Oct 23rd, 12:00 AM

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Tenth International Specialty Conference on Cold-formed Steel Structures
St. Louis, Missouri, U.S.A., October 23-24, 1990

ASCE LRFD METHOD FOR STAINLESS STEEL STRUCTURES

by

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I. INTRODUCTION

Cold-formed stainless steel sections have been increasingly used in architectural and structural applications in recent years due to their superior corrosion resistance, ease of maintenance, and attractive appearance. The current specification for the design of cold-formed stainless steel structural members and connections was published in 1974 (Ref. 1) by American Iron and Steel Institute (AISI). This design specification was based on the allowable stress design (ASD) method.

Recently, the probability-based load and resistance factor design (LRFD) criteria have been successfully applied to the design of hot-rolled steel shapes and built-up members (Ref. 2). The AISI LRFD specification is being developed as well for the design of structural members cold-formed from carbon and low alloy steels (Ref. 3). This design approach is based on the "Limit States Design" philosophy, which is related to the ultimate strength and serviceability of structural members and connections. In this method, separate load and resistance factors are applied to specified loads and nominal resistances to ensure that the probability of reaching a limit state is acceptably small.

The LRFD criteria were developed on the basis of the first order probabilistic theory, for which only the mean value and coefficient of variation of variables are specified. These random variables involved in the design reflect the uncertainties in mechanical properties of materials, load effects, design assumptions, and fabrication. Because the LRFD method includes probabilistic consideration for uncertain variables in the design formula, it can provide a more uniform overall safety and reliability for structural design.

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Due to the significant differences in material properties between carbon steels and stainless steels, the aforementioned LRFD specifications included in References 2 and 3 do not apply to the design of stainless steel structural members. In order to develop the LRFD criteria for cold-formed stainless steel structural members, a research project has been conducted since 1986 at the University of Missouri-Rolla under the sponsorship of American Society of Civil Engineers (ASCE). Based on the updated ASD specification for cold-formed stainless steel structural members (Refs. 4, 5), the proposed LRFD specification with commentary (Ref. 6) has been prepared for the consideration of the ASCE. This paper presents the background information for developing the LRFD criteria for cold-formed stainless steel structural members and connections.

II. PROCEDURES FOR DEVELOPING LRFD CRITERIA

The theoretical basis of the probability-based design approach has long been established and can be found in numerous references (Refs. 7 - 10). Basically, the model of failure probability is used to determine the risk of failure of structures The safety index, β, derived from the probability of failure is used as a relative measure of the safety of design. The model of the failure probability is expressed on the basis of the first order probabilistic theory.

A. Format of LRFD Criteria

The structural safety based on the LRFD is achieved by the probabilistic theory instead of the engineering judgement. Separate resistance and load factors are to be applied to nominal resistances and specified loads, respectively, to ensure that a limit state is not violated. The use of multiple load factors provides a refinement in design, which accounts for the different degree of uncertainties and variabilities of various design parameters.

The load and resistance factor design criteria for the combination of dead and live loads can be expressed in the following equation:

$$\phi R_n \geq \gamma_D c_D D_C + \gamma_L c_L L_C \tag{1}$$

The right side of the equation represents the effects of a combination of dead load, D_C, and live load, L_C; whereas, the left side relates to the nominal resistance, R_n, of a structural member. The resistance factor ϕ accounts for the uncertainties and variabilities inherent in the R_n, and it is usually less than unity. The load effects γ_D and γ_L are associated with the dead and live loads, respectively. The load factors are greater than unity. The values of c_D and c_L are deterministic influence coefficients, which transform the load intensities to load effects.

B. Probabilistic Basis

Structural safety is a function of the resistance, R, of the structure as well as of the load effects, Q. It is assumed that the resistance, R, and the load effects, are random variables because of the uncertainties associated with the inherent randomnesses. If these uncertainties are specified in terms of the probability density functions (probability distributions), then the measure of risk is the event of the probability of the failure, $P_F(R - Q \leq 0)$.

