seem logical initially, the use of the *x-x* axis (see Figure 1) properties has a much greater phenomenological basis vis-à-vis experimental testing response. It is interesting to point out that this same notional change of cross-sectional axis was recommended by Temple and Sakla (1998a, 1998b) as a result of their experimental and numerical investigation (discussed in the next paragraph). Mengelkoch and Yura (2002) carried out an experimental research program leading to proposed modified column slenderness equations of the form:

for $0 \le L/r_x \le 75$

$$KL/r_x = 60 + 0.8(L/r_x) \qquad (1a)$$

and for $75 \le L/r_x \le 180$

$$KL/r_x = 45 + (L/r_x) \qquad (1b)$$

Canadian researchers (Temple and Sakla, 1998a, 1998b) carried out a detailed research study aimed at quantifying the ultimate strength capacity and governing response of single-angle compression members welded by one leg to a gusset plate. The study involved the experimental evaluation of 33 single-angle compression members attached to WT sections in a fashion that was consistent with the specimen geometry used by Mengelkoch and Yura (2002), and shown in Figure 1. The experimental testing results of Temple and Sakla (1998a, 1998b) were partly used to assist in the development of detailed nonlinear finite element models of single-angle column assemblies that were consistent with those considered in the experimental phase of testing. The analytical research program of Temple and Sakla employed the commercial finite element package ABAQUS (Hibbitt, Karlson, and Sorenson, 1994) to create shell-element-based models of the test specimens wherein rigid beam elements provided the bases for connecting the shell model of the angle cross-section to the shell model of the gusset plate while at the same time preserving the required physical dimensions needed for modeling eccentricity in loading. Agreement between the finite element modeling results and the experimental testing results were favorable (Temple and Sakla, 1998a). The verified finite element modeling techniques were subsequently used to perform a detailed parametric study (Temple and Sakla, 1998b) wherein the influences of parameters such as member slenderness, initial member out-of-straightness, residual stresses, variations in Young's modulus, and various gusset plate properties were quantified. The work of Temple and Sakla (1998a, 1998b) led to a new design equation and a host of observations—some of which imply a certain insensitivity of single-angle column axial capacity to parameters such as reasonable initial out-of-straightness, the presence or absence of residual stresses arising from fabrication processes, and the variation in Young's modulus in a statistically relevant context as associated with common steel grades. In fact, Temple and Sakla (1998b) concluded that the primary factor affecting single-angle ultimate axial capacity was the gusset plate thickness; gusset plate width was also noted to be of lesser importance. Temple and Sakla (1998b) surmised that the importance of the gusset plate thickness was directly related to the ability of the gusset to provide greater moment restraint about the *x-x* axis (see Figure 1). It is actually difficult to conclude too much along these lines since, as pointed out earlier, the net result of increasing gusset plate thickness is to increase both end-moment restraint as well as loading eccentricity. The research that is the focus of the current paper considers these effects separately and independently.

SCOPE OF CURRENT RESEARCH

The current research is limited in applicability to hot-rolled equal leg single-angle compression members made from mild carbon steel such as ASTM A36. Four different overall column slenderness ratios (L/r_x = 200, 140, 80, 20) are considered within the context of two different load eccentricities (1.2 and 1.5 $\times$ $\bar{y}$). For the case of L/r_x = 20 and 200, three different end-restraint conditions are considered. In all of the research reported on herein, the angle section used is L6×6×0.857, ASTM A36. It is once again noted that the research is computational in nature and the models created have the ability to independently treat loading eccentricity and end-moment restraint. The modeling results from the cases treated herein are compared with the predicted capacity for these same members as obtained from the AISC specifications (AISC, 1999, 2000), the equations proposed by Temple and Sakla (1998b), and the equations proposed by Mengelkoch and Yura (2002).

FINITE ELEMENT MODELING APPROACH AND ASSUMPTIONS

The finite element method forms the basis for the research program reported on herein. Specifically, the commercially available software package, ADINA (ADINA, 2003), is used in the current work. All models reported on herein consider both geometric (large rotation/small strain) and material (multilinear plasticity) nonlinear effects in an effort to capture all salient features in the response of single-angle columns at incipient buckling. The nonlinear solution strategy used in all modeling reported on herein is based on a combined method of load and displacement control wherein the spherical constant arc-length method (Crisfield, 1981) is used in regions along the equilibrium path far from critical points, and a scheme based on constant increments of external work are employed within regions along the equilibrium path that are close to critical points (Bathe and Dvorkin, 1983).

