Reliability of the Member Stability Criteria in the 2005 AISC Specification

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Steel building structures in the United States have been designed since 1923 by the American Institute of Steel Construction's *Specification for Structural Steel Buildings* (AISC, 1989). Through the years since then the Specification has gone through several editions, reflecting the advances in the art and science of steel making, design theory, and fabrication practice. The early editions were maintained by committees composed of designers, steel makers, and fabricators. The rules were mainly empirical, generally quite simple, and resulted in safe and conservative designs. The format was the *allowable stress design* (ASD) method, as shown in Equation 1:

$$F_{allowable} \geq f_{calculated} \tag{1}$$

$F_{allowable}$ is the limit state stress, in other words, the yield stress or the buckling stress, divided by a factor of safety, and $f_{calculated}$ is the stress in the member as calculated by first order elastic theory.

The 1961 edition of the AISC Specification brought a number of new features. The committee was enlarged to include academicians who introduced the results of their research into the document. Among many other innovations it was required to consider the effective length of columns. Second-order effects were required to be included for frame design. Inelastic behavior was utilized, and plastic design was permitted. There were two methods of analysis provided: elastic design with the ASD format of Equation 1 and plastic design with the *load factor design* (LFD) format (Equation 2):

$$R \geq \sum_{n} \gamma_i Q_i \tag{2}$$

where

R = plastic mechanism strength of the member or frame

γ_i = load factor for load effect, Q_i, for n load combinations

The load factors were determined by calibration to simply supported beams designed by the ASD method, and they were meant to account for the uncertainties of both the load effects and the resistance.

The next big change occurred in the 1986 edition. This was the first of three editions (1986, 1993, and 1999) in the *load and resistance factor design* (LRFD) (AISC, 1986) format as shown in Equation 3

$$\phi R \geq \sum_{n} \gamma_i Q_i \tag{3}$$

The resistance factor ϕ accounts for the uncertainty of the resistance. The novel feature of these LRFD editions was that the resistance factors and the load factors were determined by a probability-based analysis of the statistical variations of the material properties, the fabrication tolerances, the bias of the analysis, and the various loads acting in combination on the structure. The stochastic analyses were performed in the 1970s, and they relied on data for resistances and load effects available to the researchers at that time (Ellingwood, Galambos, MacGregor, and Cornell, 1980; Ellingwood, MacGregor, Galambos, and Cornell, 1982; Galambos, Ellingwood, MacGregor, and Cornell, 1982; Yura, Galambos, and Ravindra, 1978; Bjorhovde, Galambos, and Ravindra, 1978; Cooper, Galambos, and Ravindra, 1978; Fisher, Galambos, Kulak, and Ravindra, 1978).

A significant change to the AISC *Specification for Structural Steel Buildings*, hereafter referred to as the 2005 AISC Specification, occurred in 2005 (AISC, 2005). Its scope was greatly enlarged, and many novel ideas, such as the provision for advanced analysis, were incorporated into this edition. This paper will not dwell on the many changes but it will discuss the probabilistic reliability of the new code. The format of the design equations in the 2005 AISC

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Specification maintain both the LRFD (Equation 3) and the ASD concept (Equation 4):

$$\frac{R}{\Omega} \geq \sum_{n} Q_i \tag{4}$$

In this equation the resistance is divided by the factor of safety, Ω, that was calibrated to the LRFD method at a live load-to-dead load ratio of 3.0. The formulas for strength used in both the LRFD and the ASD method are identical. The resistance factors, ϕ, in the 2005 Specification are based on reliability analyses that considered new data on the behavior of steel structures and their elements. The load factors are those in SEI/ASCE 7 (SEI/ASCE, 2005). These also have been updated to reflect changes and new insights on loads and load combinations gained since the 1970s.

The purpose of this paper is to review the notional reliability indices of the 2005 AISC Specification, using new information about material properties, new evaluations of the relevant experiments, and new methods of analysis. The paper will examine the reliability of beams, columns, and beam-columns.