To calculate the probability of failure, one requires knowledge of the distribution curves of variables R and Q. Although the correct distributions of R and Q are not known, it is convenient to prescribe the distribution of (R/Q) to be log-normal. Due to the fact that the probability distribution of R/Q is not practically known, the mean value and coefficient of variation of variables R and Q are used as the estimated values. Based on this probability distribution and the first order probabilistic theory (Ref. 7), the safety index or "reliability index" can be expressed as follows:

$$\beta = \frac{\ln(R_m/Q_m)}{\sqrt{V_R^2 + V_Q^2}} \qquad (2)$$

in which R_m and Q_m are mean values of the resistance of the structure and the load effects, respectively, and V_R and V_Q are their corresponding coefficients of variation. The index β is a relative measure of the safety of design. The higher the safety index, the smaller the probability of failure.

C. Resistance

The randomness of the resistance R of a structural element is due to the variabilities inherent in the mechanical properties of the material, the variations in dimensions, and the uncertainties in the design theory of member strength. The mean resistance of a structural member, R_m, is defined as follows:

$$R_m = R_n \, (M_m \, F_m \, P_m) \qquad (3)$$

in which R_n is the nominal resistance of the structural elements, and M, F, and P are dimensional random variables reflecting the uncertainties in material properties (i.e., F_y, F_u, etc.), the geometry of the cross-section (i.e., S_e, A, etc.), and the design assumptions, respectively. The subscript of m stands for the mean value of the variables.

Based on the statistical analysis of mechanical properties for stainless steels reported in Ref. 5, the following mean values and coefficients of variation are recommended for the material factor, M, for structural members and connections using austenitic and ferritic stainless steels.

For yield strength of stainless steels

$$(F_y)_m = 1.10\ F_y, \qquad V_{F_y} = 0.10$$

For ultimate strength of stainless steels

$$(F_u)_m = 1.10\ F_u, \qquad V_{F_u} = 0.05$$

The fabrication factor F is a random variable which accounts for the uncertainties caused by initial imperfections, tolerances, and variations of geometric properties. The following values are recommended for the fabrication factor in the design of cold-formed stainless steel structural members and connections.

For stainless steel members and bolted connections

$$F_m = 1.00, \qquad V_F = 0.05$$

For stainless steel welded connections

$$F_m = 1.00, \qquad V_F = 0.15$$

These values were also used in the development of the AISC LRFD criteria for hot-rolled steel structural members and connections (Ref. 10).

The professional factor P is also a random variable reflecting the uncertainties in the determination of the resistance. These uncertainties are included by the use of approximations in the simplification and idealization of complicated design formulas. The professional factor is determined by comparing the tested failure loads and the predicted ultimate loads calculated from the selected design provisions. In this study, the factor P is determined from the ratios of the tested loads to predicted values for the available test data.

By using the first order probabilistic theory and assuming that there is no correlation between M, F, and P, the coefficient of variation of resistance, V_R, can be expressed as

$$V_R = \sqrt{V_M^2 + V_F^2 + V_P^2} \tag{4}$$

in which V_M, V_F, and V_P are coefficients of variation of the random variables M, F, and P, respectively.

D. Load and Load Effects

The major load combination considered in this study is the dead load plus the maximum live load. This load combination governs the design in many practical situations and it is a particularly important case.

The mean load effect, Q_m, for a combination of dead and live loads is assumed as follows:

$$Q_m = c_D C_m D_m + c_L B_m L_m \qquad (5)$$

in which c_D and c_L are deterministic influence coefficients, B and C are random variables reflecting the uncertainties in the transformation of loads into the load effects, and D and L are random variables representing the dead and live load intensities. The subscript of m stands for mean value of variable.

If it is assumed that $B_m = C_m = 1.0$ and $c_D = c_L = c$, the mean value and coefficient of variation of load effects can be expressed as follows:

$$Q_m = c\ (D_m + L_m) \qquad (6)$$

and

$$V_Q = \frac{\sqrt{(D_m V_D)^2 + (L_m V_L)^2}}{D_m + L_m} \qquad (7)$$

where V_D and V_L are the coefficients of variation for dead and live loads.