Modeling Techniques

Given that localized nonlinearities heavily influence the activation of single-angle column limit states in stocky members, a robust modeling strategy is adopted for the current work so as to properly account for the effects of distributed plasticity as well as localized plate buckling effects. Figure 2 depicts both the schematic and finite element modeling representations of the single-angle column geometry treated herein. Specifically, the eccentricity in loading and boundary conditions is modeled in such a way as to be consistent with the case of a single-angle column loaded through one leg by a gusset plate connection; as is consistent with common practice in truss connections.

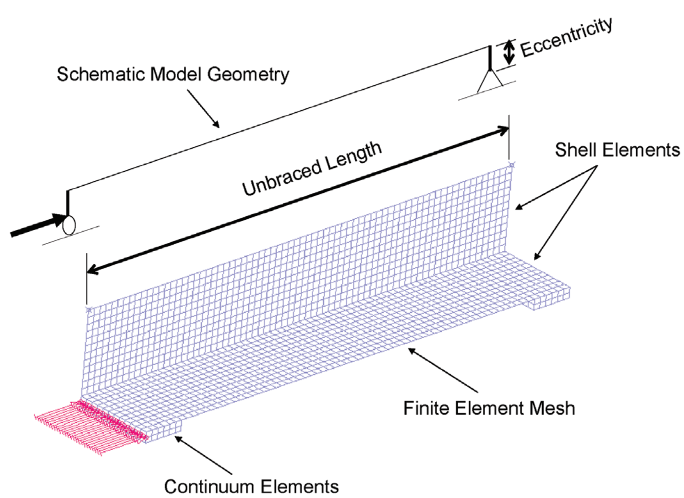

Fig. 2. Schematic and representative mesh used in single-angle column models.

Table 1. Idealized Multilinear ASTM A36 Steel Model Used in ADINA Models		
	Logarithmic Strain	True Stress
	0	0.00
	0.0014	42.27
	0.0161	43.75
Rupture strain ⇒	0.2689	75.00

Finite Element Mesh

Fully integrated, 9-node shell elements (MITC9) are used to represent the single-angle cross-section. The formulation used by ADINA for this type of shell is free from spurious zero energy modes by virtue of being fully integrated (3×3 Gauss quadrature in-plane), while at the same time being resistant to shear locking as a result of an assumed strain formulation involving mixed tensorial components of the shell kinematic response (Bathe and Dvorkin, 1986; Dvorkin and Bathe, 1984). Fully integrated 27-node continuum finite elements are used to construct the gusset plates occurring in the connection regions of the finite element models depicted in Figures 2 and 3. The 27-node continuum elements are selected so as to compliment the quadratic interpolation order used in the MITC9 shell elements occurring in the angle cross-section. Based on prior experience (Earls, 2001), it is noted that the mesh density employed in these models is much greater than is typically necessary in problems such as this, but since computational limits were not a factor, the present conservative mesh density is used for the current work.

Material Modeling

The finite element models considered herein employ a von Mises yield surface deployed within the framework of a standard associated flow metal plasticity model as the means by which inelastic material response is reckoned. For the case of the single-angle cross-section, an ASTM A36 material model is used. This material response is modeled after the coupon testing carried out by Mengelkoch and Yura (2002) on the single-angle specimens considered in their experimental testing program and presented in Table 1. It is noted that while Mengelkoch and Yura indicated that 50-ksi-yield strength steel was originally specified for their test specimens, angles are currently most commonly available in the A36 grade. In addition, the coupon testing results presented by Mengelkoch and Yura are more consistent with A36 steel. It is pointed out that since the coupon results presented by Mengelkoch and Yura were not specifically indicated to be static in nature, a 3 ksi reduction in the overall stresses was applied to the coupon results presented in their paper; a method that is consistent with the accepted practice for converting dynamic coupon test results to static test results (Galambos and Ravindra, 1978). The values reported in Table 1 are consistent with the 3 ksi reduction.

