A Comparison of Canadian, Mexican, and United States Steel Design Standards

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ABSTRACT

The steel design standards for buildings of the three countries of Canada, Mexico, and the United States are compared in this paper. The special emphasis is on the criteria for the stability design of plates, columns, beams, and beam-columns. It is shown that while the theoretical and experimental basis for all three codes is common, the final form of the criteria is not the same: different formulas are used for columns, beams, and beam-columns. Other differences arise from the fact that all three countries use different units. However, the designed proportions of the structural elements and structures are often not significantly different in the final execution. Each code has advantages and disadvantages in its details of design. However, there appears to be no major obstacle to arriving at mutually satisfactory codes which are essentially interchangeable among countries.

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INTRODUCTION

For reasons of public safety the design of structures is regulated by *building codes*. These legal documents are supported by a variety of technical *standards* which provide criteria for materials, loads, design methods, limit states,

Theodore V. Galambos is Emeritus Professor of Structural Engineering, University of Minnesota, Minneapolis, Minnesota. and test methods for the types of construction for which the building codes apply. The evolution of standards is similar in every country, being based on the same underlying scientific and technical principles, but the details, such as arrangement of topics, formulas, and assumptions regarding loads among these documents tend to diverge. This is due to the fact that historic traditions and individual preferences lead to different ways of expressing the same concepts.

In the modern world, engineering endeavors overlap national and continental boundaries, and a structural engineer often needs to operate under a variety of different standards to design the same type of structure. There is, therefore, great importance in comparing different design standards. Such comparisons explain the different ways in which, say, a beam or a column is designed in different countries, and they can point out the common aspects. Such comparisons will eventually lead to design standards in the same regions of the world becoming more alike. The evolution of the Eurocodes in Europe is one such example of consolidation (CEN, 1993). Because of the economic and technical importance of design standards, many previous comparisons have been made in the literature. Some recent studies, for example, are found in the following references: Beedle (1991), Galambos (1996, 1998), Bild and Kulak (1991), Kennedy et al. (1993), Liew et al. (1994), and White and Clarke (1997a and b).

This paper is one of the outcomes of a series of annual meetings of a group called North American Coordination Committee on Structural Steel Design, formed to explore the similarities and differences in steel design standards in Canada, Mexico, and the United States. A companion paper (Ellingwood, 1998) addresses loads, load factors, and the load combinations in the design codes of the three countries, as well as the assumptions underlying their general design criteria. The present paper compares the design standards for steel building structures of the United States, Mexico, and Canada. The three standards are the Load and Resistance Factor Design Specification for Structural Steel Buildings of the American Institute of Steel Construction (AISC, 1993a), Reglamento de Construcciones para el Distrito Federal (the construction regulations of the Federal District of Mexico, RCDF,

1987), and the Canadian Standard CAN/CSA-S16.1-94 *Limit States Design of Steel Structures* (CSA, 1994a).

We will consider first a general comparison of the two English language standards. This will be followed by an examination of the resemblances and differences of the design criteria for compactness, and of beams, columns, beam-columns, and frames. Finally, some general conclusions will be drawn from this study. The Mexico City code is in many ways similar to both the AISC and the CSA standards. Where appropriate, comparisons will be made to the Mexican code.

GENERAL COMPARISON OF THE CANADIAN AND THE AMERICAN STANDARDS

Design standards emerged in the third decade of the Twentieth Century out of the need to define common design methods so that columns designed in Chicago would be designed according to the same formulas as columns designed in New York or elsewhere. These rules represent the minimum requirements for a safe structure, as perceived by the structural engineering profession of the particular country or region. Additional knowledge needed to deal with new problems encountered by industry and the design professionals served as one of the major incentives to the structural engineering research community. The new knowledge then found its way into the structural design standard. For example, the first edition of the AISC Specification in 1923 had 23 small pages in a pocket-sized notebook. The 1993 version of this standard is 143 pages long, not counting the Commentary and the design aids. The CSA current standard is 144 pages long. Similar experiences are encountered also by all other countries which maintain structural design standards. The evolution of the standards we are comparing here is similar: they are being maintained by voluntary expert committees representing industry, researchers, and designers. Changes are made according to essentially the same rules by a consensus process. We are thus looking at "cousins," if not siblings, when we compare these standards.