Load statistics have been studied and reported by Ellingwood et al in Ref. 11, in which $D_m = 1.05D_n$, $V_D = 0.1$, $L_m = L_n$, and $V_L = 0.25$. The same publication indicates that the mean live load intensity can be taken as the code live load intensity if the tributary area is small enough so that no live load reduction is required. Substitution of the load statistics into Eqs. (6) and (7) gives

$$Q_m = c(1.05\ D_n/L_n + 1)\ L_n \qquad (8)$$

and

$$V_Q = \frac{\sqrt{(1.05D_n/L_n)^2\ V_D^2 + V_L^2}}{(1.05D_n/L_n + 1)} \qquad (9)$$

It can be seen that, in Eqs. (8) and (9), the values of Q_m and V_Q depend on the dead-to-live load ratio. Previous research reported in Refs. 12 and 13 indicated that cold-formed members typically have relatively small D_n/L_n ratios. For the purpose of determining the reliability of the LRFD criteria for cold-formed stainless steel structural members, the dead-to-live load ratio is assumed to be 1/5, and so that $V_Q = 0.21$.

E. Determination of Resistance Factors

The values of the reliability index β vary considerably with different kinds of loading, the different types of construction, and the different types of members for a given material design specification. In order to achieve a more consistent reliability, it was suggested in Ref. 14 that the following values of β_o would provide this improved consistency while at the same time give, on the average, essentially the same design by the new LRFD method as is obtained by current design for all materials of construction. These target reliabilities for use in the ANSI Code (Ref. 15) are:

For basic case: Gravity loadings, $\beta_o = 3.0$

For connections: $\beta_o = 4.5$

For wind loading: $\beta_o = 2.5$

Previous research on LRFD criteria for cold-formed carbon steel members indicated that the target reliability index β_o may be taken as 2.5. A higher target reliability index of $\beta_o = 3.5$ was recommended for connections using cold-formed carbon steels. However, these target values may not be applicable for the design of cold-formed stainless steel structures because relatively higher safety factors have been used for cold-formed stainless steel ASD specification. In order to maintain the consistency of structural safety used for cold-formed stainless steels, two target values of 3.0 and 4.0 are used in this study for members and connections, respectively.

In this study, the resistance factor, ϕ, are determined for the load combination of $1.2D_n+1.6L_n$ as used in Ref. 13 for cold-formed carbon steels. By using this load combination, the expression for the load and resistance factor design given in Eq. (1) can be written as follows:

$$\phi R_n \geq c(1.2D_n + 1.6L_n) \tag{10}$$

By assuming $D_n/L_n = 1/5$, the mean values of resistance and load effect can be written as follows:

$$R_m = 1.84\ (cL_n/\ \phi)\ M_m F_m P_m \tag{11}$$

and

$$Q_m = 1.21\ cL_n \tag{12}$$

Therefore, by using the ratio of R_m/Q_m and Eq. (2), the resistance factor, ϕ, can be computed as follows:

$$\phi = \frac{1.521\ M_m F_m P_m}{\exp(\ \beta \sqrt{V_R^2 + V_Q^2}\ )} \tag{13}$$

Equation (13) is to be used for the calibration of various design provisions for members and connections. With the available statistical data on the aforementioned variables, the resistance factor can be computed by selecting a proper target safety index.

III. DEVELOPMENT OF THE LRFD CRITERIA

In this section, the determination of resistance factors for use in the LRFD criteria is discussed. Previous research results obtained from Cornell University (Refs. 16 - 18) and other institutions (Refs. 19, 20) related to the experimental studies of cold-formed stainless steel members and connections have been used for calibrating the design provisions. In this process, the mean values and coefficients of variation of the professional factors were obtained from the ratios of the tested loads to predicted loads. By using the selected factors and target safety index, the resistance factor can be determined accordingly.