RELIABILITY OF BEAMS AND GIRDERS

The 2005 AISC Specification considers a number of aspects of beam and girder design. Definitions for compact, noncompact, and slender plate elements are given in Section B4, *Classification of Sections for Local Buckling*. The stability design of these members is provided for in Chapter F, *Design of Members for Flexure,* and in Chapter G, *Design of Members for Shear*. The remainder of this paper will discuss the probabilistic reliability of these criteria in light of contemporary data on material properties and on a new evaluation of the available full-scale tests on beams and girders.

Reliability Model for Beams

The reliability of steel beams is given here as the *reliability index*, β, (Ellingwood et al., 1980; Ellingwood et al., 1982; Galambos et al., 1982). If the *resistance*, R, and the *load effect*, Q, are assumed to be log-normally distributed, then

$$\beta = \frac{\ln\left[\frac{\bar{R}}{\bar{Q}}\right]}{\sqrt{V_R^2 + V_Q^2}} \tag{5}$$

where

$\bar{R}$ and $\bar{Q}$ = the mean values of the resistance and the load effect, respectively

V_R and V_Q = the corresponding coefficients of variation

The resistance of beams (Yura et al., 1978) is equal to:

$$R = R_n \times M \times F \times P \tag{6}$$

where

R_n = the nominal resistance defined by the formulas in the applicable portions of the 2005 AISC Specification

M = the material factor

F = the fabrication factor

P = the professional factor

M, F, and P are random parameters that characterize the variability of the resistance due to uncertainties of the material properties, the cross-sectional geometry, and the prediction model underlying the theory upon which the formulas of the nominal resistance were based. The mean resistance and its coefficient of variation are given, respectively, by

$$\bar{R} = R_n \times \bar{M} \times \bar{F} \times \bar{P} \text{ and } V_R = \sqrt{V_M^2 + V_F^2 + V_P^2} \tag{7}$$

where

$\bar{M}, \bar{F}, \bar{P}$ = the mean values

V_M, V_F, V_P = the corresponding coefficients of variation

Using a similar first-order reliability approach, the mean and the coefficient of variation of the load effect is

$$\bar{Q} = \bar{D} \times D_n + \bar{L} \times L_n + \bar{S} \times S_n \text{ and}$$
$$V_Q = \frac{\sqrt{\left(D_n \bar{D} V_D\right)^2 + \left(L_n \bar{L} V_L\right)^2 + \left(S_n \bar{S} V_S\right)^2}}{\bar{Q}} \tag{8}$$

where

D_n, L_n, S_n = the nominal dead, live, and snow load effects

$\bar{D}, \bar{L}, \bar{S}$ = the ratios of the respective mean values to the nominal values

V_D, V_L, V_S = the corresponding coefficients of variation

Table 1 gives the data for the load effects (Ellingwood et al., 1982).

The Yield Stress of Steel

Most steel shapes are currently produced to the single material grade specification ASTM A992. This steel exceeds the requirements for A36 and it meets or exceeds A572 Grade 50 steels (Bartlett, Dexter, Graeser, Jelinek, Schmidt, and Galambos, 2003). Extensive experimental studies of these materials were performed at the University of Texas (Jaquess and Frank, 1999) and at the University of Western Ontario and the University of Minnesota (Bartlett et al., 2003). Tensile coupons for these tests were taken from the flanges both in the mill and in the laboratory specimens. Most of

Table 1. Load Effect Statistics		
Type of Load	Mean Load Effect / Nominal Load Effect	Coefficient of Variation
Dead Load	1.05	0.10
Live Load	1.00	0.25
Snow Load	0.82	0.26

