

COMPARISON OF STABILITY DESIGN CRITERIA AROUND THE WORLD

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FOREWORD

This paper has been written to celebrate the memory of my good friend Professor Otto Halasz. It has been my privilege to befriend him when he was a visiting Fellow during 1964/65 in the United States. Part of his study leave was at the Fritz Engineering Laboratory of Lehigh University in Bethlehem, Pennsylvania, where I was at that time a faculty member. Professor Halasz believed in the unity of scientific knowledge in his field of structural engineering, and he did everything in his power to promote the international dissemination of that knowledge. He also promoted the harmonization of structural design rules. The platforms for the internationalization in our profession were the following organizations: The International Association of Bridge and Structural Engineers, the Structural Stability Research Council, the Council for Tall Buildings and the Urban Habitat, and the European Convention for Steel Construction. Professor Halasz was an enthusiastic contributing member of the first three of these associations. It was my privilege to see him often in the conferences of these groups in diverse places in the world during his all too short life. His vision is reflected in the code developments that are detailed in this paper. He was indeed a great world-class engineer, a fine educator and a wonderful friend.

SUMMARY

This paper presents comparisons of the design criteria for the proportioning of steel columns, beams and beam-columns. Standards, from North America, Australia and Europe are graphically portrayed side-by-side in the five figures in the text. These comparisons show that the world's major structural design standards are close, but not identical. Column design and in-plane beam-column design are in reasonable harmony, but the design of laterally unbraced beams in the inelastic range still lacks worldwide consensus.

1. INTRODUCTION

During the last two decades of the Twentieth Century the structural design standards of many countries and regions of the world underwent a transformation from the *Allowable Stress Design Methods* to *Limit States Design*. This modernization of the codes has given rise to efforts of harmonizing the rules for the design of members that

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are frequently encountered, such as beams, columns and beam-columns. This paper will compare the design criteria for steel members in the various standards.

The following standards were considered: American Institute of Steel Construction (AISC), Architectural Institute of Japan (AIJ), Australian Standard (AS), German standard (DIN), Eurocode 3 (EC3), International Standards Organization (ISO), and National Standard of Canada (CSA). The bibliographic details of these standards are listed in the *References* portion of this paper.

2. COMPARISON OF THE SPECIFICATIONS

The decades of the 1980-s and 1990-s saw the adoption of Limit States Design codes for steel structures that span a region (e. g. the Eurocodes), are shared by nations (e. g. the Canadian Standard S 16 is also adopted in South Africa) and could eventually be used by the whole world (e. g. the ISO Standard). These codes are also very similar to each other. This makes a lot of sense in view of the international scope of structural engineering. All of the above listed standards are modern. Most have been in use for less than a two decades. The oldest limit states code is the Canadian National Standard S 16. It was first adopted in 1974. All have the same basis in probability based reliability methods, and they use *load factors* and *resistance factors* (called *partial factors* in the Eurocode). The standards are based on the theory of structural mechanics, and they have their justification on essentially the same set of experiments. Many of these structural experiments were performed as part of cooperative international research projects. This is not at all surprising. In the area of the stability of steel structures, for example, there has been extensive cooperation between researchers in Europe, the USA, Canada and Australia since the end of the Second World War. This can be observed by comparing the references listed in the basic European document for structural steel stability, *European Recommendations for Steel Construction* (13) and the American *Guide to Stability Design Criteria for Metal Structures* (9). In developing the new codes the specification managers in the individual countries have finally realized that in a world where science, technology and business are truly international, design standards also must be international. This situation was not always so, even as recently as ten years ago. The worldwide rules of stability were reviewed by the author in 1989 (10) and by Beedle in 1991 (4). At that time many codes were still based on the allowable stress design concept, and there was a great deal of variability among the various countries of the world. The author has long been involved in the harmonization of steel design criteria on the North American continent. He has recently compared the stability rules in modern steel design specifications in a series of publications (7, 8, 9), and the present paper is a summary of the conclusions reached from these studies.