A. Tension Members

The tension member is designed as a structural member to carry a uniformly distributed stress in tension and its nominal strength can be reasonably predicted by the following equation:

$$R_n = A_n F_y \tag{14}$$

in which A_n is the net area of the cross section, and F_y is the yield strength of stainless steels. Due to the lack of test data for cold-formed stainless steel tension members, Eq. (14) is used for the calibration of this design provision. By using $M_m = 1.10$, $F_m = 1.0$, and assuming $P_m = 1.0$, the mean value of R_n is

$$R_m = (1.10)(1.0)(1.0)\ R_n \tag{15}$$

The coefficient of variation V_R is obtained by applying $V_M = 0.1$, $V_F = 0.05$, and $V_P = 0$ as follows:

$$V_R = \sqrt{V_M^2 + V_F^2 + V_P^2} = 0.11 \tag{16}$$

Based on a target safety index of $\beta_o = 3.0$ and the value of $V_Q = 0.21$, the resistance factor ϕ is calculated by Eq. (13) as follows:

$$\phi = \frac{1.521(1.1)(1.0)(1.0)}{\exp\left(3.0\sqrt{0.11^2+0.21^2}\right)} = 0.82 \tag{17}$$

For the design of cold-formed stainless steel tension members, a resistance factor of 0.85 is recommended.

B. Flexural Members

In the design of cold-formed stainless steel flexural members, due consideration should be given to the moment-resisting capacity and the stiffness of the member. The moment-resisting capacity of flexural members may be limited by yielding, local buckling, or lateral buckling of the beam. If local buckling and lateral buckling are prevented, the maximum bending capacity is usually determined by the yield moment. Local buckling may occur in the compression flange of the beam and the web of the beam when the compressive stress reaches the critical buckling stress. However, it may not fail due to the postbuckling strength. If the members are laterally supported at a relatively large interval, lateral buckling strength may govern the design.

The web of beams should also be checked for shear, web crippling, and combinations thereof. The maximum shear strength of beam webs is based on shear yielding or shear buckling. For beam webs having small h/t ratios, the shear yield stress can be determined by the von Mises yield theory. For relatively large h/t ratios, the shear strength of beam webs is governed by elastic shear buckling. Inelastic shear buckling is taken into account by using a plasticity reduction factor (Ref. 21). In the design of cold-formed stainless steel beams, due consideration should also be given to web crippling. This type of failure may occur in the web of beams under the concentrated loads or at the supports. For combination bending and shear, combined bending and web crippling, shear lag effect, and flange curling, Reference 22 provides detailed information.

Due to the lack of test data, the calibration of the design requirements for flexural members deals only with the sectional bending strength of beams. The sectional bending strength of beams can be calculated either on the basis of the initiation of yielding or on the basis of the inelastic reserve capacity as applicable. For bending strength based on initiation of yielding, the nominal strength R_n is determined on the basis of the effective cross section and the specified minimum yield strength, i.e., $R_n = S_e F_y$. For the design consideration of inelastic reserve capacity, Reference 6 provides detailed discussions.

Based on a total of 17 beam tests, the ratios of tested to predicted moments are used to calculate the professional factor. These values are given as $P_m = 1.189$ and $V_P = 0.061$. Together with the aforementioned material and fabrication factors, i.e, $M_m = 1.1$, $V_M = 0.10$, $F_m = 1.0$, and $V_F = 0.05$, the resistance factor can be computed by Eq. (13).

The relationship between the safety index, resistance factor, and the ratio of D_n/L_n for stainless steel beams subjected to bending is shown in Figure 1. From this figure, it can be seen that based on the ratio of $D_n/L_n = 0.2$, the computed safety index is 3.04 if the value of the resistance factor is taken as 0.95. The safety indices computed for other ϕ values are also given in Figure 1. Based on the selected target safety index of 3.0 for beam members, a resistance factor of 0.95 is recommended for cold-formed stainless steel beams subjected to flexural bending.

C. Concentrically Loaded Compression Members

Cold-formed sections are made of thin materials, and in many cases the shear center does not coincide with the centroid of the section. Therefore in the design of such compression members, consideration should depend on the shape of the cross section, thickness of material, and the stiffness of the compression members.