Table 2. Resistance Statistics						
Limit State	Type of Applied Load	Number of Tests	Type of Manufacture	*M* Mean (COV)	*F* Mean (COV)	*P* Mean (COV)
Flexure	Uniform Moment	117	Welded Girder	1.06 (0.06)	1.00 (0.05)	1.00 (0.08)
Flexure	Uniform Moment	112	Rolled Beam	1.06 (0.06)	1.00 (0.05)	0.99 (0.06)
Flexure	Moment Gradient	28	Welded Girder	1.06 (0.06)	1.00 (0.05)	1.13 (0.11)
Flexure	Moment Gradient	27	Rolled Beam	1.06 (0.06)	1.00 (0.05)	1.16 (0.12)
Shear	Shear Load	122	Welded Girder	1.10 (0.11)	1.105 (0.013)	1.051 (0.122)

the data were generated from mill test reports. The coupons were tested at the standard ASTM speed, and so they are "dynamic" values. These have to be adjusted for the "static" value that pertains to the actual loading rate on the structure in service. The mill report yield stress values used in this discussion are the data from Bartlett et al. (2003). There are 20,295 data points in the set. The mean dynamic yield stress is 55.8 ksi (385 MPa), and the coefficient of variation is 0.058. The adjustment of the dynamic yield stress to obtain a static value is at best approximate. If the reduction of 4 ksi (28 MPa) that was used in the original development of the AISC LRFD Specification is applied (Galambos and Ravindra, 1978), then the bias of the yield stress is

$$\frac{55.8-4}{50}=1.036$$

Using the reduction of 2.4 ksi, as given by Jaquess and Frank (1999), then the bias is

$$\frac{55.8-2.4}{50}=1.068$$

An approximate average value will be used in this paper: mean, $\bar{M}=1.06$ and coefficient of variation, $V_M = 0.06$.

The Fabrication Factor, *F*

This factor is the ratio of the actual random dimensions of the member to the published or nominal value. The following statistics are taken from previous usage: mean, $\bar{F}=1.0$ and coefficient of variation, $V_F = 0.05$ (Galambos et al., 1982).

The Professional Factor, *P*

This factor is the ratio of the ultimate test strength to the prediction of the maximum capacity of the beam or girder computed for the measured yield stress and measured dimensions of the test specimen according to the design model in the 2005 AISC Specification. In the past several years D.W. White and his colleagues and students have collected and analyzed test data on beams and girders that were reported in the past 50 years (White and Yung, 2004; White and Kim, 2004; White and Barker, 2004). The summary of the resistance statistics is shown in Table 2.

The variation of the resistance factors that were computed by Equation 5 from the data in Table 2 are presented in Figure 1. The curves in Figure 1a show the variation of the reliability index β against the live-to-dead load ratio as this ratio varies from 0 to 5. The nominal resistance, R_n, in Equation 7 is the required value determined from the provisions of Chapter F in the 2005 AISC Specification. The four curves are specific to the type of manufacture, welded girder or rolled beam, and the type of moment variation along the length of the member, uniform moment, or moment gradient. The dip at a live-to-dead load ratio of about 0.2 is due to the intersection of the two load condition, $1.4D$ and

1.2D+1.6L, in the SEI/ASCE 7 (ASCE, 2005) load standard. In the practical range of the live-to-dead load ratio of 0.5 to 5 it is seen that β is fairly constant for each of the categories of beams. The reliability index, β, of rolled beams is slightly higher than β for welded girders, and beams subject to moment gradient have a higher reliability index, β, than beams under uniform moment. This is due to the more pronounced influence of strain hardening in the case of beams loaded by a single concentrated load. The curves in Figure 1b show that the reliability index, β, is slightly higher for the snow and dead load combination than for the live and dead load combination. Figure 1c presents the reliability index, β, for shear panels failing in shear buckling and/or by tension field action. For these tests the material and fabrication factor mean, and coefficient of variation values are different from the values used in the beam tests because data for the actual tests were used.

The curves in Figure 1d show the variation of the reliability index, β, with the live-to-dead-load ratio for rolled beams under uniform moment (data from Table 2) when design is performed by the ASD method (Equation 4) and by the LRFD method. The ASD method is calibrated to have the same reliability index as the LRFD method at a live-to-dead load ratio (L/D) of 3.0, and the curves in Figure 1d show that. For the range of $2 < L/D < 5$ the two approaches give about the same reliability index, except for the case where $L/D < 2$ and ASD has higher βs, indicating a more conservative design, and possibly larger required beam sizes.

