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ASCE LRFD METHOD FOR STAINLESS STEEL STRUCTURES

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ABSTRACT: In recent years, probability-based load-and-resistance-factor-design (LRFD) method has been successfully applied to the design of hot-rolled steel sections and cold-formed steel members in the United States and foreign countries. In order to develop the LRFD criteria for the design of cold-formed stainless steel structural members and connections, a research project was conducted at the University of Missouri-Rolla since 1986 under the sponsorship of the American Society of Civil Engineers (ASCE). This newly developed LRFD Specification with Commentary has been adopted by ASCE as a new standard in 1990. It supersedes the 1974 edition of the Specification for the Design of Cold-Formed Stainless Steel Structural Members issued by the American Iron and Steel Institute. The basic theory of probability-based design approach and the development of the ASCE LRFD criteria for cold-formed stainless steel structural members are presented in this paper.

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 (*Stainless* 1974) by the American Iron and Steel Institute (AISI). This design specification was based on the allowable stress design (ASD) method.

Recently, 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 (*Manual* 1986). Also, an AISI LRFD specification has been developed for the design of cold-formed structural members from carbon and low alloy steels (Hsiao et al. 1989). The LRFD design approach based on a "limit states design" philosophy considers directly 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 first-order probabilistic theory, for which only the mean value and coefficient of variation of random variables are specified. The 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 proba-

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bilistic consideration for uncertain types in the design formulas, it can provide a more uniform overall safety and reliability for structural design.

Due to the significant differences in material properties between carbon steels and stainless steels, the aforementioned LRFD specifications (*Manual* 1986; Hsiao et al. 1989) 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 (Lin et al. 1988a, 1988b), the ASCE LRFD specification with commentary (Lin et al. 1989) has been prepared. It was subsequently reviewed and approved by the ASCE Stainless Steel Cold-Formed Sections Standards Committee ("Specification" 1990). This paper presents the background information associated with the development of LRFD criteria for cold-formed stainless steel structural members and connections.

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 (Ang and Cornell 1974; Ellingwood and Ang 1974; Ravindra et al. 1974; Ravindra and Galambos 1978). 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 a design. The model of the failure probability is expressed on the basis of first-order probabilistic theory.

Format of LRFD Criteria

The structural safety factor based on the LRFD is achieved by the probabilistic theory. Separate resistance and load factors are applied to nominal resistances and specified loads, respectively, to ensure that a strength limit state is not violated. The use of multiple load and resistance 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 is expressed in the following equation:

$$\phi R_n \geq \gamma_D c_D D_C + \gamma_L c_L L_C \quad \text{(1)}$$

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 R_n, and it is usually less than unity. The load factors 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.

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 and the load effects are random variables because of the uncertainties associated with their inherent randomness. If these uncertainties are specified in terms of the probability density functions (i.e., the probability distributions), then a measure of risk is the probability of 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 actual distributions of R and Q are not known, it is convenient to prescribe the distribution of $\ln(R/Q)$ to be normal. Due to the fact that the probability distribution of R/Q is also not known, the mean value and coefficient of variation of the variables R and Q are estimated. Based on the probability distribution and first-order probabilistic theory (Ang and Cornell 1974), the safety index or *reliability index* is expressed as follows:

$$\beta = \frac{\ln\left(\frac{R_m}{Q_m}\right)}{\sqrt{V_R^2 + V_Q^2}} \quad \text{(2)}$$

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

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 used to express the member strength. The mean resistance of a structural member, R_m, is defined as follows:

$$R_m = R_n(M_m)(F_m)(P_m) \quad \text{(3)}$$

in which R_n = the nominal resistance of the structural elements, and M, F, and P = 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 (Lin et al. 1988b), 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 the yield strength:

$$(F_y)_m = 1.10F_y \quad \text{(4a)}$$

$$V_{F_y} = 0.10 \quad \text{(4b)}$$

For the ultimate strength:

$$(F_u)_m = 1.10F_u \quad \text{(5a)}$$

$$V_{F_u} = 0.05 \quad \text{(5b)}$$

The fabrication factor F = a random variable that accounts for the uncertainties associated with initial imperfections 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 members and bolted connections:

$$F_m = 1.00 \qquad (6a)$$

$$V_F = 0.05 \qquad (6b)$$

For welded connections:

$$F_m = 1.00 \qquad (7a)$$

$$V_F = 0.15 \qquad (7b)$$

These values were also used in the development of the AISC LRFD criteria for hot-rolled steel structural members and connections (Ravindra and Galambos 1978).