Boundary Conditions and Loading

As it is that in practice single-angle compression members are frequently subjected to eccentric loads transferred in the angle section through some sort of plate element typically attached to only one leg of the angle, care is taken to replicate this type of condition in all modeling reported on herein. Specifically, continuum elements are used to provide the physical dimensions required to develop any desired eccentricities with respect to the single-angle section centroid. Furthermore, since a common application scenario for single-angle compression members loaded through one leg is the case of a truss or lattice frame wherein some chord, or corner, member provides some degree of rotational restraint at the joint, capabilities to accommodate this effect are incorporated into the current models. Figure 3 displays schematically how the rotational spring is introduced into the finite element modeling in order that the effects of various degrees of rotational restraint about the *x-x* axis (see Figure 1) may be studied. Linear rotational spring finite elements are employed in the ADINA model to specifically treat this modeling requirement. The spring elements are positioned at each end of the column near the location where the loading and boundary conditions are imposed.

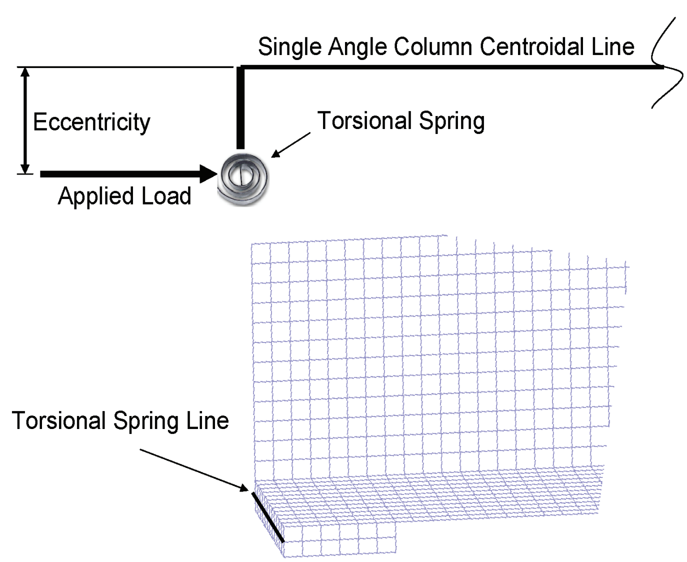

Fig. 3. Single angle column and end connection detail.

Besides springs providing restraint, the remaining boundary conditions are classified as being simple pinned-roller type in nature. All boundary conditions are imposed along a line passing through the centroid of the connector plate attached on one leg of the single-angle shell finite element mesh. The boundary conditions are imposed directly to all nodes along the line parallel to the angle *x-x* axis that passes through the connector plate centroid. The applied loading is imposed as concentrated nodal forces occurring along this same line.

Validation of Modeling Approach

As with any research program whose results are based solely on a given analytical technique, it is important to clearly demonstrate the validity of the modeling assumptions and strategies within the context of the problem under investigation. A useful way to address this point is through the application of the subject analytical modeling approach to the case of a carefully conducted experimental testing program focusing on the same type of problem. Fortunately for this research, just such a carefully conducted research program was carried out at the University of Texas by Mengelkoch and Yura (2002).

Overview of Test Geometry and Loading

Since the scope of the present research is limited to the case of equal-leg angle cross-sections, Specimens L1 and L2 from the test population of Mengelkoch and Yura were considered; specifically, the slender version of these specimens was considered. The important details related to testing geometry are that the angle cross-sections consisted of L3½×3½×⅜ for members that were 108 in. long. The end conditions employed in the test rig were consistent with what would be expected in a truss joint configuration in that WT 10½×22 tee stubs were bolted to the same single-angle leg, top and bottom. The flange of the WT was subsequently bolted to a base plate within the test section of a universal testing machine. Figure 1 provides more detail regarding the nature of the end segments and connections of the single-angle columns considered by Mengelkoch and Yura. The rotational spring elements (described earlier in the discussion on modeling approach) were not used in the verification studies since the level of restraint provided by the mentioned WT was quite small (in other words, considering the given loading eccentricity, the WT flange web junction forms a yield line at a load that is 0.7% of ultimate).