2.1. Columns

The harmonization among the world's standards has been most successful in the design criteria for axially loaded columns. This is evident in the curves shown in Figure 1. This curve portrays the relationship between the non-dimensional slenderness ratio λ ,

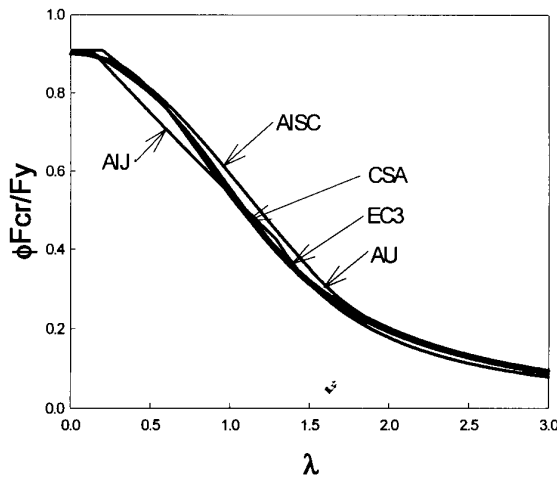
as the abscissa, and the factored critical strength ratio $\phi F_{cr}/F_y$ as the ordinate. The terms are defined as follows:

$$\lambda = \frac{kL}{r\pi} \sqrt{\frac{F_y}{E}}$$

- Where kL - effective column length
 F_y - yield stress of steel
 E - modulus of elasticity of steel
 F_{cr} - critical stress
 R - radius of gyration
 ϕ - resistance factor, same as the reciprocal of the European *partial factor for material*.

The differences between the design capacities of axially loaded columns are seen to be relatively slight, even though the equations on which these curves are based are quite different in their mathematical forms. However, the same parameters are used in them. Thus for this basic stability element a column designed in Budapest would have the same design strength as one designed in Chicago.

Fig. 1 Column Curve Comparisons



2.2. Beams

The stability curves for beams are shown in Figures 2 and 3. These curves depict the variation of the lateral-torsional buckling moment M_{cr} , non-dimensionalized by the plastic moment capacity M_p , with the minor axis slenderness ratio L/r_y for a doubly symmetric wide-flange beam (W690X125) with a yield stress of 345 Mpa. The curves in Figure 2 are for the case where the laterally unsupported length of the simply supported beam is subjected to equal end moments causing uniform bending along the whole length of the member. The curves in Figure 3 represent the strength of laterally unsupported simply supported beams subject to an end moment only at one end. As can be seen, the differences between the various standards are larger than for axially loaded

columns, especially in the case of the beams under moment gradient (Fig.3). This is especially evident when the European curve is compared to the American curve in the slenderness range of 50 to 150, where most practical beams occur. The differences between the various standards are of the following two kinds:

1. The end restraints and loading conditions are treated with more or less completeness in these codes. The Australian standard is by far the most complete in this aspect, having a provision for including the effective length to account for the restraint from adjacent spans. It gives also a very extensive catalogue of different practical loading cases. Other standards are much simpler, using the conservative approach of not counting on end restraint.
2. The greatest divergence is in the way the transition between the elastic buckling moment and the full plastic moment is handled. The American and the Japanese specifications have a straight-line transition between the elastic curve and M_p at a certain value of the minor axis slenderness ratio that defines the limit of elastic buckling behavior. In order to accomplish this three equations are required. The other codes have a built-in imperfection factor that gives a continuous smooth transition, but it always gives a lower critical moment than the elastic buckling moment even when the slenderness ratio is large. Except for the Canadian standard, where two equations are used, the others use only one equation to define buckling capacity. In the case of the Eurocode and ISO the beam equations are the same as the column equations.

2.3. Beam-Columns

The comparisons in Figures 4 and 5 are made for a simply supported beam-column subjected to equal end moments about the major axis of a W200X59. These curves relate the ultimate axial force P_u (nondimensionalized by the squash load P_y) and bending moment M_u (nondimensionalized by the plastic moment M_p). The value of M_u is assumed to include the second-order amplification. The curves are reasonably bunched together in the case of in-plane behavior, where lateral bracing prevents lateral-torsional buckling. (see Fig. 5).

In viewing these comparisons it should be realized that the calculations for the plots do not include resistance factors, i.e., $\phi = \gamma_{mat} = 1.0$. In case of in-plane bending, the criteria of design would give essentially the same beam-column sizes in each country where these rules are used. The situation is not the same for the case where the member is able to buckle laterally (see Fig. 4). Here there are substantial differences in the region of relatively low axial force and high bending moment. This divergence is not due to the form of the interaction equations, but due to the differences in the design rules for beam buckling (compare with Figs. 2 and 3).