The professional factor P is also a random variable reflecting the uncertainties in the determination of the resistances. These uncertainties are associated with 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 computed 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. These values are given for the different design criteria later.

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

$$V_R = \sqrt{V_M^2 + V_F^2 + V_P^2} \qquad (8)$$

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

Load and Load Effects

The major load combination considered in this study for the purpose of calibration is the dead load plus the maximum live load. This was also the basis of the AISI LRFD Specification for cold-formed carbon steel sections (Hsiao et al. 1988). This load combination governs the design in many practical situations and thus it is a particularly important case.

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

$$Q_m = c_D C_m D_m + c_L B_m L_m \qquad (9)$$

in which c_D and c_L = deterministic influence coefficients, B and C = random variables reflecting the uncertainties in the transformation of loads into the load effects, and D and L = 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 the load effects can be expressed as follows (Ellingwood et al. 1980):

$$Q_m = c(D_m + L_m) \qquad (10)$$

and

$$V_Q = \frac{\sqrt{(D_m V_D)^2 + (L_m V_L)^2}}{D_m + L_m} \qquad (11)$$

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

Load statistics have been studied and reported (Ellingwood et al. 1980), in which $D_m = 1.05D_n$, $V_D = 0.1$, $L_m = L_n$, and $V_L = 0.25$. The same reference 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 live-load reduction is not required. Substitution of the load statistics into (10) and (11) gives

$$Q_m = c\left(1.05\frac{D_n}{L_n} + 1\right)L_n \qquad (12)$$

and

$$V_Q = \frac{\sqrt{\left(1.05\frac{D_n}{L_N}\right)^2 V_D^2 + V_L^2}}{\left(1.05\frac{D_n}{L_n} + 1\right)} \qquad (13)$$

It can be seen that, in (12) and (13), the values of Q_m and V_Q depend on the dead-to-live-load ratio. Previous research (Supornsilaphachai 1980; Hsiao et al. 1988) indicates that cold-formed members typically have relatively small D_m/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, which is a reasonable value for cold-formed stainless steel structures, and so that $V_Q = 0.21$. The influence of the D_n/L_n on safety index is illustrated in Figs. 1, 2, and 3.

Determination of Resistance Factors

In allowable stress design, the values of the reliability index β vary considerably with different kinds of loading, different types of construction, and different types of members for a given material design specification.

Previous research on LRFD criteria for cold-formed carbon steel members indicated that the target reliability index β_0 may be taken as 2.5. A higher target reliability index of 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 higher safety factors have been used for the cold-formed stainless steel ASD specification. In order to maintain the consistency of structural safety used for cold-formed stainless steels in the previous specification (*Stainless* 1974), the target values of 3.0 and 4.0 are used in this study for stainless steel members and connections, respectively.

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

$$\phi R_n \geq c(1.2D_n + 1.6L_n) \qquad (14)$$

By assuming $D_n/L_n = 1/5$, the mean values of resistance and load effect are:

$$R_m = 1.84\left(\frac{cL_n}{\phi}\right)M_mF_mP_m \qquad (15)$$

and

$$Q_m = 1.21cL_n \qquad (16)$$

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

$$\phi = \frac{1.521M_mF_mP_m}{\exp(\beta\sqrt{V_R^2 + V_Q^2})} \qquad (17)$$

Eq. (17) is 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.

DEVELOPMENT OF 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 (Johnson 1966; Wang 1969; Errera et al. 1970) and other institutions (Van der Merwe and Van den Berg 1987; Van den Berg and Van der Merwe 1988) 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.

