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ELASTIC LATERAL-DISTORTIONAL BUCKLING OF SINGLY SYMMETRIC I-BEAMS: THE 2005 AISC SPECIFICATION

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***Abstract**. It is clear from prior research studies that the web distortional flexibility can lead to a substantial reduction relative to the beam theory lateral-torsional buckling resistance for I-sections with stocky flanges and slender webs. Hence, the 2005 AISC Specification gives specific rules for controlling the unconservative errors due to the neglect of web distortion effects. The accuracy of the 2005 AISC code predictions in case of elastic lateral-distortional buckling of singly symmetric I-beams is investigated in this paper through comparison with the accurate finite strip analysis distortional buckling solutions as well as the theoretical predictions of two elastic distortional buckling design equations proposed by other researchers. The code predictions in case of lateral-distortional buckling of slender-web singly symmetric I-beams are found to be by and large conservative, and even overconservative in some cases.*

1 INTRODUCTION

For the *slender-web* I-sections, the 2005 AISC Specification [1] bases the lateral-torsional buckling resistance on Eq. (1), but the St. Venant torsional constant J is taken equal to zero.

$$F_{cr} = \frac{C_b \pi^2 E}{(L_b/r_t)^2}\sqrt{1+0.078\frac{J}{S_x h_o}(L_b/r_t)^2} \tag{1}$$

In fact, the implicit use of $J = 0$ in Section F5 of the 2005 AISC Specification is intended to account for the influence of web distortional flexibility on the lateral-torsional buckling resistance for slender-web I-section members [2].

In addition to the destabilizing effect of web distortion in a slender-web *singly* symmetric I-beam which results in lowering of the torsional rigidity of the beam, this may be coupled with the influence of the *Wagner* effect to reduce significantly the buckling strength of the singly symmetric beam [3].

This paper focuses on distortional buckling of singly symmetric I-shaped flexural members with slender webs, and evaluates the effectiveness of the 2005 AISC code rules by comparing the code predictions with finite strip analysis (FSA) distortional buckling solutions developed using the finite strip analysis software CUFSM [4] as well as the theoretical predictions of Bradford's (Eq. (2)) [5] and Wang *et al.*'s (Eq. (3)) [6] proposed distortional buckling design equations in the elastic range of structural response.

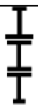
$$\frac{F_{crd}}{F_{cr}} = 1 - \frac{490\left(t_f / b_{ft}\right)\left(t_f / t_w\right)\left(1 - 0.560\left(b_{fc} / b_{ft}\right)\right)}{E / F_{cr}} \tag{2}$$

$$\frac{\gamma}{\bar{\gamma}_o} = 1 - \frac{f\Psi\left[\Phi\alpha + 10(1-\alpha)\right]}{\alpha\xi\Phi^2} \le 1.0 \tag{3}$$

2 CONSIDERED I-BEAMS

All of the I-beams in this study have compact flanges and slender webs in accordance with the compact-flange and noncompact-web limits specified in the AISC Specification [1]. The cross-sectional dimensions, lengths, and yield strengths of the I-beams considered for each case study are summarized in Table 1.

Table 1: Summary of beam dimensions and yield strengths of the I-beams

Case	Section	h_o *(mm)*	t_w *(mm)*	b_{fc} *(mm)*	t_{fc} *(mm)*	b_{ft} *(mm)*	t_{ft} *(mm)*	L_b *(mm)*	F_y *(MPa)*
1		750	4.5	varies	20	105	20	3,000	345
		750	4.5	105	20	105	20	3,000	345
		750	4.5	105	20	varies	20	3,000	345
2		625	4.0	90	15	110	15	varies	345
		625	4.0	110	15	90	15	varies	345
3		890	varies	85	12	120	12	4,000	345
		890	varies	120	12	85	12	4,000	345
4		670	4.0	60	varies	100	varies	3,000	345
		670	4.0	100	varies	60	varies	3,000	345
5		800	4.0	70	20	110	20	3,500	varies
		800	4.0	110	20	70	20	3,500	varies

3 EFFECT OF SECTION MONOSYMMETRY (CASE 1)

Based on the findings of the previous studies, for a beam whose compression flange is the smaller flange, the reductions in the elastic critical stress due to web distortion increase as the degree of monosymmetry increases, while when the larger flange is the compression flange, the reductions in the elastic critical stress decrease as the degree of monosymmetry increases. The formula for the coefficient of monosymmetry (β_x) for a general I-shaped singly symmetric beam is provided by Galambos [7]. The elastic distortional buckling results are summarized in Table 2.

