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AISI LRFD Method for Cold-Formed Steel Structural Members

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Abstract: In the design of steel buildings, the allowable stress design (ASD) method has long been used for cold-formed steel structural members in the United States. Recently, the load and resistance factor design (LRFD) criteria have been developed for steel buildings using hot-rolled shapes and built-up members fabricated from steel plates. In order to develop load and resistance factor design (LRFD) criteria for cold-formed steel structural members, a joint research project is conducted at the University of Missouri-Rolla, Washington University, and the University of Minnesota under the sponsorship of the American Iron and Steel Institute. Based on the 1986 edition of the American Iron and Steel Institute (AISI) ASD specification, the LRFD specification for cold-formed steel structural members with commentary is prepared for consideration of the American Iron and Steel Institute. The background information for developing the proposed design criteria for structural members is discussed in this paper.

Introduction

In the design of steel buildings, the allowable stress design (ASD) method has long been used for cold-formed steel structural members in the United States and other countries. In this approach, the forces (bending moments, axial forces, shear forces) in structural members are computed by accepted methods of structural analysis for the specified working loads. These member forces or moments should not exceed the allowable values permitted by the applicable design specification (AISI: *Specification* 1986b).

Recently, the load and resistance factor design (LRFD) criteria have been developed for steel buildings using hot-rolled shapes and built-up members fabricated from steel plates in the United States (AISC: *Load and Resistance* 1986). The limit-states design method has been used in Canada and Europe for the design of steel structural members (CSA: "Cold Formed Steel" 1984; ECCS: "European Recommendations" 1983). 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. These factors reflect the uncertainties of analysis, design, loading, material properties, and fabrication.

In order to develop load and resistance factor design (LRFD) criteria for cold-formed steel structural members, a joint research project was conducted at the University of Missouri-Rolla, Washington University, and the University of Minnesota under the sponsorship of the American Iron and Steel Institute. Initial results were presented in several publications (Galambos and Yu 1984; Rang et al. 1979a, 1979b, 1979c, 1979d; Snyder et al. 1984;

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Supornsilaphachai et al. 1979). Based on the 1986 edition of the American Iron and Steel Institute (AISI) ASD Specification, the revised LRFD specification for cold-formed steel structural members with commentary has been prepared for consideration of the AISI (Hsiao et al. 1988b). This proposed document contains six sections for designing cold-formed steel structural members and connections. The background information for developing the proposed design criteria for structural members is discussed in this paper. For connections, additional information can be found in the proposed document (Hsiao et al. 1988b).

Design Procedure

Load and Resistance Factor Design

As discussed in the Introduction, the current method of designing cold-formed steel structural members, as presented in the 1986 AISI specification (AISI: *Specification* 1986b), is based on the allowable stress design method. The allowable load or moment is determined by dividing the nominal load or moment at a limit state by a factor of safety. Usual factors of safety inherent in the AISI "Specification for the Design of Cold-Formed Steel Structural Members" (AISI: *Specification* 1986b) are 5/3 for tension members and beams and 23/12 for columns.

A limit state is the condition at which the structural usefulness of a load-carrying element or member is impaired to such an extent that it becomes unsafe for the occupants of the structure, or the element no longer performs its intended function. Typical limit states for cold-formed steel members are excessive deflection, yielding, buckling, and attainment of maximum strength after local buckling (i.e., postbuckling strength). These limit states have been estabished through experience in practice or in the laboratory, and they have been thoroughly investigated through analytical and experimental research. The background for the establishment of the limit states is extensively documented in the commentary on the AISI specification and other publications (AISI: "Commentary" 1986a; Winter 1970; Pekoz 1986; Yu 1985), and a continuing research effort provides further improvement in understanding them.

The factors of safety are provided to account for the uncertainties and variabilities inherent in loads, analysis, limit-state model, material properties, geometry, and fabrication. Through experience it has been established that the present factors of safety provide satisfactory designs.

The allowable stress-design method employs only one factor of safety for a limit state. The use of multiple load factors provides a refinement in the design that can account for the different degrees of the uncertainties and variabilities of the design parameters. Such a design method is called load and resistance factor design, and its format is expressed by the following criterion:

$$\phi R_n \geq \Sigma \gamma_i Q_i \quad (1)$$

where R_n = the nominal resistance; ϕ = resistance factor; γ_i = load factor; and Q_i = load effect.