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:

$$R_n = A_nF_y \qquad (18)$$

in which A_n = the net area of the cross section, and F_y = the yield strength of stainless steels. Due to the lack of test data for cold-formed stainless steel tension members, (18) 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

$$R_m = (1.10)(1.0)(1.0)R_n \qquad (19)$$

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

$$V_R = \sqrt{V_M^2 + V_F^2 + V_P^2} = 0.11 \qquad (20)$$

Based on a target safety index of $\beta_0 = 3.0$ and the value of $V_Q = 0.21$, the resistance factor ϕ is calculated by (17) as follows:

$$\phi = \frac{1.521(1.1)(1.0)(1.0)}{\exp(3.0\sqrt{0.11^2 + 0.21^2})} = 0.82 \qquad (21)$$

Therefore, for the design of cold-formed stainless steel tension members, a resistance factor of 0.85 is recommended for the limit state of yielding.

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 and the web of the beam when the compressive stress reaches the critical stress. However, the member may not fail due to postbuckling strength. If a member is laterally supported at a relatively large spacing, 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 and shear buckling. For beam webs with small h/t ratios, the shear yield stress can be determined by 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 (Bleich 1952). In the design of cold-formed stainless steel beams, due consideration should also be given to web crippling, shear lag effects, and flange curling; detailed information is provided in Yu (1991).

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 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, detailed discussions are provided in Lin et al. (1989).

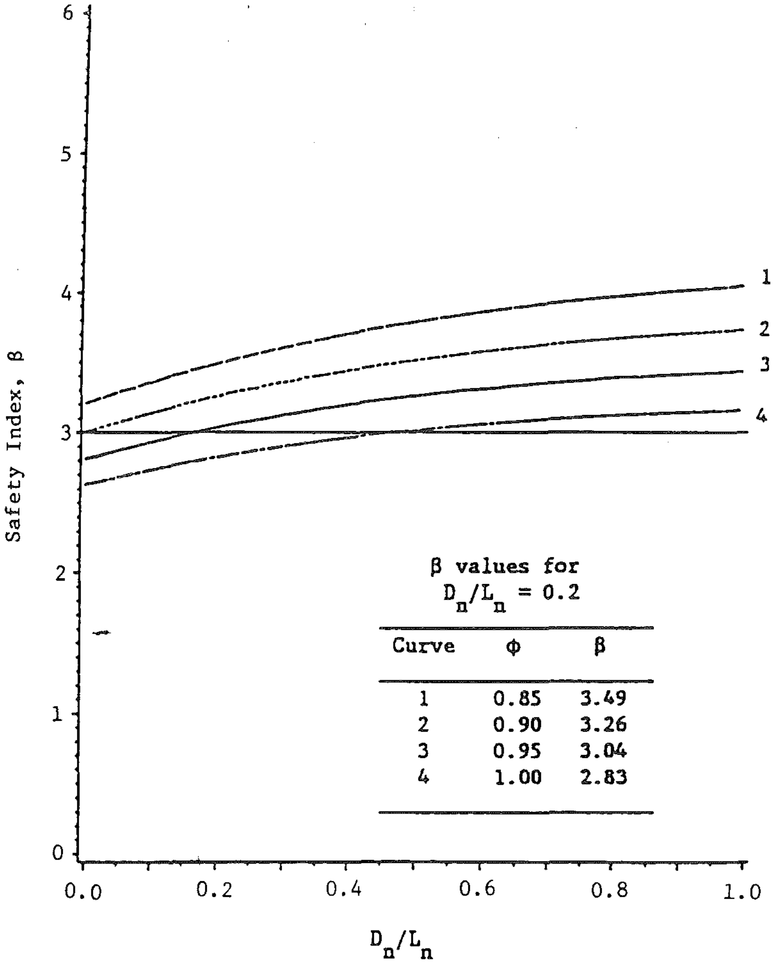
Based on a total of 17 beam tests (Lin et al. 1988b), the ratios of test to predicted moments are used to calculate the professional factor. These values are given as $P_m = 1.189$ and $V_p = 0.061$. Together with the aforementioned material and fabrication factors, i.e., $M_m = 1.1$, $V_M = 0.10$, $F_m = 1.0$, and $V_F = 0.05$, the resistance factor is computed by (17).