The similarities and differences in the American and Canadian documents can best be viewed by examining the tables of contents, as reproduced side-by-side in Table 1. An examination of the two lists of principal sections

Table 1 Contents of CSA (1994) and AISC (1993)				
	Canadian Standard		AISC LRFD Specification	
1.	Scope and Application	Α.	General Provisions	
2.	Definitions and Symbols	В.	Design Requirements	
3.	Reference Publications	C.	Frames and Other Structures	
4.	Drawings	D.	Tension Members	
5.	Material: Standards and Identification	Ε.	Columns and Other Compression Members	
6.	Design Requirements	F.	Beams and Other Flexural Members	
7.	Loads and Safety Criterion	G.	Plate Girders	
8.	Analysis of Structure	Н.	Members Under Combined Forces and Torsion	
9.	Design Length of Members	١.	Composite Members	
10.	Slenderness Ratios	J.	Connections, Joints and Fasteners	
11.	Width-Thickness Ratios: Elements in Compression	K.	Concentrated Forces, Ponding and Fatigue	
12.	Gross and Net Areas	L.	Serviceability Design Considerations	
13.	Member and Connection Resistance	М.	Fabrication, Erection, and Quality Control	
14.	Fatigue			
15.	Beams and Girders			
16.	Open Web Steel Joists			
17.	Composite Beams			
18.	Concrete-Filled Hollow Structural Sections			
19.	General Requirements For Built-Up Members			
20.	Stability of Structures and Members			
21.	Connections			
22.	Bolting Details			
23.	Structural Joints Using ASTM A325M, A490M, A325, or A490 Bolts			
24.	Welding			
25.	Column Bases			
26.	Anchor Bolts			
27.	Seismic Design Requirements			
28.	Fabrication			
29.	Cleaning, Surface Preparation, and Painting			
30.	Erection			
31.	Inspection			

indicates that the CSA standard is more inclusive. It contains provisions for the design of open-web steel joists, and for the seismic design of steel structures. However, these subjects are covered under separate standards in the United States: steel joists are under the purview of the Steel Joist Institute (SJI, 1994), and seismic design is covered by a separate AISC standard (AISC, 1997a). In addition, the AISC has other separate provisions for single angle structures (AISC, 1993b) and for structures made from hollow structural sections (AISC, 1997b). If one looks through either of these two sets of standards, one can find all that is needed to perform the design of steel structures and members fabricated from hot-rolled steel plates and shapes. The scope of the two standards is essentially the same. The Canadian document states that it applies to the design of all steel structures except for bridges, antenna structures, offshore structures, and structures made from cold-formed steel. For these there are other CSA standards. The AISC standard says that it is for buildings only, but in reality it is used for all steel designs except for those listed above for the CSA. In fact, the two documents which are the subject of this paper, are the basic "flagship" standards of the steel design community in both countries.

Besides the many similarities there are some notable differences. Many of these will be discussed in the later parts of this paper. One difference, however, concerns the philosophy regarding the role of the Appendices. In AISC (1993a), the Appendices are extensions of specification sections within the body of the document, giving the more complex provisions which would be encountered less frequently by the designer in routine design. The body is thus kept relatively simple. The whole document, including the Appendices, is mandatory. In CSA (1994a) the Appendices serve mainly as a commentary, since only one of the thirteen sections, the requirements for torsional flexural buckling of columns, is mandatory. The others give useful design guides for dealing with floor vibrations, wind sway vibrations, recommended maximum values for deflection under live and wind loads, and other design considerations.

COMPARISON OF SELECTED PROVISIONS

1. Plate Slenderness Limits

Plate slenderness limits are provided in the standards to ensure that the plate elements of the cross section, e.g., the flanges and the web of wide-flange shapes, will not reduce the capacity of the whole member by local plate buckling. In the case of *pure axial compression* of the wide-flange shape, the slenderness limit is introduced to assure that the whole member will yield before local buckling. The respective limits of the American, Canadian, and the Mexican codes are presented in Table 2.