For short columns, yielding and local buckling are the usual modes of failure. The overall instability caused by elastic flexural buckling is often a mode of failure for long columns. Compression members having moderate slenderness ratios usually fail inelastic flexural buckling or torsional-flexural buckling. For some cases, the column strength may be limited by the interaction between local buckling of individual elements and overall buckling of columns.

The nominal axial load for compression members is determined by the following formula:

$$P_n = A_e F_n \tag{18}$$

in which A_e is the effective area calculated at the stress F_n, and F_n is the least value of flexural buckling, torsional buckling, and torsional-flexural buckling stresses. For determining the buckling stress in the inelastic range, the tangent modulus obtained from the modified Ramberg-Osgood equation is used in this study. Reference 6 provides detailed design requirements for columns.

Based on the available test data on cold-formed stainless steel compression members, the design provisions for concentrically loaded compression members were calibrated and reported in Ref. 5. In this paper, the result from the calibration for columns subject to flexural buckling and torsional-flexural buckling is presented. The test results were compared to the predicted values for the appropriate failure mode.

The ratios of the tested to predicted failure loads are used as the professional factor. The material factor and fabrication factor used in this study are $M_m = 1.1$, $V_M = 0.10$, $F_m = 1.0$, and $V_F = 0.05$. Using the formula given in Section II of this paper, the safety index and its resistance factor can be determined readily.

A total of 29 tests were calibrated for compression members subject to flexural buckling. The mean value of ratios of P_{test}/P_{pred} is 1.194, and its coefficient of variation is 0.114. The relationship between the safety index and resistance factor was studied and reported in Ref. 5. It indicated that for $D_n/L_n = 0.2$, a safety index of 3.26 can be achieved if the resistance factor is taken as 0.85. This resistance factor of $\phi = 0.85$ is also used in the LRFD criteria for cold-formed carbon steel sections (Ref. 13) and hot-rolled shapes (Ref. 2).

The experimental work on torsional-flexural buckling strength of cold-formed stainless steel columns has been studied in Ref. 20. These test results were compared with the predicted values given in Ref. 6. Based on a total of 45 tests, the mean value of the professional factor, P_m, is 1.111, and its coefficient of variation, V_P, is 0.074. Reference 5 provides a detailed discussion for the determination of resistance factor. Figure 2 shows the relationship between the safety index, resistance factor, and the ratio of D_n/L_n for stainless steel columns subject to torsional-flexural buckling. From this figure, it can be seen that a safety index of 3.17 can be achieved for $D_n/L_n = 0.2$ if the resistance factor of 0.85 is used. This resistance factor was determined on the basis of a load combination of $1.2D_n + 1.6L_n$.

D. Welded Connections

Based on the reevaluation of the test results, the design provisions for welded connections have been developed and are included in Ref. 6. The welded connections should be designed to transmit the maximum load in connected members. Proper regard should be given to eccentricity. The test results of welded connections obtained from previous Cornell research program (Refs. 18 and 23) and Ref. 24 were studied to calibrate the design provisions for groove welds in butt joints, longitudinal fillet welds, and transverse fillet welds. The resistance factors obtained from this investigation were reported in Ref. 5. A target safety index of 4.0 was used for the calibration of cold-formed stainless steel welded connections.

A total of 43 butt-joint welds were collected from the previous experimental work. The mean value of the tested to predicted failure strengths is $P_m = 1.113$, and its coefficient of variation, V_P, is 0.084. This value is considered to be the professional factor. The material factor and fabrication factor used in this study are taken as $M_m = 1.10$, $V_M = 0.05$, $F_m = 1.0$, and $V_F = 0.15$. By using these factors, the safety index can be computed for a specified resistance factor and a ratio of D_n/L_n. Figure 3 illustrates the variation of safety indices with respect to the ratio of D_n/L_n for using groove welds. It indicated that by using a resistance factor of 0.6, the computed safety index for $D_n/L_n = 0.2$ is equal to 4.13, which is larger than the target value ($\beta_o = 4.0$).