The various relationships between the safety index, the resistance factor, and the ratio of D_n/L_n for stainless steel beams subjected to bending are shown in Fig. 1 on the basis of (17). 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 Fig. 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 bending.

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, the shape of the cross section, the thickness of material, and the stiffness of the compression members should be considered.

For short columns, yielding and local buckling are the usual modes of failure. Overall instability caused by elastic flexural buckling is often the mode of failure for long columns. Compression members having moderate



Curve	φ	β
1	0.85	3.49
2	0.90	3.26
3	0.95	3.04
4	1.00	2.83

FIG. 1. Safety Indexes, β, for Different Resistance Factors, φ, and D_n/L_n Ratios for Stainless Steel Beams

slenderness ratios usually fail by 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:

$$P_n = A_e F_n \quad \text{(22)}$$

in which A_e = the effective area calculated at the stress F_n, and F_n = 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 was used in this study. Detailed design requirements for columns are provided in Lin et al. (1989).

The design provisions for concentrically loaded compression members were calibrated based on the available test data and cold-formed stainless steel compression members (Lin et al. 1988). In this paper, the results from the calibration of columns subjected to flexural buckling and torsional-flexural buckling are 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 earlier in this paper, the safety index and the corresponding resistance factor can be determined readily.

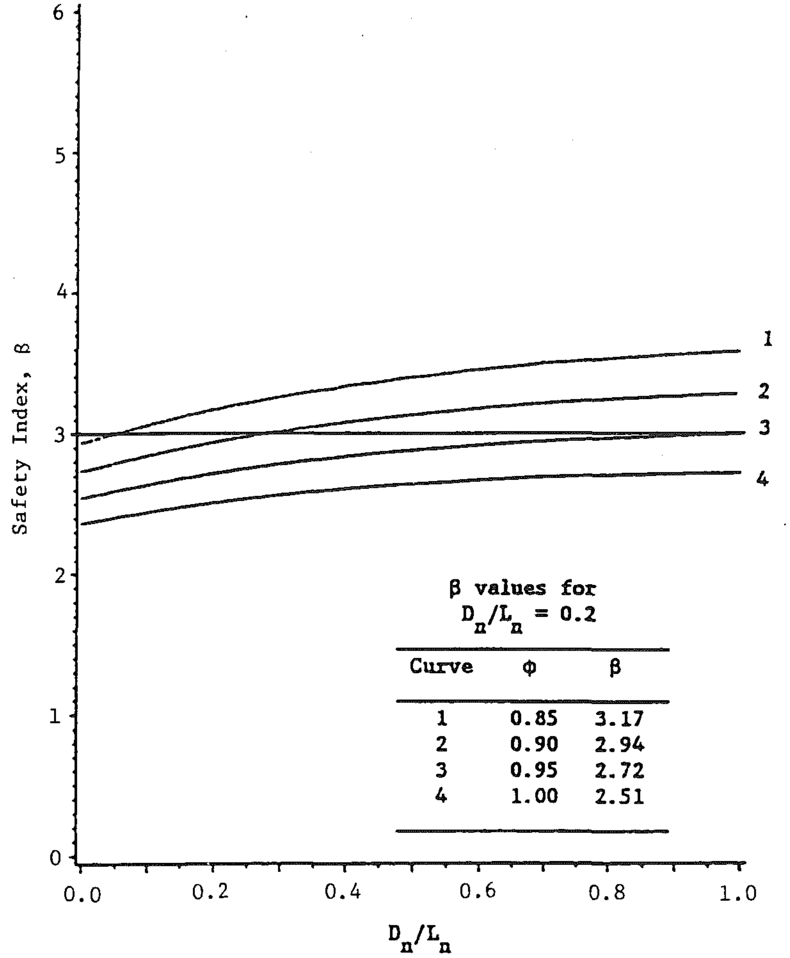
A total of 29 tests (Lin et al. 1988b) were calibrated for compression members subjected to flexural buckling. The mean value of ratios of P_{test}/P_{pred} is 1.194, and its coefficient of variation is 0.114. The relationship between the safety index and resistance factor was studied using (13) (Lin et al. 1988b). This study 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. A resistance factor of $\phi = 0.85$ is also used in the LRFD criteria for cold-formed carbon steel sections (Hsiao et al. 1988) and hot-rolled shapes (*Manual* 1986).