Comparison of Experimental and Analytical Column Response

At the mid-height of the columns, Mengelkoch and Yura monitored the centroidal displacement in the *x-y* plane (depicted in Figure 1) as well as cross-sectional twist. A comparison of the experimental results with the finite element analogues constructed using the same modeling strategies as outlined earlier herein, are presented in Figures 4 and 5. It is also noted that the experimental buckling load, averaged over the two Specimens L1 and L2, was 35.5 kips. The bucking load predicted by ADINA was 34.2 kips. Based on the foregoing, it seems reasonable to consider the finite element modeling techniques to be accurate in their ability to treat the single-angle column problem that is the focus of the current study.

SINGLE-ANGLE COLUMN GEOMETRIES CONSIDERED IN THE CURRENT WORK

Four different overall column slenderness ratios (L/r_x = 200, 140, 80, 20) are considered within the context of two different load eccentricities (1.2 and 1.5 × $\bar{y}$). The maximum slenderness limit considered herein is based on an application

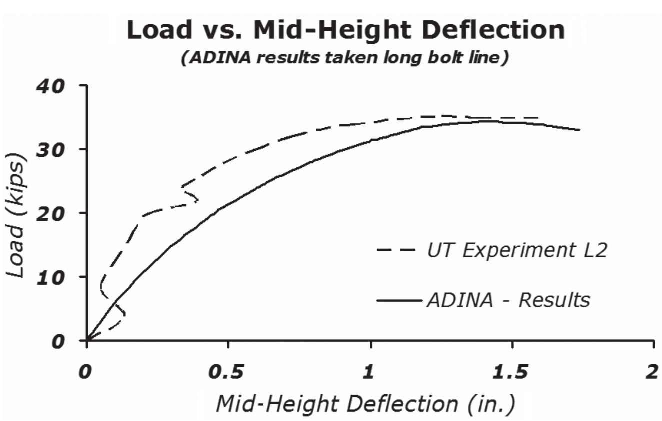

Fig. 4. Comparison of experimental and finite element results.

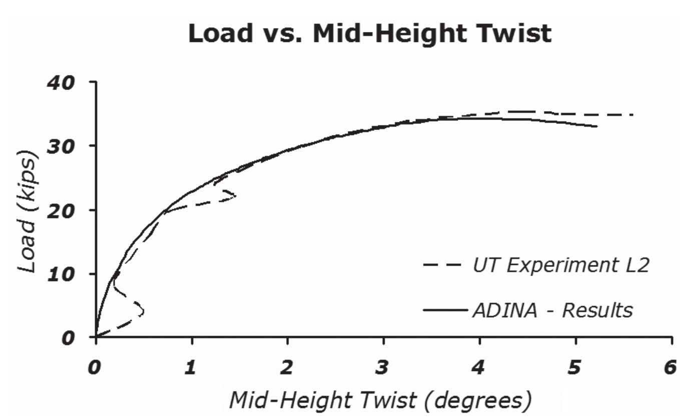

Fig. 5. Comparison of experimental and finite element results.

of the slenderness limit for columns provided in Section B7 of the AISC *LRFD Specification for Structural Steel Buildings* (AISC, 1999). Similarly, the minimum slenderness used in the present study was arrived at by considering the shortest angle length that would still result in classical flexural behavior (in other words, the effects of shear are still small). Based on prior experience, a value of 7 times the depth of the cross-section is a reasonable limit to focus on with regard to the maintenance of beam behavior; thus resulting in the present lower bound on column slenderness of 20.

For the cases of L/r_x = 20 and 200, three different end-restraint conditions are considered. In all of the research reported on herein, the angle section used is an ASTM A36, L6×6×0.857. This section is selected since it results in a plate slenderness ratio (b/t) of 7.0—a value significantly less than the limiting plate slenderness value delineating slender element response (b/t = 10.7). This condition on the angle cross-section is desirable because the effects of localized buckling are considered inapplicable to the current discussion since the design equations of Mengelkoch and Yura (2002), and thus ASCE (2000) as well, consider a modified overall column slenderness approach—a methodology that is not conducive to the consideration of local buckling effects.

Loading Eccentricity

As mentioned in the previous section on finite element modeling strategies, the models achieve the desired loading eccentricity through the overt consideration of actual gusset plate dimensions. This means that while thickness values are implied in the shell finite element formulation used in the MITC9 elements making up the single-angle cross-section, actual continuum (or so-called brick) elements are used to build up the proper gusset plate thickness in the model through the imposition of nodal coordinates dictated by physical gusset plate dimensions. The eccentricities considered in this work vary from 2.2 in. up to 2.7 in. (in other words, 1.2 to 1.5 × $\bar{y}$).