For longitudinal and transverse fillet welds, a total of 10 connection tests reported in Ref. 18 were used in this study. Based on the results of calibration, it was found that a resistance factor of 0.55 can be used for the LRFD criteria to prevent both sheet metal and weld metal failures of longitudinal fillet welds. For transverse fillet welds, two resistance factors of 0.55 and 0.65 are recommended for the LRFD criteria against plate and weld metal failures, respectively. These resistance factors were determined on the basis of $D_n/L_n = 0.2$ and with the computed safety indices larger than the target value.

E. Bolted Connections

Previous Cornell test results (Ref. 18) indicated that the failure modes of bolted connections in cold-formed stainless steel construction are similar to that in cold-formed carbon steel construction because of the thinness of the connected parts. Four fundamental types of failure mode were observed and described as follows: Type I - longitudinal shearing of the sheet along two parallel lines, Type II - bearing or piling up of material in front of bolt, Type III - tearing of the sheet in the net section, and Type IV - shearing of the bolt. The calibration of design provisions for shear failure in connected parts, bearing, and tension failure of bolted connections has been investigated and reported in Ref. 5. The design provision for shear and tension failure in bolts was not calibrated due to the lack of test data.

The professional factor used in this study was obtained from the comparison of the tested loads to predicted values. The material factor and fabrication factor used for bolted connections were taken as $M_m = 1.10$, $V_M = 0.05$, $F_m = 1.0$, and $V_F = 0.05$. Using these values and the computed professional factors, the safety index and corresponding resistance factor can be determined by using the formula given in Section II of this paper.

Table 1 shows the results of calibration for cold-formed stainless steel bolted connections subject to shear, bearing, and tension failures. These resistance factors determined for $D_n/L_n = 0.2$ can provide a safety index which is larger than the target value of 4.0.

Table 1

Computed Safety Index β and Resistance Factor φ

for Bolted Connections

Failure Mode	Computed Safety Index β for $D_n/L_n = 0.2$	Resistance Factor φ
Type I - Shear Failure in Connected Parts	4.10	0.70
Type II - Bearing Failure	4.14	0.65
Type III - Tension Failure in Connected Parts	4.04	0.70

F. Local Distorsion

When local distorsions in structural members under nominal service loads must be limited, the design strength is determined on the basis of the permissible compressive stress for stiffened and unstiffened compression elements and the cross-sectional properties of full, unreduced cross section. The resistance factor used for determining the design strength due to local distortion is taken as 1.0. Reference 6 provides detailed discussion on this subject. This design provision is considered to be necessary for stainless steel structural members because of its low proportional limits and due to the fact that more attention is often given to the appearance of exposed surfaces of stainless steel used for architectural purposes.

IV. SUMMARY AND CONCLUSIONS

The probability-based LRFD criteria for the design of cold-formed stainless steel structural members and connections have been developed on the basis of the first order probabilistic theory. The resistance factors have been determined by calibrating the appropriate design provisions as reported in Ref. 5. These design criteria have been based on a target safety index of 3.0 for structural members and 4.0 for connections. This paper presents a brief discussion of the reasoning behind, and the justification for, various provisions. In view of the fact that the resistance factors were obtained from the calibrations of various design provisions on the basis of a limited number of test data, additional tests are needed to refine the resistance factors achieved in this study.

V. ACKNOWLEDGMENTS

This project was sponsored by American Society of Civil Engineers. The financial assistance provided by the Chromium Centre in South Africa, the Nickel Development Institute in Canada, and the Specialty Steel Industry of the United States is gratefully acknowledged. Special thanks are extended to members of the ASCE Steering Committee (Dr. Ivan M. Viest, Mr. Don S. Wolford, and Mr. John P. Ziemianski), Mr. Edwin Jones and Mr. Ashvin A. Shah of the American Society of Civil Engineers, Dr. W. K. Armitage of the Chromium Centre, and Mr. Johannes P. Schade of the Nickel Development Institute for their technical guidance.