From the results of the calculations leading to the curves in Figure 1, it can be seen that the reliability indices in the usual ranges of the dead, live, or snow combinations are above approximately 2.5. This is in the range of the reliability index of 2.6 that was determined in the first 1986 AISC LRFD Specification. Based on the reanalysis of the reliabil-

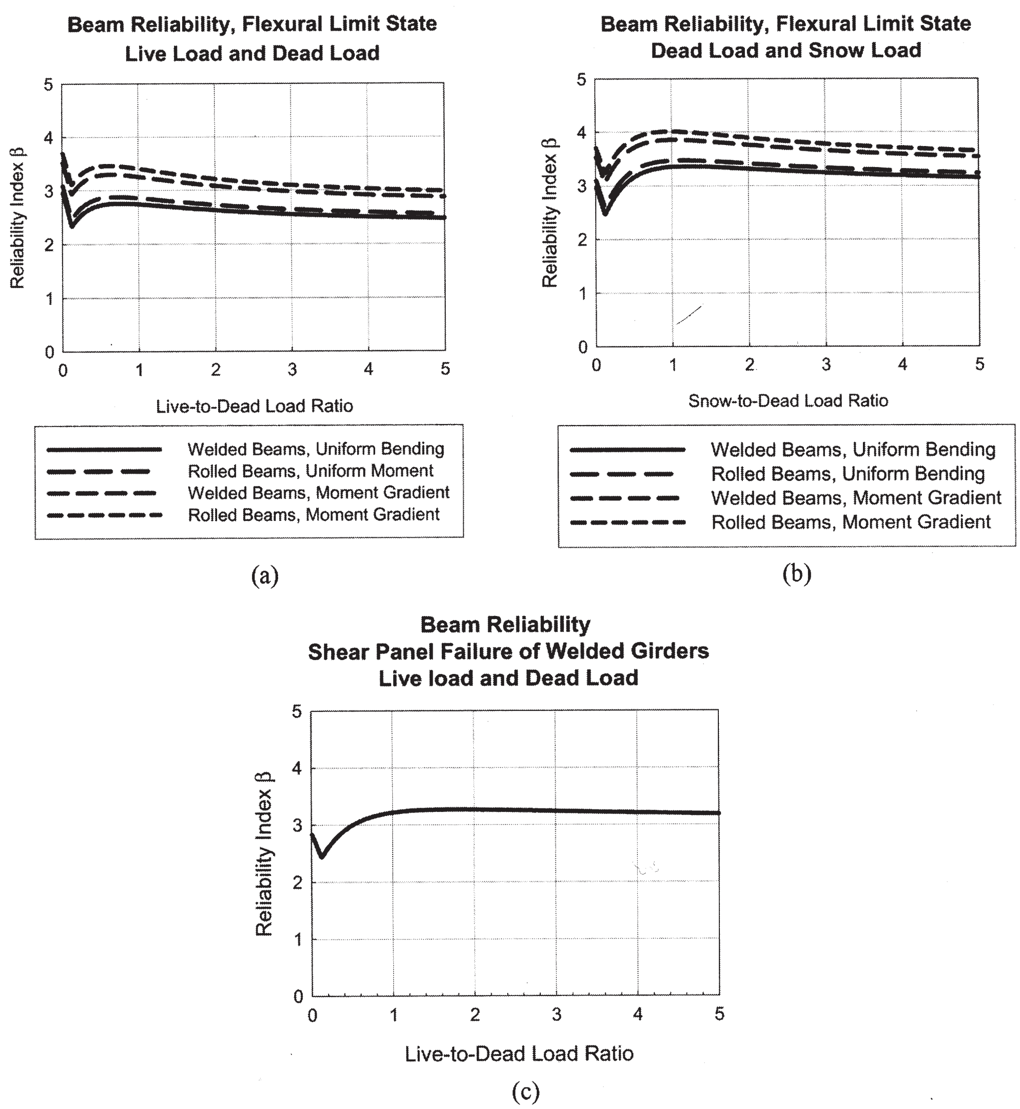

Fig. 1. Beam reliability plots.