Table 2: Distortional buckling results (Case 1)

Section	Beam	β_x	M_{nAISC}/M_{nLTB}	M_{nFSA}/M_{nLTB}	$M_{nBradford}/M_{nLTB}$	$M_{nWangetal.}/M_{nLTB}$
	B1-1	-591.71	0.58	0.81	0.90	0.76
	B1-2	-466.66	0.66	0.82	0.88	0.76
	B1-3	-331.87	0.73	0.84	0.86	0.76
	B1-4	-195.08	0.78	0.86	0.84	0.77
	B1-5	-62.87	0.82	0.87	0.83	0.77
	B1-6	0.00	0.83	0.88	0.82	0.77
	B1-7	62.87	0.84	0.88	0.82	0.78
	B1-8	195.08	0.85	0.89	0.84	0.78
	B1-9	331.87	0.85	0.89	0.86	0.78
	B1-10	466.66	0.86	0.90	0.90	0.79
	B1-11	591.71	0.87	0.91	0.99	0.80

As it is seen in the table, the AISC code predictions seem to be remarkably conservative relative to the FSA solutions particularly in  sections as the section monosymmetry increases.

4 VARIATION OF LENGTH (CASE 2)

In this case, the code predictions are evaluated as a result of variation of length, while the cross-sectional dimensions of the beams are all kept constant. The summary of the elastic distortional buckling results is presented in Table 3.

Table 3: Distortional buckling results (Case 2)

Section	Beam	L_b (mm)	M_{nAISC}/M_{nLTB}	M_{nFSA}/M_{nLTB}	$M_{nBradford}/M_{nLTB}$	$M_{nWangetal.}/M_{nLTB}$
	B2-1	3,000	0.81	0.90	0.90	0.87
	B2-2	3,500	0.76	0.91	0.93	0.90
	B2-3	4,000	0.72	0.92	0.94	0.91
	B2-4	4,500	0.68	0.93	0.95	0.93
	B2-5	5,000	0.65	0.94	0.96	0.94
	B2-6	6,000	0.58	0.96	0.97	0.96
	B2-7	8,000	0.47	0.99	0.98	1.00
	B2-8	3,000	0.89	0.93	0.90	0.86
	B2-9	3,500	0.85	0.93	0.92	0.88
	B2-10	4,000	0.82	0.93	0.94	0.89
	B2-11	4,500	0.79	0.93	0.95	0.90
	B2-12	5,000	0.76	0.94	0.96	0.90
	B2-13	6,000	0.70	0.94	0.97	0.91
	B2-14	8,000	0.59	0.94	0.98	0.92

It is generally accepted that the distortional effects are smaller in longer beams. This fact is clearly demonstrated by the FSA as well as the theoretical predictions of other two design equations, as shown in

Table 3. However, the AISC code predictions demonstrate a distinct trend by providing reductions increasing from 19% to 53% for ⊥ sections, and 11% to 41% for ⊤ sections, as the beam length increases. It is quite obvious that the 2005 AISC code [1] equations provide remarkably conservative results relative to the FSA and the other considered theoretical predictions especially in longer beams.

5 VARIATION OF WEB THICKNESS (CASE 3)

The elastic distortional buckling code predictions are assessed in this case as a result of variation of web thickness, while the other beam dimensions are kept constant. The results of this case study are tabulated in Table 4.

Table 4: Distortional buckling results (Case 3)

Section	Beam	h_o/t_w	M_{nAISC}/M_{nLTB}	M_{nFSA}/M_{nLTB}	$M_{nBradford}/M_{nLTB}$	$M_{nWangetal.}/M_{nLTB}$
	B3-1	296.67	0.66	0.91	0.96	0.90
	B3-2	254.29	0.70	0.93	0.97	0.92
	B3-3	222.50	0.73	0.95	0.98	0.94
⊥	B3-4	197.78	0.75	0.96	0.98	0.95
	B3-5	178.00	0.78	0.97	0.98	0.96
	B3-6	161.82	0.79	0.98	0.98	0.97
	B3-7	148.33	0.80	0.99	0.99	0.98
	B3-8	296.67	0.82	0.94	0.96	0.87
	B3-9	254.29	0.85	0.95	0.97	0.89
	B3-10	222.50	0.87	0.96	0.97	0.91
⊤	B3-11	197.78	0.88	0.96	0.98	0.92
	B3-12	178.00	0.90	0.97	0.98	0.93
	B3-13	161.82	0.91	0.97	0.98	0.94
	B3-14	148.33	0.92	0.98	0.98	0.94