End Restraint

In quantifying reasonable end-restraint conditions, consideration is paid to Section A2 in Chapter A of the Commentary to the *LRFD Specification for Structural Steel Buildings* (AISC, 1999), hereafter referred to as the LRFD Specification, wherein the definition of a "simple" or "type I" connection is one in which end restraint in the amount of 0.2 times the theoretical plastic section capacity of the member results in a connection rotation of no more than 0.02 radian. For the section considered in this research, the limiting connection stiffness for simple response would then be 4,866 kip-in./radian.

Considering the elastic response of the single-angle member as a beam, the theoretical member stiffness under the action of concentrated end-moments (in this case due to eccentrically applied axial loading) results in a value of 1,852,187/L (in other words, $k_{IDEAL} = 2EI/L$). In the modeling of single-angle cases having column slenderness values of L/r_x = 20 and 200, values of $0.1k_{IDEAL}$ and $0.2k_{IDEAL}$ are arbitrarily selected as reasonable stiffness values for consideration in the finite element modeling. While these cases seem to be reasonable when considering elementary structural theory, some discussion is necessary with regard to these stiffness choices in relation to the "pinned" connection limits from Section A2 of the Commentary to the LRFD Specification.

In the case of the more slender member (in other words, L/r_x = 200), the values for $0.1k_{IDEAL}$ and $0.2k_{IDEAL}$ result in connections that are 9.54 times less stiff and 4.77 times less stiff, respectively, than the 4,866 kip-in./radian limit specified by Section A2 of the LRFD Commentary for this angle cross-section. In the case of the stockier member (in other words, L/r_x = 20), the values for $0.1k_{IDEAL}$ and $0.2k_{IDEAL}$ result in connections that are 1.05 times more stiff and 2.11 times more stiff, respectively, than the 4,866 kip-in./radian limit specified by Commentary Section A2 for this angle cross-section.

So the fundamental question in terms of a comparison of relative end restraint is related to whether it is more proper to consider the plastic capacity of the cross-section as the basis for comparison or, rather, the elastic stiffness of the given member as a whole. While both approaches have their own merits, it seems that the consideration, in the latter case, of overall member flexibility is more germane to the current work since this is consistent with notions related to classical effective length considerations [and thus consistent with the design approach considered by ASCE (2000) and Mengelkoch and Yura (2002)].

RESULTS AND DISCUSSION

A summary of the results obtained from the present study are contained in Table 2 and contextualized in terms of differing strength predictions, both from current practice and the archival literature, in Table 3. Based on the last column in Table 2, wherein the normalized ultimate load for the cases with L/r_x = 20 and 200, with varying degrees of end restraint, is presented, it is clear that reasonable levels of end restraint do not significantly affect the single-angle compressive capacity. Indeed, it is the effect of loading eccentricity itself that has the most profound effect (see Table 2), and then only for the case of very stocky angle columns.

In the case of the existing literature and design specification guidance, this issue clearly is important. In terms of the 1999 AISC LRFD Specification (AISC, 1999), we see from Table 3 that a gross underprediction of single-angle column capacity results from its use. The column capacities obtained

Table 2. Finite Element Results						
ADINA Results: Single-Angle Ultimate Compressive Capacity, P_u						
KL / r_x	End Restraint (Multiple of k_{IDEAL})	Eccentricity			Restrained P_u / Unrestrained P_u	
		P_u $e1 = 1.2\ \bar{y} = 2.2$ in.	P_u $e2 = 1.5\ \bar{y} = 2.7$ in.	P_{ue1} / P_{ue2}	e1	e2
200	0	51.2	48.0	0.94	–	–
	0.1	51.2	48.1	0.94	1.000	1.002
	0.2	51.2	48.1	0.94	1.000	1.002
140	0	88.74	80.9	0.91	–	–
80	0	169.0	147.3	0.87	–	–
20	0	246.9	161.6	0.65	–	–
	0.1	247.5	162.1	0.65	1.002	1.003
	0.2	247.5	162.1	0.65	1.002	1.003