Table 3. Column Selection Table (Fig. 3.27 in SSRC Guide)							
Fabrication Details		Axis	Specified Minimum Yield Stress of Steel (ksi)				
			≤ 36	37 to 49	50 to 59	60 to 89	≥ 90
Hot-rolled W-Shapes	Light and Medium W-Shapes	Major Minor	2 2	2 2	1 2	1 1	1 1
	Heavy W-Shapes (flange over 2 in.)	Major Minor	3 3	2 3	2 2	2 2	2 2
Welded Built-up H-Shapes	Flame-cut Plates	Major Minor	2 2	2 2	2 2	1 2	1 1
	Universal Mill Plates	Major Minor	3 3	3 3	2 3	2 2	2 2
Welded Box Shapes	Flame-cut and Universal Mill Plates	Major Minor	2 2	2 2	2 2	1 1	1 1
Square and Rectangular Tubes	Cold-formed	Major Minor	N/A N/A	2 2	2 2	2 2	2 2
	Hot-formed and Cold-formed Heat-treated	Major Minor	1 1	1 1	1 1	1 1	1 1
Circular Tubes	Cold-formed	N/A	2	2	2	2	2
	Hot-formed	N/A	1	1	1	1	1
All Stress-Relieved Shapes		Major and Minor	1	1	1	1	1
Reprinted from the SSRC Guide (Galambos, 1998).							

ity of beams, it is seen that new material and test data do not change significantly the underlying notional safety of steel beams and girders.

RELIABILITY OF AXIALLY LOADED COLUMNS

The statistical data for examining the reliability of axially loaded columns were generated by Bjorhovde (1972). Based on the reported material properties of yield stress and residual stress, and on the geometry of 112 steel column cross sections, Bjorhovde calculated 112 corresponding column curves by numerical integration. A column curve is the relationship between the strength and the length of the pinned-end column. He then grouped these curves into three categories and proposed empirical equations obtained from curve-fitting for the mean of each. He also calculated the standard deviation for the cluster of column curves belonging to each category. The three column curves are shown in Chapter 3 of the Structural Stability Research Council Guide, or SSRC Guide (Galambos, 1998), for sinusoidal initial imperfections of length/1000 and length/1470.

Column category 2 applies to most column cross sections, category 1 is stronger, and category 3 is weaker. Ideally it would have been preferable to specify all three column equations in the AISC Specification. However, the AISC Committee on Specifications chose to prescribe only one column curve in the first LRFD Specification (AISC, 1986) for all column types, and this decision carried through to all subsequent editions, including the 2005 edition. Since this single equation was to be used for the whole spectrum of the 112 column curves, the dispersion is larger, and so a resistance factor $\phi = 0.85$ was prescribed in the 1986, 1993, and 1999 editions of the LRFD Specification.

In the 2005 version the resistance factor was increased to $\phi = 0.90$ for the following reasons: (1) the standard grade of constructional steels is now 450 MPa (50 ksi) and (2) welded built-up columns are no longer made from universal mill plates, but from flame-cut plates. The consequences of these facts can be seen in Table 3. This table is Figure 3.27 in the SSRC Guide (Galambos, 1998; the original reference is Bjorhovde, 1988), and it shows the applicable column curve for a given yield stress and type of fabrication. If heavy H-shapes made from universal mill plate and yield stress values below 50 ksi (450 MPa) are exempted, only category 2 and category 1 columns are left. Conservatively, then, all columns to be covered by the new specification can now be designed as category 2 columns. The AISC column formulas in Chapter E of the 2005 Specification are approximately the same as Bjorhovde's category 2P curve that has an assumed out-of-straightness of length/1470.

Table 4. Statistics of F_{cr}			
$\lambda = \frac{L}{\pi r}\sqrt{\frac{F_y}{E}}$	$\frac{F_{cr}}{F_y}$ for Pinned-End Column	C	$V_{F_{cr}}$
0.3	0.96	1.05	0.02
0.5	0.90	1.08	0.04
0.7	0.82	1.08	0.06
0.9	0.72	1.07	0.08
1.1	0.58	1.09	0.08
1.3	0.46	1.11	0.07
1.5	0.37	1.14	0.06
1.7	0.30	1.13	0.06
1.9	0.24	1.12	0.05