As it is seen in Table 4, the difference between the results of the distortional and lateral-torsional solutions in both monosymmetry cases tends to increase as the web becomes more slender. However, the code reductions in case of singly symmetric beams with smaller compression flange are relatively larger than those of singly symmetric beams with larger compression flange. It is notable that both sets of reductions are comparatively larger than the respective reductions of the FSA as well as the other theoretical solutions.

6 VARIATION OF FLANGE THICKNESS (CASE 4)

The effects of web distortion may also vary as a result of variation of flange thickness in I-beams. Hence, in this case, the accuracy of the code predictions is investigated for varying flange slenderness ratios in singly symmetric I-beams. Table 5 summarizes the elastic distortional buckling results for both orientations of the I-beam.

Table 5: Distortional buckling results (Case 4)

Section	Beam	b_{fc}/t_{fc}	M_{nAISC}/M_{nLTB}	M_{nFSA}/M_{nLTB}	$M_{nBradford}/M_{nLTB}$	$M_{nWangetal.}/M_{nLTB}$
I	B4-1	7.50	0.77	0.99	0.99	0.99
	B4-2	6.00	0.74	0.97	0.98	0.97
	B4-3	5.00	0.70	0.95	0.97	0.95
	B4-4	4.29	0.66	0.91	0.95	0.91
	B4-5	3.75	0.63	0.87	0.93	0.86
	B4-6	3.33	0.59	0.82	0.90	0.80
T	B4-7	12.50	0.93	0.97	0.99	0.93
	B4-8	10.00	0.93	0.97	0.99	0.91
	B4-9	8.33	0.91	0.96	0.98	0.89
	B4-10	7.14	0.90	0.94	0.98	0.87
	B4-11	6.25	0.88	0.93	0.97	0.84
	B4-12	5.56	0.86	0.91	0.96	0.81

From the table, it is found that the predictions of the AISC code equation are by and large below the predictions of the FSA as well as the two proposed design equations, and the amount of conservatism of the code predictions seems to be relatively high in sections with smaller compression flange.

7 VARIATION OF YIELD STRENGTH (CASE 5)

The effect of variation of yield strength on distortional buckling of singly symmetric I-beams is investigated in this study, which is believed to provide us with a better understanding of the implications of web distortion as a result of variation of yield strength. A wide range of yield strengths, *i.e.* from 250 MPa to 690 MPa, are considered in this study, which are tabulated in Table 6. Distortional buckling results of this case study are given in Table 6.

Table 6: Distortional buckling results (Case 5)

Section	Beam	F_y (MPa)	M_{nAISC}/M_{nLTB}	M_{nFSA}/M_{nLTB}	$M_{nBradford}/M_{nLTB}$	$M_{nWangetal.}/M_{nLTB}$
I	B5-1	250	0.62	0.80	0.90	0.74
	B5-2	290	0.61	0.80	0.90	0.74
	B5-3	345	0.60	0.80	0.90	0.74
	B5-4	415	0.59	0.80	0.90	0.74
	B5-5	485	0.58	0.80	0.90	0.74
	B5-6	550	0.57	0.80	0.90	0.74
	B5-7	620	0.57	0.80	0.90	0.74
	B5-8	690	0.56	0.80	0.90	0.74
T	B5-9	250	0.77	0.89	0.93	0.78
	B5-10	290	0.85	0.89	0.93	0.78
	B5-11	345	0.84	0.89	0.93	0.78
	B5-12	415	0.83	0.89	0.93	0.78
	B5-13	485	0.82	0.89	0.93	0.78
	B5-14	550	0.82	0.89	0.93	0.78
	B5-15	620	0.82	0.89	0.93	0.78
	B5-16	690	0.81	0.89	0.93	0.78

From the table, it is evident that the predictions of the FSA as well as the other proposed equations are not affected by the variation of the yield strength in both $\perp$ and $\top$ cases, while the reductions induced by the AISC code equations are found to increase slightly in case of $\perp$ sections, and also initially decrease and then increase gradually in case of $\top$ sections, with the increasing of the yield strength. In any case, the AISC code equations seem to yield conservative predictions relative to the FSA results, and the conservatism in case of sections with smaller compression flange is considerably high.