Fig. 2 Lateral-Torsional Buckling Curves
Uniform Moment
 W690x125, $F_y = 345$ MPa

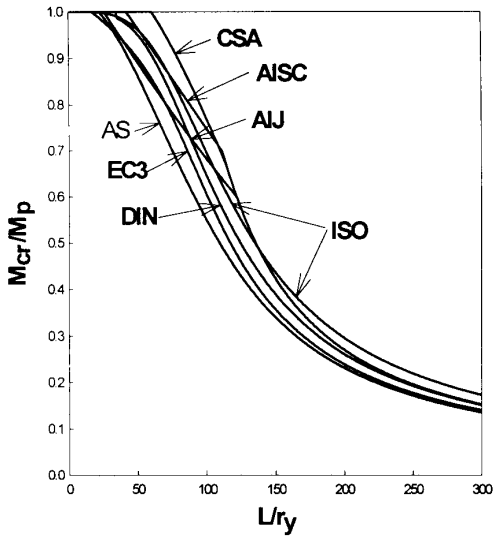


Fig. 3 Lateral-Torsional Buckling Curves
Moment at one end only
 W690x125, $F_y = 345$ MPa

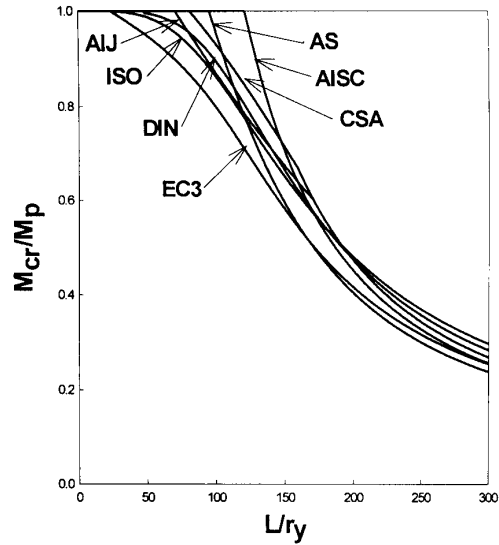


Fig. 4 Lateral-Torsional Buckling Interaction Curves
 $F_y = 345$ MPa, W200X59
 major-axis bending, uniform moment, $L/r_x = 51$

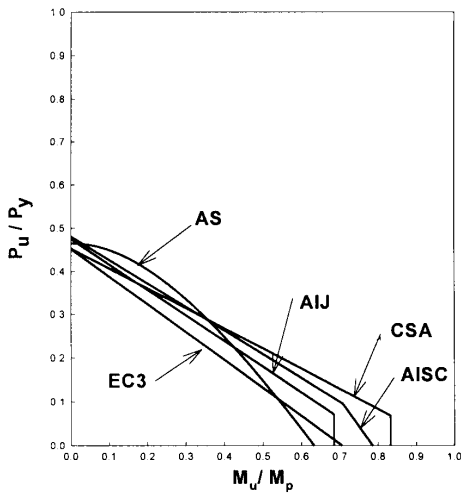
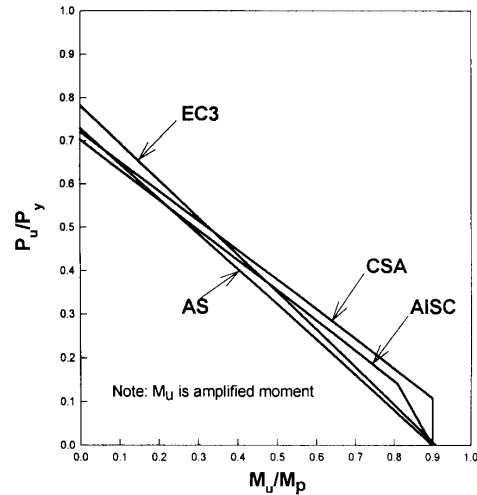


Fig. 5 In-Plane Interaction Curves
 $F_y = 345$ MPa, W200X59
 major axis bending, uniform moment, $L/r_x = 51$



3. SUMMARY AND CONCLUSIONS

The design criteria of some of the world's major modern structural steel codes for determining the design strength of columns, beams and beam-columns were compared in this paper. The conclusions from examining these comparisons have to be tempered by the fact that the loads, load factors, resistance factors (or partial factors), are also different in the various regions. This reflects the different social, economic and scientific conditions in the various parts of the world. The comparisons show that there is slight difference in the case of columns and in-plane beam-columns, but that there is

still a major difference in the philosophy of designing beams against lateral-torsional buckling when buckling occurs in the inelastic range.

Considering that twenty years ago there was a much greater divergence, the present situation is a great improvement over the past. This is mainly due to the internationalization of the structural engineering technical community. Engineers in one country design structures in other countries and continents, and researchers share information in international conferences. In additions, researchers from different countries conduct joint studies and experiments. There is thus a basis of trust and confidence, and a willingness to share. In this regard it is very important to support organizations like the International Standards Organization (ISO) in its efforts to promote a worldwide common framework for structural design.

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Note: the bold lettered acronyms are the designations used in Figures 1-5.