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APPENDIX II. NOTATION

The following symbols are used in this paper:

A	= Area of the full, unreduced cross section
A_n	= Net area of cross section
B	= Random variable reflecting the uncertainties in the transformation of live loads into live load effects
C	= Random variable reflecting the uncertainties in the transformation of dead loads into dead load effects
c_D, c_L	= Deterministic influence coefficients translating load intensities to load effects; subscripts D and L denote dead and live loads, respectively
D	= Random variable characterizing dead load
D_c	= Specified dead load intensity
D_n	= Specified dead load
F	= Random variable representing uncertainties in fabrication
F_n	= Nominal buckling stress
F_u	= Tensile strength of the connected sheet in the longitudinal direction
F_y	= Yield strength
L_c	= Specified live load intensity
L_n	= Nominal specified live load
M	= Random variable characterizing the uncertainties in material strength
P	= Random variable reflecting the uncertainties in design assumptions
P_F	= Probability of failure
P_n	= Nominal axial strength of member
P_{pred}	= Predicted failure load
P_{test}	= Tested failure load
Q	= Load effect
R	= Member resistance
R_n	= Nominal resistance of a structure member
S_e	= Effective section modulus of reduced section
V_x	= Coefficient of variation of random variable x; V denotes the coefficient of variation
$(x)_m$	= Mean value of random variable x; subscript m denotes mean value
	= Safety index
	= Target safety index
$_D$	= Dead load factor
$_L$	= Live load factor
	= Resistance factor

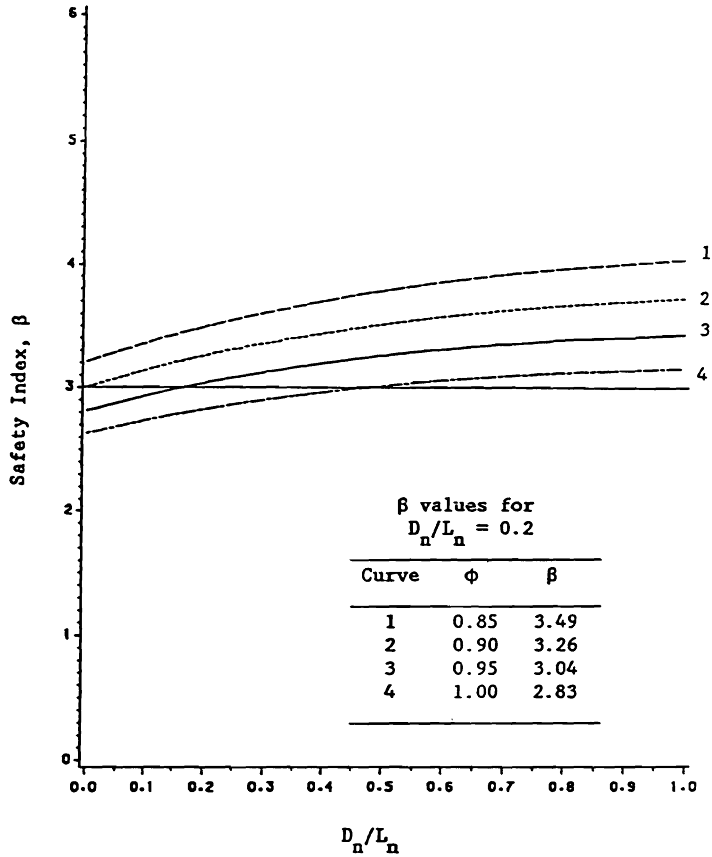

Figure 1 Safety Indices, β, for Different Resistance Factors, φ, and D_n/L_n Ratios for Stainless Steel Beams