The mean resistance of a column can be expressed as the following equation:

$$\bar{R} = A \times \bar{F}_{cr} \times \bar{M} \times \bar{F} \times \bar{P} \tag{9}$$

The coefficient of variation is then:

$$V_R = \sqrt{V_{F_{cr}}{}^2 + V_M^2 + V_F^2 + V_P^2} \tag{10}$$

The cross-sectional area, A, of the column is the required area calculated from Chapter E, *Design of Members for Compression,* in the 2005 AISC Specification. The material factor, M, and the fabrication factor have already been defined earlier in this paper. The professional factor, P, is the ratio of the test strength to the predicted strength for the measured material and cross-section properties of a column test. The mean and the coefficient of variation for this factor is given by Bjorhovde (1972) as $\bar{P} = 1.03$ and V_P = 0.05 for tests that were conducted with true pinned ends.

The statistics of F_{cr} consist of the category 2 column curve critical stress for the pinned end case with an initial imperfection of length/1470 as determined by Bjorhovde (1972, 1988) multiplied by a factor C, as explained later. The initial imperfection is the mean value from the tests investigated by Bjorhovde (1972). The factor C is the ratio of the strength calculated by numerical integration for an effective length factor k = 0.96, representing the implied end restraint in the AISC column design procedure. The value of k = 0.96 corresponds to the restraint flexibility coefficient of G = 10 recommended in the 2005 AISC Specification Commentary for a nominally pinned end column. The factor C is taken from the paper by Galambos (1983). The statistics of F_{cr} are summed up in Table 4.

All the required data are now presented for the reliability calculation. The reliability index is determined using Equation 5. The results are shown in Figure 2. This plot shows the variation of the reliability index, β, with the slenderness parameter, λ (see Equation 17 for definition of this term), for five different ratios of the live load to the dead load. The reliability index dips down to as low as 2.6, the same as for beams.

RELIABILITY OF BEAM-COLUMNS

The strength determination of steel beam-columns uses the prediction model of an interaction equation that relates the axial force, P, and the end-bending moment, M_o, for a prismatic member. For the purposes of the following reliability analysis, it will be assumed that the moments are equal at each end and they cause single-curvature deflection, as

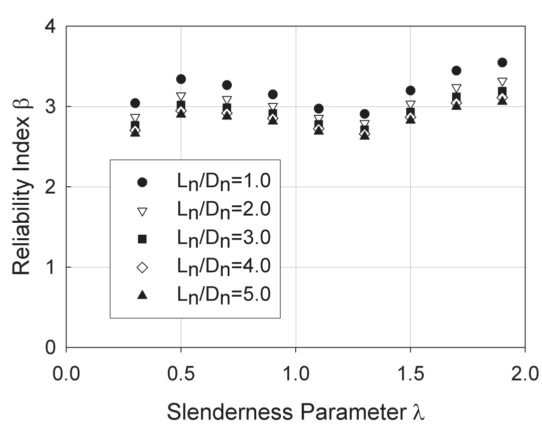

Fig. 2. Reliability indices for axially loaded columns.

shown at the left of Figure 3. Bending is about the major axis of the wide-flange section and lateral-torsional buckling is prevented by bracing. The empirical predictive interaction relationship that will be used is given in the SSRC Guide (Galambos, 1998):

$$\frac{P}{P_{cr}}+\frac{M_o}{M_p\left(1-\frac{P}{P_e}\right)}=1.0 \tag{11}$$

The axial load, P, and the end-moment, M_o, are the values required by the interaction equations in Chapter H of the 2005 AISC Specification. (These interaction equations are not the same as Equation 11.) P_{cr} and P_e are defined here by the Column Research Council (CRC) column formula, and M_p is the plastic moment of the cross section.