These limits are not much different from each other, except for the flange slenderness limit in the Canadian standard, which is somewhat more restrictive than the other two limits. In the equations the terms are defined as follows:

- $b_f =$ flange width
- h = web depth clear of fillets
- t_f = flange thickness
- t_w = web thickness
- F_y = yield stress
- \vec{E} = modulus of elasticity

The Canadian and Mexican standards define four classes of cross sections in *pure bending:*

- *Class 1 shape*: plastic design permitted. The corresponding classification in the AISC specification is *compact shape*.
- *Class 2 shape*: plastic moment *M_p* can be reached. The corresponding classification in the AISC specification is *compact shape*.
- Class 3 shape: yield moment M_y can be reached. The corresponding classification in the AISC specification is noncompact shape.
- *Class 4 shape*: yield moment cannot be reached; an effective section must be used according to the coldformed design rules in another standard. The corresponding classification in the AISC specification is *slender shape*.

The limiting criteria are shown in Table 3 for the three standards. Slender plate elements will not be compared here since, in the case of the CSA standard (CSA, 1994a), they require the introduction of another standard, that is, the Canadian cold-formed specification (CSA, 1994b). The values in Table 3 show considerable differences between the three standards. The AISC criteria are more liberal than the other two. For the Class 1 cross section, where the development of a plastic mechanism is the limit of structural usefulness, the AISC assumes an inelastic

Slender (Table 2 Slenderness Limits in Pure Compression		
h (F	AISC	CSA	RCDF
$\frac{D_f}{2t_f}\sqrt{\frac{F_y}{E}}$	0.56	0.45	0.58
$\frac{h}{t_w}\sqrt{\frac{F_y}{E}}$	1.49	1.50	1.47

Table 3 Slenderness Limits in Pure Bending				
	Standard	Class 1 cross section	Class 2 cross section	Class 3 cross section
$\frac{b_{\rm f}}{2t_{\rm f}}\sqrt{\frac{F_{\rm y}}{E}}$	AISC CSA RCDF	0.38 (0.31*) 0.32 0.32	0.38 0.38 0.38	** 0.45 0.58
$\frac{h}{t_w}\sqrt{\frac{F_y}{E}}$	AISC CSA RCDF	3.76 (3.05*) 2.46 2.45	3.76 3.80 3.71	5.70 4.25 5.60
* limits fo ** variab	 * limits for seismic design ** variable, depending on the moment capacity (see Figure 1) 			



Fig. 1 Flange slenderness limits for wide-flange members in bending.

rotation of three times the rotation at the attainment of the first plastic hinge. The other two standards implicitly require a greater rotation capacity, as also demanded by AISC for *seismic design*. A graphical representation of the comparisons are given in Figures 1 and 2. The curves in the figures show the relationship between the non-dimensional moment capacity as the ordinate and the plate slenderness ratio limits as the abscissa. Figure 1 depicts the relationships for the flanges of wide-flange shapes, while Figure 2 represents the web limits for these cross sections.

In addition to the more liberal slenderness limits in the AISC specification there is a linear transition from the plastic moment M_p to the yield moment M_y . The other two standards have a step transition. It should be noted that for more slender plate elements the Canadian and the Mexican standard define the Type 4 section with a moment capacity which reduces essentially as the AISC reduction

shown in Figure 1. If one considers CSA and RCDF Class 1 sections as "seismic," and Class 2 as "plastic" sections, the differences almost disappear.

2. Column Curves

The design of columns is made according to the following general design formula:

$$\phi_{\rm c}AF_{cr} \geq P_u$$

where

$$\phi_c$$
 = resistance factor

A =cross-sectional area of column

 F_{cr} = critical stress

 P_u = required factored load

Table 4 presents the individual column formulas from the AISC, CSA, and the RCDF standards.



Fig. 2 Web slenderness limits for wide-flange members in bending.

In the column equations the non-dimensional slenderness ratio is

$$\lambda_{\rm c} = \frac{L}{r} \sqrt{\frac{F_y}{\pi^2 E}}$$

where

- L =length of pinned end column
- r = radius of gyration of the column cross section

 F_{y} = yield stress of the steel

E = modulus of elasticity

The column curves are compared graphically in Figure 3. The abscissa is the slenderness parameter λ_c and the ordinate represents the critical load multiplied by the resistance factor. The CSA employs two equations. The higher curve is for sections which belong in Column Category No. 1 in the Structural Stability Research Council (SSRC) column selection table (Galambos, 1998). The remaining cross sections follow SSRC curve No. 2. Because of this stratification the spread of the testresults is narrower, and thus a resistance factor of 0.90 is justified. The AISC has only one column curve for all