8 CONCLUSION

The evaluation of effectiveness of the 2005 AISC code design rules in case of distortional buckling of singly symmetric I-beams demonstrates that the 2005 AISC code equations generally provide conservative strength estimates for elastic distortional buckling. Even the amount of this conservatism is found to be relatively high in case of singly symmetric I-beams with smaller compression flange. This indicates that the assumption of $J = 0$, which is used in Section F5 of the 2005 AISC Specification with the aim of controlling the unconservative errors due to the neglect of web distortion effects, may not be an appropriate approach to the problem, since it may impose economic burden in some cases.

REFERENCES

[1] American Institute of Steel Construction. *Specification for structural steel buildings*, Chicago, IL, AISC, 2005.

[2] American Institute of Steel Construction. *Commentary on the specification for structural steel buildings*, Chicago, IL, AISC, 2005.

[3] Bradford, M.A. and Waters, S.W. "Distortional instability of fabricated monosymmetric I-beams". *Computers & Structures*, **29**(4), 715-724, 1988.

[4] Schafer, B.W. CUFSM 3.12, *Elastic buckling analysis of thin-walled members by finite strip analysis*, 2006. (http://www.ce.jhu.edu/bschafer/cufsm)

[5] Bradford, M.A. "Distortional buckling of monosymmetric I-beams". *Journal of Constructional Steel Research*, **5**(2), 123-136, 1985.

[6] Wang, C.M., Chin, C.K., and Kitipornchai, S. "Parametric study on distortional buckling of monosymmetric beam-columns". *Journal of Constructional Steel Research*, **18**(2), 89-110, 1991.

[7] Galambos, T.V., *Guide to Stability Design Criteria for Metal Structures*, 5th Ed., John Wiley & Sons, New York, 1998.

SYSTEM STABILITY DESIGN CRITERIA FOR ALUMINUM STRUCTURES

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Abstract. *The 2010 Aluminum Association Specification for Aluminum Structures has been significantly revised to include more transparent stability provisions. Second-order effects, including P-Δ and P-δ moments, and factors known to accentuate these effects, such as geometric imperfections and member inelasticity, will need to be considered in determining required strengths. This paper provides an overview of these provisions and describes experimental and analytical studies that investigated their effectiveness.*

1 INTRODUCTION

Widely used in the US since its first publication in 1967, the Aluminum Association's (AA) *Specification for Aluminum Structures* [1] has always addressed the stability of individual structural members. With regard to beams and columns, the Specification provides equations for determining the strength of beams and columns that account for local buckling of elements such as flanges or webs, and flexural, flexural-torsional, and lateral-torsional buckling of members. Prior to the 2010 Specification, a moment-amplification factor was used to address the P-δ effect, which is the effect of axial load acting on the deflected shape of a member between its ends, on the stability of beam-columns.

Although it addressed the stability of individual members, earlier editions of the Specification have not directly considered the stability of structural systems as a whole. The Specification has never required engineers to design for the P-Δ effect, which is the effect of loads acting on the displaced location of joints in a structure, and only in more recent editions of the Specification was system response included through the use of the effective length concept. As a result, the strength of a structural system designed by previous editions of the Specification can be significantly less than the strength of its weakest member.

With some collapses of aluminum structures attributed to system instability, the AA decided to provide more comprehensive and transparent stability provisions in the 2010 edition of the Specification. Recognizing that accurately determining the effective length of members is complicated by the wide variety of non-orthogonal structural geometries used in aluminum structures, the AA has abandoned the use of effective length. In an effort to be more consistent with other US design specifications, the AA adopted stability provisions similar to those that appear in the 2010 American Institute of Steel Construction's (AISC) *Specification for Structural Steel Buildings* [2]. Because of differences in (1) the stiffness and strength of steel and aluminum, in particular that the E/σ_y ratio for steel is approximately twice that of aluminum, and (2) the manufacturing processes of aluminum profiles and hot-rolled steel sections, a study that includes experimental and analytical components was conducted to confirm the adequacy of adopting the AISC provisions. A summary of this study is presented below.