The nominal resistance is the strength of the element or member for a given limit state, computed for nominal section properties and for minimum specified material properties according to the appropriate analytical model that defines the strength. The resistance factor ϕ accounts for the uncertain-

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ties and variabilities inherent in R_n, and it is usually less than unity. The load effects Q_i are the forces on the cross section (bending moment, axial force, shear force) determined from the specified minimum loads by structural analysis, and γ_i are the corresponding load factors that account for the uncertainties and variabilities of the loads. The load factors are greater than unity.

The advantages of LRFD are: (1) The uncertainties and the variabilities of different types of loads and resistances are different (e.g., dead load is less variable than wind load), and so these differences can be accounted for by use of multiple factors; and (2) by using probability theory, all designs can ideally achieve a uniform reliability. Thus, LRFD provides the basis for a more rational and refined design method than is possible with the allowable stress design method.

Probabilistic Concepts

Factors of safety or load factors are provided against the uncertainties and variabilities that are inherent in the design process. Structural design consists of comparing nominal load effects Q to nominal resistances R, but both Q and R are random parameters. A limit state is violated if $R < Q$. While the possibility of this event ever occurring is never zero, a successful design should, nevertheless, have only an acceptably small probability of exceeding the limit state. If the exact probability distributions of Q and R were known, then the probability of $(R - Q) < 0$ could be exactly determined for any design. In general, the distributions of Q and R are not known, and only the means, Q_m and R_m, and the standard deviations, σ_Q and σ_R, are available. Nevertheless it is possible to determine relative reliabilities of several designs by using the concept of the "reliability index" β, which is extensively discussed in several publications (Ellingwood et al. 1980, 1982; Galambos et al. 1982; Ravindra and Galambos 1978). This reliability index can be expressed by the equation

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

where $V_R = \sigma_R/R_m$ and $V_Q = \sigma_Q/Q_m$, the coefficients of variation of R and Q, respectively. The index β is called the "reliability index," and it is a relative measure of the safety of the design. When two designs are compared, the one with the larger β is more reliable.

The concept of the reliability index can be used in determining the relative reliability inherent in current design, and it can be used in testing out the reliability of new design formats, as illustrated by the following example of simply supported braced beams subjected to dead and live loading.

The design requirement of the 1986 AISI specification (AISI: *Specification* 1986b) for such a beam

$$\frac{S_e F_y}{FS} = \frac{L^2 s}{8}(D_n + L_n) \quad \text{(3)}$$

where S_e = elastic section modulus based on the effective section; $FS = 5/3$ = the factor of safety for bending; F_y = specified yield point; L = span

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length, and s = beam spacing. D_n and L_n are, respectively, the code specified dead and live load intensities.

The mean resistance is defined as (Ravindra and Galambos 1978)

$$R_m = R_n(P_m M_m F_m) \quad (4)$$

In this equation, R_n = the nominal resistance, which in this case is

$$R_n = S_e F_y \quad (5)$$

that is, the nominal moment predicted on the basis of the specification. The mean values P_m, M_m, and F_m, and the corresponding coefficients of variation V_P, V_M, and V_F, are the statistical parameters that define the variability of the resistance: P_m = the mean ratio of the experimentally determined moment to the predicted moment for the actual material and cross-sectional properties of the test specimens; M_m = mean ratio of the yield point to the minimum specified value; and F_m = mean ratio of the actual section modulus to the specified (nominal) value.

The coefficient of variation of R equals

$$V_R = \sqrt{V_P^2 + V_M^2 + V_F^2} \quad (6)$$

The values of these data were obtained from examining the available tests on beams having different compression flanges with partially and fully effective flanges and webs, and from analyzing data on yield-point values from tests and cross-sectional dimensions from many measurements. This information was developed (Hsiao et al. 1988a) and is given as: $P_m = 1.11$, $V_P = 0.09$; $M_m = 1.10$, $V_M = 0.10$; $F_m = 1.0$, $V_F = 0.05$; and, thus, $R_m = 1.22R_n$ and $V_R = 0.14$.