Table 4 Column Equations			
AISC Column Curve	CSA Column Curve	RCDF Column Curve	
$oldsymbol{\phi_c} = 0.85$	$oldsymbol{\phi}_{c} = 0.90$	$\phi_c = 0.90$	
$F_{cr} = 0.658^{\lambda_c^2} F_y$	$F_{cr} = F_{y} \left(1 + \lambda_{c}^{2n} \right)^{\frac{1}{n}}$	$F_{cr} = \frac{F_{y}}{\left(1 + \lambda_{c}^{2n} - 0.15^{2n}\right)^{\frac{1}{n}}}$	
for $\lambda_c \leq 1.5$	n = 2.24, SSRC column curve No. 1, welded H-shapes with flame-cut edges, hollow structural sections (class H)	n = 1.4, most shapes; if $F_y > 345$ MPa (50 ksi), a larger value of <i>n</i> may be used	
$F_{cr} = \left(\frac{0.877}{\lambda_c^2}\right) F_y$	$\underline{n} = 1.34$, SSRC column curve No. 2, all other shapes, Class C H55	$\underline{n} = 1.0$, welded shapes with universal mill plates	
for $\lambda_c \geq 1.5$	$\underline{n} = 0.98$ SSRC column curve No.3 is recommended for welded shapes with universal mill plates in the Commentary		



Fig. 3 Design column strength.

column types, and so a more conservative resistance factor, 0.85, is called for. The AISC column formulas imply an initial crookedness of L/1500 and an end-restraint factor giving an implicit effective length factor of 0.97. The CSA curve is based on a completely pinned-end column and an initial out-of straightness of L/1000. The Mexican code also has two equations. The one applicable to most sections is almost coincident with the second Canadian curve. The second equation applies to welded shapes made from universal mill plates, and it is below the other curves. For most of the practical range of column slenderness, the three curves applicable for most columns are almost coincident.

There are further differences between the two codes in how columns with slender plate-elements are treated. The AISC uses a form-factor (Q-factor), while CSA uses an effective cross section. Torsional and flexural-torsional buckling is handled in an almost identical manner by all three codes.

3. Web Shear Capacity

The shear strength of webs is essentially the same in all three codes. The formulas are presented in Table 5, and the corresponding non-dimensional shear force limit and web slenderness are shown in Figure 4. The Canadian and the Mexican criteria are almost identical. The difference between these two and the AISC code is that the latter has a shear yield stress of $0.6F_{y}$ while the former uses

 $0.66F_y$ for this limit. The criteria given here are for unstiffened webs. Similar conclusions are valid for stiffened webs of plate-girders.

4. Laterally Unsupported Wide-Flange Beams

The formulas for the strength of compact laterally unsupported rolled wide-flange beams are given in Table 6. There is a considerable difference in the format. The CSA/RCDF equations provide a non-linear transition between the elastic buckling formula to the full plastic moment, while the AISC formula for this transition is linear.

The common features of all three standards are as follows: Resistance factor, $\phi = 0.9$

Critical moment:
$$M_{cr} = \frac{\pi}{L_b} \sqrt{EI_y GJ + \left(\frac{\pi E}{L_b}\right)^2 I_y C_w}$$

The formulas for the non-uniform moment amplification factor C_b are different in the three codes, but the results are the same for end-moments applied at the beam ends.

The curves relating the moment capacity and the unbraced slenderness ratio L/r_y are shown for uniform bending in Figure 5 and for the case of one end-moment in Figure 6. The differences between the two code sets are in the inelastic transition range. For uniform moment the AISC curve lies below the CSA/RCDF curve (Figure 5), while for the one end-moment case the situation is