The experimental work on torsional-flexural buckling strength of cold-formed stainless steel columns has been studied by Van den Berg and Van der Merwe (1988). These test results were compared with the predicted values (Lin et al. 1989). Based on a total of 45 tests, the mean value of the professional factor, P_m, is 1.11, and its coefficient of variation, V_P, is 0.074. The discussion for determining the resistance factor was provided earlier in this paper. Fig. 2 shows the relationship between the safety index, the resistance factor, and D_n/L_n ratios as defined by (17) for cold-formed stainless steel columns subjected 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.

Welded Connections

Based on a reevaluation of the test results, the design provisions for welded connections have been developed (Lin et al. 1989). 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 (Errera et al. 1970; Errera et al. 1974) and other research (Flannery 1968) were used 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 are provided in the research report (Lin et al. 1988b). A target safety index of 4.0 was used for the calibration of cold-formed stainless steel welded connections.

The data for a total of 43 butt-joint welds were collected from previous experimental work. The mean value of the tested-to-predicted strength ratios 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 and fabrication factors 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 load ratio D_n/L_n. Fig. 3 illustrates the variation of safety indexes with respect to the ratio D_n/L_n according to (13)



β values for $D_n/L_n = 0.2$

Curve	φ	β
1	0.85	3.17
2	0.90	2.94
3	0.95	2.72
4	1.00	2.51

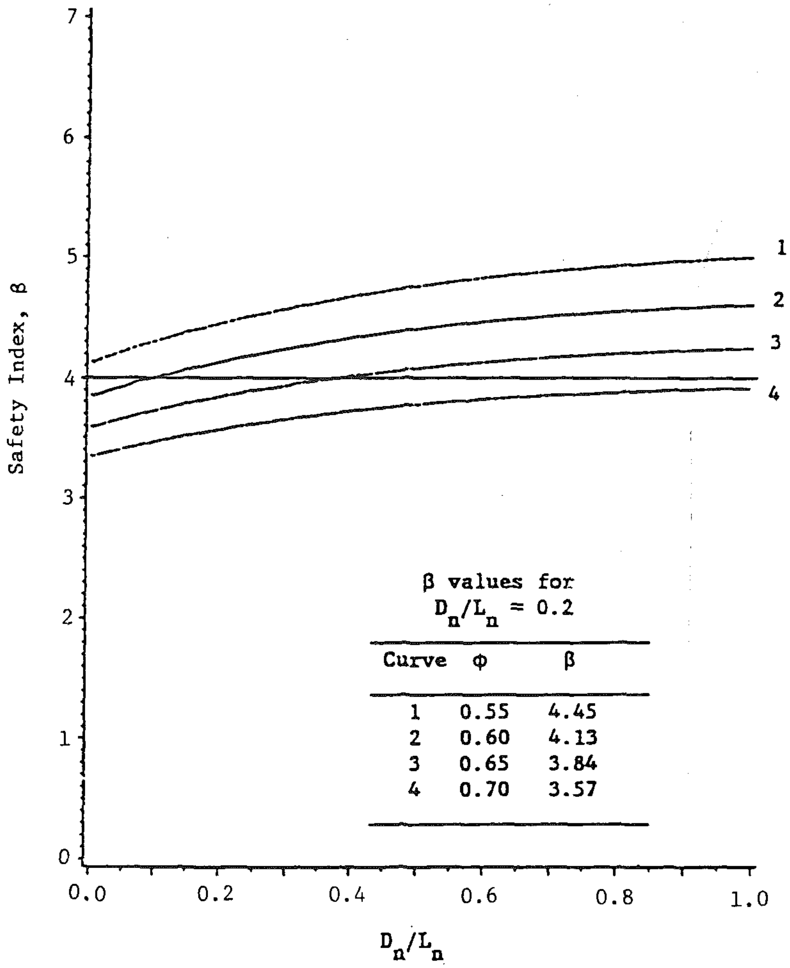
FIG. 2. Safety Indexes, β, for Different Resistance Factors, φ, and D_n/L_n Ratios for Stainless Steel Columns Subjected to Torsional-Flexural Buckling

for 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 slightly larger than the target value of 4.0.