The mean-load effect is equal to

$$Q_m = \frac{L^2 s}{8}(D_m + L_m) \quad (7)$$

and

$$V_Q = \frac{\sqrt{(D_m V_D)^2 + (L_m V_L)^2}}{D_m + L_m} \quad (8)$$

where D_m and L_m = the mean dead and live load intensities, respectively, and V_D and V_L = the corresponding coefficients of variation.

Load statistics have been analyzed (Ellingwood et al. 1980) and it was shown that $D_m = 1.05D_n$, $V_D = 0.1$; and $L_m = L_n$, $V_L = 0.25$.

The mean live load intensity equals the code live load intensity, if the tributary area is small enough so that no live load reduction is included. Substitution of the load statistics into Eqs. 7 and 8 gives

$$Q_m = \frac{L^2 s}{8}\left(\frac{1.05D_n}{L_n} + 1\right)L_n \quad (9)$$

$$V_Q = \frac{\sqrt{\left(1.05\,\dfrac{D_n}{L_n}\right)^2 V_D^2 + V_L^2}}{\left(1.05\,\dfrac{D_n}{L_n} + 1\right)} \quad (10)$$

Q_m and V_Q thus depend on the dead-to-live load ratio. Cold-formed steel beams typically have small D_n/L_n, and for the purposes of checking the reliability of these LRFD criteria, it will be assumed that $D_n/L_n = 1/5$, and so $Q_m = 1.21L_n(L^2s/8)$ and $V_Q = 0.21$.

From Eq. 3, we obtain the nominal design capacity for $D_n/L_n = 1/5$ and $FS = 5/3$. Thus

$$\frac{R_m}{Q_m} = \frac{(1.22)(2.0)L_n\left(\frac{L^2s}{8}\right)}{1.21L_n\left(\frac{L^2s}{8}\right)} = 2.02 \qquad (11)$$

and, from Eq. 2:

$$\beta = \frac{\ln(2.02)}{\sqrt{0.14^2 + 0.21^2}} = 2.79 \qquad (12)$$

By itself, $\beta = 2.79$, for beams having different compression flanges with partially and fully effective flanges and webs designed by the 1986 AISI specification (AISI: *Specification* 1986b), means nothing. However, when this is compared to β for other types of cold-formed members, and to β for designs of various types from hot-rolled steel shapes or even for other materials, then it is possible to say that this particular cold-formed steel beam has about an average reliability (Galambos et al. 1982).

Basis for LRFD of Cold-Formed Steel Structures

A great deal of work has been performed for determining the values of the reliability index β inherent in traditional design, as exemplified by the current structural design specifications such as the AISC specification for hot-rolled steel, the AISI specification for cold-formed steel, the ACI code for reinforced concrete members, etc. The studies for hot-rolled steel were summarized (Ravindra and Galambos 1978), also many further papers were referenced that contain additional data. The determination of β for cold-formed steel elements or members is presented in several publications (Hsiao et al. 1988a; Supornsilaphachai et al. 1979; Rang et al. 1979a, 1979b, 1979c, 1979d), where both the basic research data and β's inherent in the AISI specification are presented in great detail.

The entire set of data for hot-rolled steel and cold-formed steel designs, as well as data for reinforced concrete, aluminum, laminated timber, and masonry walls was reanalyzed in several publications (Ellingwood et al. 1980, 1982; Galambos et al. 1982) by using: (1) updated load statistics; and (2) a more advanced level of probability analysis that was able to incorporate probability distributions that describe the true distributions more realistically. The details of this extensive reanalysis are presented in several publications (Ellingwood et al. 1980, 1982; Galambos et al. 1982) and also only the final conclusions from the analysis are summarized herein.

The values of the reliability trade index β vary considerably for the different kinds of loading, the different types of construction, and the different types of members with a given material design specification. In order to achieve more consistent reliability, it was suggested (Galambos et al. 1982) that the following values of β 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 β_o, for use in LRFD, are $\beta_o = 3.0$ for gravity loading; $\beta_o = 4.5$ for connections; and $\beta_o = 2.5$ for wind loading.

These target reliability indices are the ones inherent in the load factors recommended by the American National Standards Institute (ANSI) ("Minimum Design Loads" 1982).