Table 5Web Shear Limit States Stresses Fv			
AISC	CSA	RCDF	
$K_{v} = 5.0$ (buckling coefficient)	$K_{v} = 5.34$	$K_{v} = 5.0$	
$F_v = 0.6F_y$	$F_v = 0.66F_y$	$F_v = 0.66F_y$	
for $\frac{h}{t_w} \leq 2.45 \sqrt{\frac{E}{F_y}}$	for $\frac{h}{t_w} \leq 2.27 \sqrt{\frac{E}{F_y}}$	for $\frac{h}{t_w} \leq 2.19 \sqrt{\frac{E}{F_y}}$	
$F_{v} = \frac{1.47\sqrt{\frac{E}{F_{v}}}}{\frac{h}{t_{w}}} \times F_{y}$	$F_{v} = \frac{1.50\sqrt{\frac{E}{F_{y}}}}{\frac{h}{t_{w}}} \times F_{y}$	$F_{v} = \frac{1.44\sqrt{\frac{E}{F_{y}}}}{\frac{h}{t_{w}}} \times F_{y}$	
for 2.45 $\sqrt{\frac{E}{F_y}} \le \frac{h}{t_w} \le 3.07 \sqrt{\frac{E}{F_y}}$	for 2.27 $\sqrt{\frac{E}{F_y}} \le \frac{h}{t_w} \le 3.21 \sqrt{\frac{E}{F_y}}$	for 2.19 $\sqrt{\frac{E}{F_y}} \le \frac{h}{t_w} \le 3.13\sqrt{\frac{E}{F_y}}$	
$F_{\nu} = \frac{4.25E}{\left(\frac{h}{t_{w}}\right)^{2}}$	$F_{v} = \frac{4.81E}{\left(\frac{h}{t_{w}}\right)^{2}}$	$F_{v} = \frac{4.52E}{\left(\frac{h}{t_{w}}\right)^{2}}$	
for 3.07 $\sqrt{\frac{E}{F_y}} \le \frac{h}{t_w} \le 260$	for 3.21 $\sqrt{\frac{E}{F_y}} \le \frac{h}{t_w} \le 260$	for $3.13\sqrt{\frac{E}{F_y}} \le \frac{h}{t_w} \le 260$	
$h =$ web depth, $t_w =$ web thickness			



Fig. 4 Nominal shear strength (unstiffened webs).

Table 6 Capacity of Laterally Unbraced Beams		
AISC	CSA and RCDF	
for $L_b \leq L_p$: $M_n = M_p = F_y Z_x$ for $L_p \leq L_b \leq L_r$: $M_n = C_b \left[M_p - (M_p - M_r) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p$ for $L_b > L_r$: $M_n = C_b M_{cr}$	$ \begin{array}{l} M_n = M_u \\ M_u = 1.15 M_p \left(1 - \frac{0.28 M_p}{C_b M_{cr}} \right) \leq M_p \\ & \text{when } C_b M_{cr} > \frac{2 M_p}{3} \\ M_u = C_b M_{cr} \\ & \text{when } C_b M_{cr} \leq \frac{2 M_p}{3} \end{array} $	
where : $L_p = 1.76r_y \sqrt{\frac{E}{F_y}}$ $L_r = \frac{r_y X_1}{F_L} \sqrt{1 + \sqrt{1 + X_2 F_L^2}}$ $X_1 = \frac{\pi}{S_x} \sqrt{\frac{EGJA}{2}}$ $X_2 = \frac{4C_w}{l_y} \left(\frac{S_x}{GJ}\right)^2$ $F_L = F_y - F_r$ $F_r = 70$ MPa for rolled shapes, maximum compressive residual stress at flange tips		

reversed (Figure 6). In this latter case the AISC gives credit for the fact that because of localized yielding at the end where the moment is largest, the remainder of the beam is still mainly elastic.

The limits for the bracing spacing to accommodate plastic design are shown in Figure 7. In this case the AISC and the RCDF rules are the same, but the CSA criteria are different. Under moment gradient $(M_1/M_2 > 0.5)$ the CSA rules are much more conservative than the criteria of the other two codes.

5. Beam-Columns

The philosophy behind the beam-column interaction equations of the CSA and the AISC standards is somewhat different. The CSA equations are strictly applicable to the strength of the member, given the axial and bending forces acting on the beam-column, while the AISC interaction curves are also a vehicle that takes care of frame stability, as explained subsequently. The Mexico City code follows essentially the CSA approach. Thus it is sufficient to examine and compare only the AISC and the CSA criteria. For the case of compact (class 2) wide-flange section beam-columns bent about the major axis these are:

AISC Specification:

Interaction equations:

for
$$\frac{P_u}{\phi_c P_n} \ge 0.2$$
 $\frac{P_u}{\phi_c P_n} + \frac{8}{9} \times \frac{M_{ux}}{\phi_b M_{nx}} \le 1.00$
for $\frac{P_u}{\phi_c P_n} \le 0.2$ $\frac{P_u}{2\phi_c P_n} + \frac{M_{ux}}{\phi_b M_{nx}} \le 1.00$



Fig. 5 Beam-buckling curves for rolled beams, uniform moment.