Table 3. Comparison of Finite Element Results with Predictive Equations						
ADINA Results: Ultimate Compressive Capacity (kips)				Predicted Ultimate Compressive Capacity (kips)		
KL / r_x	End Restraint (Multiple of k_{IDEAL})	Eccentricity		AISC	Mengelkoch & Yura	Temple & Sakla**
		P_u $e1 = 1.2\ \bar{y} = 2.2$ in.	P_u $e2 = 1.5\ \bar{y} = 2.7$ in.			
200	0	51.2	48.0	e1 → 20.0 e2 → 19.1	39.9*	e1 → 39.7 e2 → 59.1
	0.1	51.2	48.1			
	0.2	51.2	48.1			
140	0	88.74	80.9	e1 → 36.2 e2 → 33.7	70.0	e1 → 62.9 e2 → 93.6
80	0	169.0	147.3	e1 → 79.9 e2 → 70.9	153.5	e1 → 109.8 e2 → 163.3
20	0	246.9	161.6	e1 → 138.0 e2 → 117.4	282.9	e1 → 180.4 e2 → 268.2
	0.1	247.5	162.1			
	0.2	247.5	162.1			

* The design expressions from Mengelkoch & Yura are not strictly applicable for L/r_x >180.
** The design expressions from Temple and Sakla are strictly only applicable for cases wherein L/r_x is from 50 to 200; the gusset width-to-angle leg width ratio is between 1.4 and 4.0; and the ratio of gusset plate thickness-to-angle leg thickness is between 0.1 and 0.25.

from the AISC approach are less than half the capacity determined with nonlinear finite element techniques. This result is in agreement with very similar observations made by Temple and Sakla (1998b).

When considering the strength predictions recommended by Mengelkoch and Yura (2002), a much better agreement is noted between the finite element modeling and their predictions. For column slenderness values between 200 and 80, the predictions of Mengelkoch and Yura are on the conservative side. However, at a column slenderness value of 20, their prediction appears to be unconservative. It is also important to note that the approach of Mengelkoch and Yura (2002) is not able to treat varying degrees of eccentricity in the loading within the single-angle column. While it is important to note this shortcoming for the sake of completeness, in a practical sense this can be made to be unimportant if the results are always conservative for the maximum eccentricity that might be realistically expected (arguably e2 as noted in Table 3). Using an approach such as this, the predictions of Mengelkoch and Yura will be more conservative for smaller eccentricities, but any such conservatism is insignificant when compared with the conservatism of the current AISC approach. It is important to understand the nature of the observed unconservatism at the short column lengths before the approach of Mengelkoch and Yura (2002) is adopted for design purposes. It is pointed out that the approach of Mengelkoch and Yura (2002) is not capable of treating the case of variable loading eccentricities and it may well be that this characteristic is related to the observed unconservatism, at short column lengths, noted previously.

Table 3 presents results from the application of the approach recommended by Temple and Sakla (1998b), which is obtained using an equation that is a curve-fit to a very large sample of finite element data. Strictly speaking, the angle geometry considered in the current work falls outside the range of applicability for the equation as noted by Temple and Sakla (1998b), and thus it is not reasonable to expect excellent agreement with the current modeling results. The results from the predictions of Temple and Sakla (1998b) are only included for general interest and to highlight the fact that, potentially, the effects of end restraint may be important if the formation of a plastic hinge within the gusset plate is the critical factor governing single-angle column capacity [as in all of the work reported on by Temple and Sakla (1998a, 1998b), they employed very thin gussets, without fail, in all 1,800 finite element cases reported on].

CONCLUSIONS

Based on the result of the present study involving the consideration of single-angle column capacity, it is clear that the previous AISC (1999) approach for computing column capacity was grossly conservative (in other words, it underpredicts strength by a factor of 2 or more). The new approach for computing single-angle column capacity that is given by Mengelkoch and Yura (2002), and that has subsequently been adopted by AISC and included in the new combined specification (AISC, 2005), is a dramatic improvement over the earlier AISC approach (AISC, 1999). However, the unconservative strength predictions at low column slenderness values are important to note. Also important to note is that the approach of Mengelkoch and Yura (2002) and AISC (2005) is not capable of treating the case of variable loading eccentricities, and it may well be that this shortcoming is related to the observed unconservatism at short column lengths noted previously.

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