For cold-formed simply supported braced steel beams with stiffened flanges, which were designed according to the 1986 AISI allowable stress-design specification or to any previous version of this specification, it was previously shown that for the representative dead-to-live load ratio of 1/5, the reliability index $\beta = 2.8$. Considering the fact that for other such load ratios, or for other types of members, the reliability index inherent in current cold-formed steel construction could be more or less than this value of 2.8. A somewhat lower target reliability index of $\beta_o = 2.5$ is recommended as a lower limit for the new LRFD specification. The resistance factors ϕ were selected such that $\beta_o = 2.5$ is essentially the lower bound of the actual β's for members. In order to assure that failure of a structure is not initiated in the connections, a higher target reliability of $\beta_o = 3.5$ is recommended for joints and fasteners. These two targets of 2.5 and 3.5 for members and connections, respectively, are somewhat lower than those recommended by ANSI (ANSI: "Minimum Design Loads" 1982) (i.e., 3.0 and 4.5, respectively), but they are essentially the same targets as are the basis for the 1986 AISC LRFD specification (AISC: "Load and Resistance" 1986).

Based on the 1982 ANSI load code (ANSI 1982), the following load factors are recommended in the new AISI LRFD specification for six load combinations:

- $1.4D_n + L_n$
- $1.2D_n + 1.6L_n + 0.5(L_{rn} \text{ or } S_n \text{ or } R_{rn})$
- $1.2D_n + (1.4L_{rn} \text{ or } 1.6S_n \text{ or } 1.6R_{rn}) + (0.5L_n \text{ or } 0.8W_n)$
- $1.2D_n + 1.3W_n + 0.5L_n + 0.5(L_{rn} \text{ or } S_n \text{ or } R_{rn})$
- $1.2D_n + 1.5E_n + (0.5L_n \text{ or } 0.2S_n)$
- $0.9D_n - (1.3W_n \text{ or } 1.5E_n)$

where D_n = nominal dead load; E_n = nominal earthquake load; L_n = nominal live load; L_{rn} = nominal roof live load; R_{rn} = nominal roof rain load; S_n = nominal snow load; and W_n = nominal wind load (exception: for wind load on individual purlins, girts, wall panels, and roof decks, multiply the load factor for W_n by 0.9).

In view of the fact that the dead load of cold-formed steel structures is usually smaller than that of heavy construction, the first case of load combinations included in the LRFD specification is $(1.4D_n + L_n)$ instead of the ANSI value of $1.4D_n$. This AISI requirement is identical with the ANSI code, when $L_n = 0$.

Because of special circumstances inherent in cold-formed steel structures, the following additional LRFD criteria apply for roof, floor, and wall construction using cold-formed steel.

For roof and floor composite construction

$$1.2D_n + 1.6C_{wn} + 1.4C_n \quad (13)$$

where C_{wn} = nominal weight of wet concrete during construction; and C_n =

nominal construction load, including equipment, workmen, and formwork, but excluding the weight of the wet concrete.

This suggestion is included in the Commentary to provide safe construction practices for cold-formed steel decks and panels that otherwise may be damaged during construction. The load factor used for the weight of wet concrete is 1.6, because it is frequently dumped into a pile or impacted onto the deck. An individual sheet can be subjected to this load. The use of a load factor of 1.4 for the construction load reflects a general practice of a 33% strength increase for concentrated loads.

It should be noted that for the third case of load combinations previously listed, the load factor used for the nominal roof live load, L_{rn}, in the AISI LRFD specification is 1.4, instead of the ANSI value of 1.6. The use of a relatively smaller load factor is because the roof live load is due to the presence of workmen and materials during repair operations and, therefore, can be considered as a type of construction load.

For roof and wall construction, the load factor for the nominal wind load W_n, to be used for the design of individual purlins, girts, wall panels, and roof decks, should be multiplied by a reduction factor of 0.9 because these elements are secondary members subjected to a short duration of wind load and thus, can be designed for a smaller reliability than primary members such as beams and columns. For example, the reliability index of a wall panel under wind load alone is approximately 1.5 with this reduction factor.

Deflection calculations for serviceability criteria should be made with the appropriate unfactored loads.