Lateral-torsional buckling, uniform moment

where

- P_u = required compressive strength
- $\phi_c = 0.85$, resistance factor for compression
- $\phi_b = 0.9$, resistance factor for flexure
- P_n = nominal compressive strength of member, determined for the effective length in the plane of the frame if lateral-torsional buckling of the beam-column is prevented by lateral bracing (x-axis buckling), and for the larger of the effective length in the plane of the frame and the actual length of the member, if out-of-plane buckling is not prevented (y-axis buckling). The employment of the x-axis effective length factor, K > 1.0, is the device used to take care of in-plane frame stability.
- M_{ux} = required flexural strength of the beam-column. This moment may be computed using a secondorder frame analysis, or it may be determined

from the approximation

$$M_u = B_1 M_{nt} + B_2 M_{lt}$$

where

- M_{nt} = required flexural strength assuming there is no lateral translation
- M_{lt} = required flexural strength as a result of lateral translation of the frame only

$$B_1 = \frac{C_m}{1 - \frac{P_u}{P_{e1}}} \ge 1.0$$
$$P_{e1} = \frac{\pi^2 E I_x}{I^2}$$

where

 $C_m = 0.6 - 0.4 \frac{M_1}{M_2}$ (for beam-columns with end moments only)

Lateral-torsional buckling, moment at one end only



Fig. 6 Beam-buckling curves, moment at one end only.

$$B_2 = \frac{1}{1 - \sum P_u \left(\frac{\Delta_{oh}}{L \ge H}\right)}$$

where

- E = modulus of elasticity of steel
- I_x = x-axis moment of inertia
- L = length of beam-column
- M_1/M_2 = ratio of the smaller to the larger moment at the ends of the beam-column, positive when the member is bent in reverse curvature
- $\sum P_u$ = required axial strength of all columns in a story
- Δ_{oh} = lateral inter-story deflection
- $\sum H = \text{sum of all story horizontal forces producing} \Delta_{oh}$

In the RCDF the amplification factor B_2 may be used only for regular frames. A second-order analysis is required for non-regular frames. CSA Standard:

Interaction equations:

$$\frac{P_u}{\phi_c P_n} + \frac{0.85U_{1x}M_{ux}}{\phi_b M_{nx}} \le 1.0$$
$$M_{ux} \le \phi_b M_{nx}$$

where

 $\phi_c = \phi_b = 0.90$

- P_u = required axial capacity
- M_{ux} = required flexural capacity. This quantity is determined by either a second-order analysis, or an approximate amplified first-order moment M_{ux} = $M_{nt} + B_2 M_{lt}$. The terms here are the same as those defined for the AISC criteria above. The sway amplification factor B_2 may, however, not exceed 1.4 in the CSA standard, while it is unrestricted in the AISC specification. For frames loaded only by gravity forces, notional



Fig. 7 Lateral bracing limits for plastic design.

lateral loads are to be applied at each story level equal to $0.005 \sum P_u$ for determining M_{lt} .

Three interaction checks are required:

- Case 1. Cross-sectional strength
- Case 2. Overall member strength
- Case 3. Lateral-torsional buckling strength

The remaining parameters of the interaction equations will now be defined for each of the three limit state criteria.

- $P_n = AF_y$ for Case 1 = axial capacity determined for L/r_x for Case 2 (including unbraced frames)
 - = axial capacity determined for L/r_y for Case 3
- $M_{nx} = M_{px}$ for Case 1 and Case 2
 - = flexural capacity for the laterally unbraced member for Case 3

 $U_{1x} = 1.0$ for Case 1

$$= \frac{\omega_1}{1 - \frac{P_u}{P_{e1}}} \text{ for Case 2}$$

$$=\frac{\omega_1}{1-\frac{P_u}{P_{e^1}}}$$
, but never less than 1.0 for Case 3

$$\omega_1 = 0.6 - 0.4 \frac{M_1}{M_2} \ge 0.4$$

The interaction curves of the three codes are compared in Figures 8 and 9. The curves are for beam-columns bent by uniform moment about the x-axis for a compact section of high strength steel. The curves in Figure 8 are for the case of in-plane behavior (lateral bracing is present), and Figure 9 compares the design criteria for the laterally unbraced situation. The comparisons are made for individual beam-columns in a braced frame, that is, the AISC effective length factor K = 1. As can be seen, the differences are not excessive for the member considered here. However, the seemingly small differences may influence the comparative reliability index of the three standards. The AISC limits appear to be somewhat more conservative than the CSA/RCDF limits. The differences between the methods of the American and the Canadian codes are more evident in the treatment of beam-columns in unbraced frames. These differences are:

- The AISC specification uses the effective length factor to account for frame stability, while the CSA standard requires the application of notional lateral forces.
- CSA limits the sway amplification factor to 1.4. This factor is not restricted in AISC.
- The moment amplification factor B_1 is limited to a value larger than 1.0 in AISC, whereas the CSA value of this factor (named U_1 in the equations above) may become less than 1.0.



Fig. 8 Comparison of beam-column interaction curves, in-plane behavior.

The differences between the AISC and the CSA standards for unbraced frames are examined in detail in two papers by White and Clarke (1997a and b). The scope of their investigation also includes Eurocode 3 and the Australian steel design standard. The authors identify the merits and disadvantages associated with each standard, and observe that the philosophies and methods used lead to results that are neither dangerously unconservative nor overly conservative. Acting upon the recommendations of White and Clarke, it is reasonable to expect that a unified approach could be developed that incorporates the best features of each set of design rules.

SUMMARY AND CONCLUSIONS

Comparisons are made in this paper between the strength requirements for steel building structures in the Limit States Design (or LRFD) standards of Canada, the United States, and the Federal District of Mexico. The fundamental concepts and the research background for the three documents are the same. The majority of the strength criteria are either identical or very similar. There are, however, a number of philosophical and operational differences between the codes. Following are some of these:

- *Plate slenderness limits.* The Canadian and the Mexican standards use a step-wise shape slenderness classification, similar to that used in Eurocode 3, while the American code specifies a continuous transition in the inelastic range from the plastic moment to the elastic buckling equation. The latter code avoids abrupt changes from one class of section to the weaker class (see Figures 1 and 2).
- Axially loaded columns. The AISC Specification uses one column curve for the whole spectrum of possible column populations. Because of the larger variability a resistance factor of $\phi_c = 0.85$ is specified. The other two standards provide multiple column curves, thus



Fig. 9 Comparison of beam-column interaction curves, lateral-torsional buckling.

justifying a higher resistance factor $\phi_c = 0.90$ for the smaller population of column types served by each formula (see Figure 3).

- Shear capacity of webs. There is only a slight difference between the codes when the webs are slender, caused by round-off due to the different units used in each of the three standards. The Canadian and the Mexican codes permit a higher nominal shear yield stress, $0.66F_y$, than the $0.60F_y$ in the AISC Specification (see Figure 4).
- Laterally unsupported beams. The transition between the plastic moment and the elastic buckling moment is a linear equation in the AISC and a non-linear equation in the CSA and the RCDF standards. This leads to situations when one or the other equation is more conservative (see Figures 5 and 6). The bracing criteria for plastic design are significantly different between the American and the Mexican codes on the one hand, and the Canadian standard on the other hand (see Figure 7).
- *Frame design*. The beam-column interaction equations contain essentially the same parameters, but the form of the equations is not identical, nor are the axial capacity and flexural capacity exactly the same. This leads to some differences when the beam-column is part of a braced frame (see Figures 8 and 9). The largest differences exist for unbraced frames, where the AISC Specification uses an effective length factor larger than 1.0 to account for frame stability, while the Canadian code employs a notional lateral load for this effect. It is in this point that the greatest conceptual difference exists. The final designs may not be so different, but the intellectual path by which the codes arrive at those results is not identical.

There are, no doubt, other differences which exist among these three North American design codes for steel buildings. It is believed that this discussion has highlighted the most important ones. The purpose of this paper is to inform designers who work across boundaries on the North American continent that (a) differences do exist, and (b) the effects of these differences are not so detrimental as to seriously compromise either safety or economy. The paper is also written so that code-writing organizations can take note of the various ways in which common design problems are treated in the countries of this continent. They thus may be motivated toward more similarity rather than divergence in the development of revised editions of the standards.

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