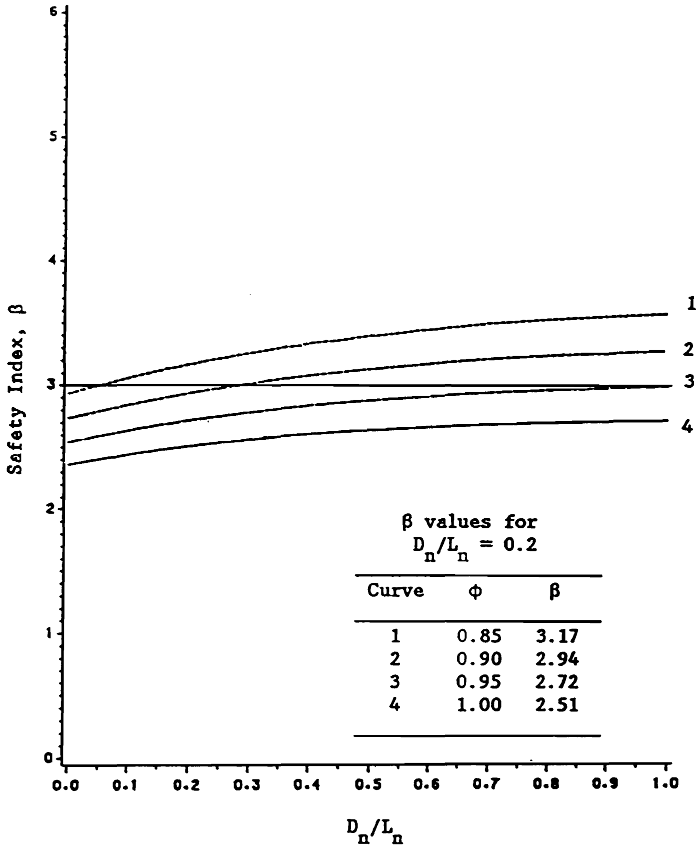

Figure 2 Safety Indices, β, for Different Resistance Factors, ϕ, and D_n/L_n Ratios for Stainless Steel Columns Subjected to Torsional-Flexural Buckling

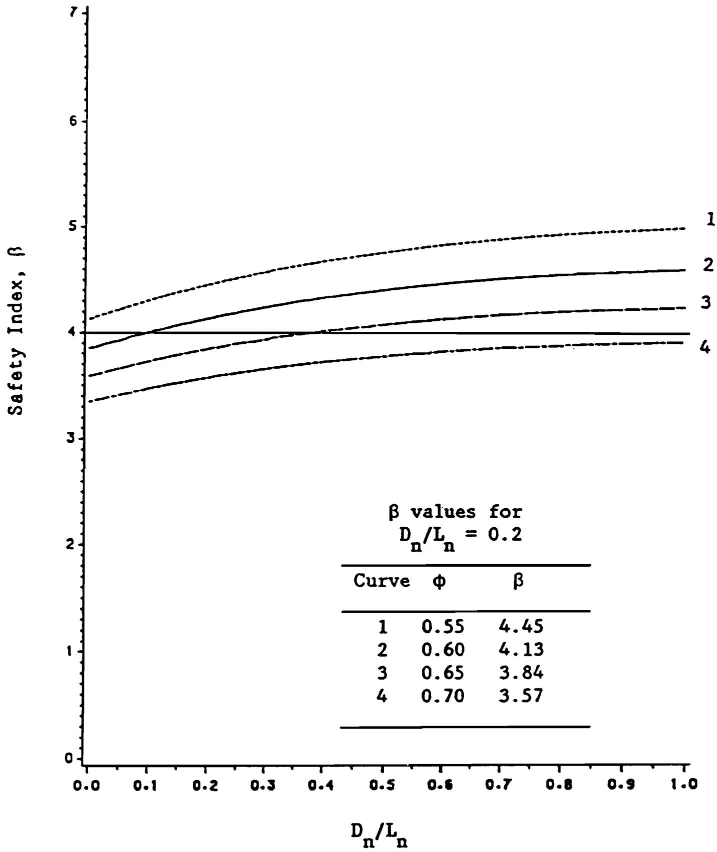

Figure 3 Safety Indices, β, for Different Resistance Factors, φ, and D_n/L_n Ratios for Groove Welds