$$P_{cr}=\begin{cases} P_y\left(1-0.25\lambda^2\right) \text{ for } \lambda\leq\sqrt{2} \\ \dfrac{P_y}{\lambda^2} \text{ for } \lambda>\sqrt{2} \end{cases} \tag{12}$$

$$P_e=\frac{\pi^2 EI}{L^2} \tag{13}$$

The interaction equation is the prediction model that will be used in the subsequent reliability analysis. The interaction equation is shown schematically in Figure 3 as a dashed line. Also shown by a solid line is the strength prediction calculated by numerical analysis. In Chapter 7 of the first edition of the *Structural Engineering Handbook* (Gaylord and Gaylord, 1968), a convenient tabulation of the numerically calculated strength of wide-flange beam-columns bent about the major axis is given. This will be considered the "exact" theoretical strength. In addition, experimental results are shown as schematic points X in Figure 3. Since the basis of the reliability analysis is the interaction equation (Equation 11), the statistics of the transformation from experiment to "exact" theory, and from "exact" theory to interaction equation must be determined. This is done along lines of equal eccentricity, as illustrated in Figure 3. The applicable beam-column tests were statistically analyzed by Van Kuren and Galambos (1964) and the experimental bias was reported by Galambos (1968) using the following equation:

$$B_1=\frac{\text{Test strength}}{\text{Strength from numerical analysis}} \tag{14}$$

The mean and the coefficient of variation are, respectively, $\bar{B}_1=1.005;\ V_{B_1}=0.093$. The bias factor, B_2, relates the numerical analysis to the empirical relationship of the interaction curve:

$$B_2=\frac{\text{Strength from numerical analysis}}{\text{Strength from interaction equation}} \tag{15}$$

The mean and the coefficient of variation are, respectively, $\bar{B}_2=1.01;\ V_{B_2}=0.04$. The resulting performance function

$$g=B_1B_2-\left(\frac{P}{P_{cr}}\right)-\left[\frac{P\times e}{M_p\left(1-\frac{P}{P_e}\right)}\right]\equiv 0 \tag{16}$$

is analyzed by the second-order reliability method (Nowak and Collins, 2000) to obtain the reliability index, β. Independently, β was also verified by Monte Carlo simulation (Nowak and Collins, 2000). The axial load P in Equation 16 is obtained from the interaction equation in the 2005 AISC Specification. The symbol e is the load eccentricity as defined in Figure 3.

Various results of the analysis are illustrated in Figure 4. The curves in Figure 4a illustrate the effect of the dependence and the independence of the axial force, P, and the end bending moment, M_o, on the reliability index, β. Independence of P and M means that the end moment, M_o, and the axial force, P, are not proportional; that is, the axial force and the bending moment come from different load sources. Full linear dependence means that $M_o = P_e$. For a live-to-dead load ratio of 1.0 and a slenderness parameter of $\lambda = 1.0$, and for the range of the eccentricity ratio ε of 0 to 10, it is seen that β is smaller in value if P and M_o are independent of each other when compared to the case where they are proportional. Total independence is an unlikely occurrence, so the higher value is probably more appropriate.

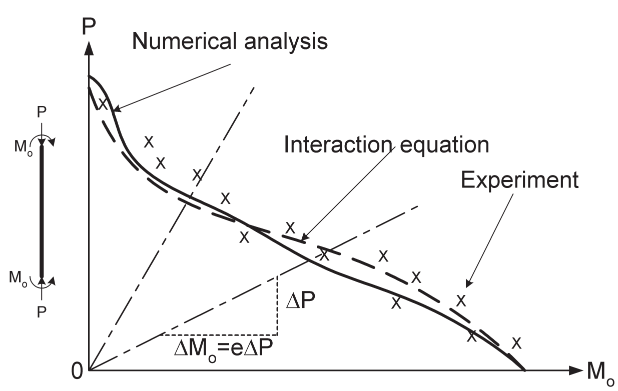

Fig. 3. Schematic of axial force versus moment interaction.

Figure 4b shows the variation of β with λ, while Figure 4c depicts the relationship between β and the live-to-dead load ratio when *P* and *M* are proportional. Finally, Figure 4d illustrates the variation of β with the live load moment-to-dead load moment ratio when *P* and M_o are independent. The following equations define the slenderness parameter, λ, and the eccentricity ratio, ε:

$$\lambda = \frac{L}{\pi r}\sqrt{\frac{F_y}{E}} \text{ and } \varepsilon = \frac{e \times A}{S} \tag{17}$$

In Equation 17, *L* is the length of the member, *r* is its radius of gyration, *A* and *S* are the cross-sectional area and the elastic section modulus, respectively. *E* and F_y are the elastic modulus and the yield stress of the steel, respectively.