For longitudinal and transverse fillet welds, a total of 10 connection tests (Errera et al. 1970) were used in this study. Based on the results of calibration (Lin et al. 1988b), 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, resistance factors of 0.55 and 0.65 are recommended for the LRFD criteria against plate and weld metal failures, respectively.

Bolted Connections

Previous Cornell test results (Errera et al. 1970) indicated that the failure modes of bolted connections in cold-formed stainless steel construction are



β values for $D_n/L_n = 0.2$

Curve	φ	β
1	0.55	4.45
2	0.60	4.13
3	0.65	3.84
4	0.70	3.57

FIG. 3. Safety Indexes, β, for Different Resistance Factors, φ, and D_n/L_n Ratios for Groove Welds [(17)]

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 1: longitudinal shearing of the sheet along two parallel lines; type 2: bearing or piling up of material in front of bolt; type 3: tearing of the sheet at the net section; and type 4: shearing of the bolt. The calibrations of design provisions for shear failure in connected parts, bearing, and tension failure of bolted connections have been investigated and reported in Lin et al. (1988b). The design provisions for shear and tension failure in bolts were not calibrated due to 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 and fabrication factors 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

TABLE 1. Computed Safety Index β and Resistance Factor ϕ for Bolted Connections

Failure mode (1)	Computed safety index for $D_n/L_n = 0.2$ (2)	Resistance factor (3)
Type 1: Shear failure in connected parts	4.10	0.70
Type 2: Bearing failure	4.14	0.65
Type 3: Tension failure in connected parts	4.04	0.70

factors presented by Lin et al. (1988b), the safety index and corresponding resistance factor can be determined by using the formula given earlier in this paper.

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

Local Distortion

When local distortions 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. Detailed discussion on this subject is provided in Lin et al. (1989). This design provision is considered to be necessary for stainless steel structural members because of stainless steel's low proportional limit and due to the fact that attention is often given to the appearance of the exposed surface of stainless steel used for architectural purposes.

SUMMARY AND CONCLUSIONS

Probability-based LRFD criteria for the design of cold-formed stainless steel structural members and connections have been developed on the basis of first-order probabilistic theory. The resistance factors have been determined by calibrating the appropriate design provisions (Lin et al. 1988b). 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. Because all resistance factors were obtained by calibrations of various design provisions on the basis of the available test data, additional tests are needed to refine the resistance factors achieved.

In view of the fact that the 1/5 ratio was used as an average value of D_n/L_n in the development of the LRFD Specification, it is generally expected that the LRFD method may be found to be slightly conservative with respect to allowable stress design when the D_n/L_n ratio is less than 0.2.

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APPENDIX II. NOTATION

The following symbols are used in this paper:

A = area of full, unreduced cross section;
A_n = net area of cross sections;
B = random variable reflecting uncertainties in transformation of live loads into live-load effects;
C = random variable reflecting uncertainties in transformation of dead loads into dead-load effects;
c_D, c_L = deterministic influence coefficients translating load intensities to load effects; subscript 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 longitudinal direction;
F_y = yield strength;
L_c = specified live-load intensity;
L_n = nominal specified live load;
M = random variable characterizing uncertainties in material strength;
P = random variable reflecting 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 structure member;
S_e = effective section modulus of reduced section;
V = coefficient of variation;
V_x = coefficient of variation of random variable x;
$(x)_m$ = mean value of random variable x; subscript m denotes mean value;

β = safety index;
β_0 = target safety index;
γ_D = dead load factor;
γ_L = live load factor; and
ϕ = resistance factor.