It can be observed from the various comparisons in Figure 4 that the reliability index, β, for the beam-column cases investigated varies between approximately 2.5 and 3.7. This range is in line with the values of β found for beams and columns in the previous part of this paper.

SUMMARY AND CONCLUSION

The notional reliability of the 2005 AISC Specification is examined in this paper. First- and second-order reliability methods as well as Monte Carlo simulation were used to calculate the reliability index, β, for beams, columns, and beam-columns. These analyses used new data on material properties for ASTM A992 steel (Bartlett et al., 2003). White and his coworkers evaluated many laboratory tests of the strength of rolled beams and welded girders. The reliability index, β, was obtained as a function of type of load, in other words, snow-versus-live load, live-to-dead load ratio, load proportionality and eccentricity (for beam-columns), type of fabrication (in other words, welded-versus-rolled girders and beams), and column slenderness. The computed reliability indices, β, ranged mostly between 2.5 and 3.5, with the snow load values reaching β = 4.0. This variation is essentially in the same range as the variation of β in the 1986 LRFD Specification, where the reliability index β = 2.6 is the value reported in the Commentary (AISC, 1986) for compact beams under uniform moment.

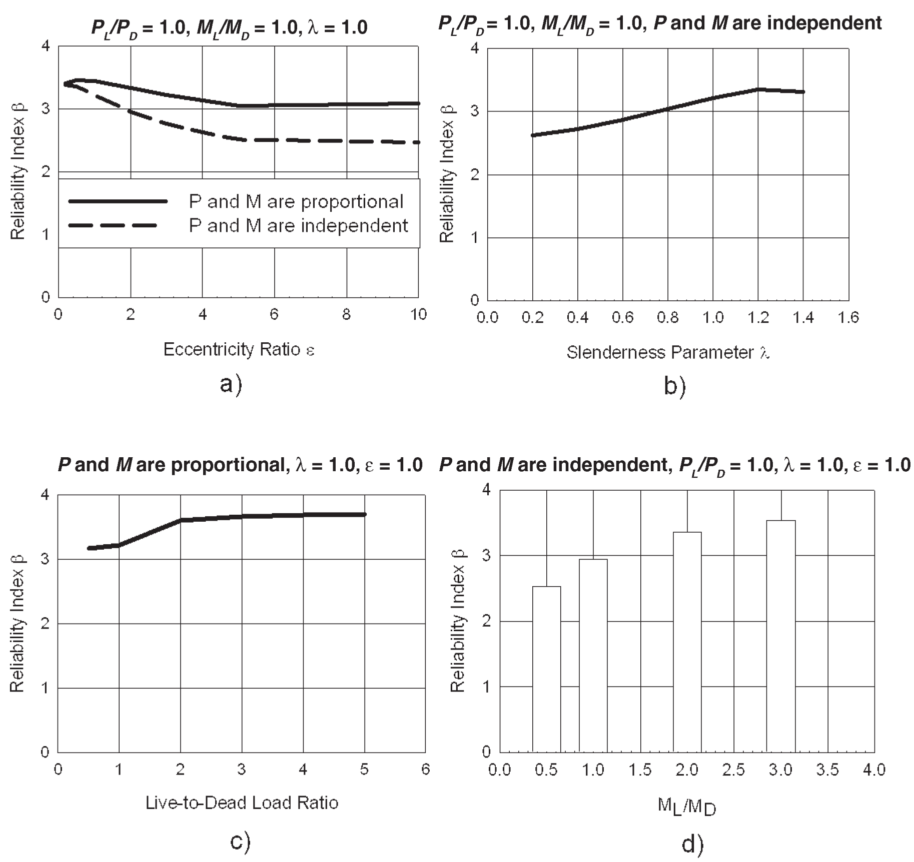

Fig. 4. Reliability of beam-columns.

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