The load factors and load combinations just given are recommended for use with the LRFD criteria for cold-formed steel. The following portions of this paper present the background for the resistance factors ϕ listed in Table 1, which are recommended for use in the AISI LRFD specification. These ϕ factors are determined in conformance with the load factors just given, to approximately provide a target β_o of 2.5 for members and 3.5 for connections, respectively, for the load combination $1.2D_n + 1.6L_n$. For practical reasons, it is desirable to have relatively few different resistance factors, and so the actual values of β may differ from the derived targets. This means that

$$\phi R_n = c(1.2D_n + 1.6L_n) = \left(1.2\frac{D_n}{L_n} + 1.6\right)cL_n \quad \text{(14)}$$

where c = the deterministic influence coefficient translating load intensities to load effects.

By assuming $D_n/L_n = 1/5$, Eqs. 14 and 9 can be rewritten as follows:

$$R_n = 1.84\left(\frac{cL_n}{\phi}\right) \quad \text{(15)}$$

$$Q_m = \left(1.05\frac{D_n}{L_n} + 1\right)cL_n = 1.21cL_n \quad \text{(16)}$$

Therefore

$$\frac{R_m}{Q_m} = \frac{1.521}{\phi}\left(\frac{R_m}{R_n}\right) \quad \text{(17)}$$

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TABLE 1. Resistance Factors

Type of strength (1)	Resistance factor, ϕ (2)
Stiffeners	
Transverse stiffeners	0.85
Shear stiffeners[a]	0.90
Tension members	0.95
Flexural members	
Bending strength	
For sections with stiffened or partially stiffened compression flanges	0.95
For sections with unstiffened compression flanges	0.90
Laterally unbraced beams	0.90
Web design	
Shear strength[a]	0.90
Web crippling	
For single unreinforced webs	0.75
For I-sections	0.80
Concentrically loaded compression members	0.85
Combined axial load and bending	
ϕ_c for compression	0.85
ϕ_b for bending	
Using nominal section strength	0.90–0.95
Using lateral buckling strength	0.90
Cylindrical tubular members	
Bending strength	0.95
Axial compression	0.85
Wall studs and wall stud assemblies	
Wall studs in compression	0.85
Wall studs in bending	
For sections with stiffened or partially stiffened compression flanges	0.95
For sections with unstiffened compression flanges	0.90

[a]When $h/t \leq \sqrt{Ek_v/F_y}$, $\phi = 1.0$.

The ϕ factors can be computed from Eq. 17 and the following equation by using $V_Q = 0.21$:

$$\text{Target } \beta_o = \frac{\ln\left(\frac{R_m}{Q_m}\right)}{\sqrt{V_R^2 + V_Q^2}} \quad \text{(18)}$$

Design of Members

Yield Point

The following statistical data (mean values and coefficients of variation) on material and cross-sectional properties were developed in two publica-

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tions (Rang et al. 1979a, 1979b) for use in the derivation of the resistance factors ϕ:

$$(F_y)_m = 1.10F_y; \quad M_m = 1.10; \quad V_{Fy} = V_M = 0.10 \quad (19)$$

$$(F_{ya})_m = 1.10F_{ya}; \quad M_m = 1.10; \quad V_{Fya} = V_M = 0.11 \quad (20)$$

$$(F_u)_m = 1.10F_u; \quad M_m = 1.10; \quad V_{Fu} = V_M = 0.08 \quad (21)$$

$$F_m = 1.00; \quad F_F = 0.05 \quad (22)$$

The subscript m refers to mean values. F_y, F_{ya}, and F_u = respectively, the specified minimum yield point, the average yield point including the effect of cold forming, and the specified minimum tensile strength.

These data are based on the analysis of many samples, and they are representative properties of materials and cross sections used in the industrial application of cold-formed steel structures.

Tension Members

The resistance factor of $\phi_t = 0.95$ used for tension member design was derived from the aforementioned procedure and a selected β_o value of 2.5. In the determination of the resistance factor, the following formulas were used for R_m and R_n:

$$R_m = A_n(F_y)_m \quad (23)$$

$$R_n = A_nF_y \quad (24)$$

i.e.

$$\frac{R_m}{R_n} = \frac{(F_y)_m}{F_y} \quad (25)$$

in which A_n = the net area of the cross section, $(F_y)_m = 1.10F_y$, as discussed in the previous section. By using $V_M = 0.10$, $V_F = 0.05$, and $V_P = 0$, the coefficient of variation V_R is:

$$V_R = \sqrt{V_M^2 + V_F^2 + V_P^2} = 0.11 \quad (26)$$

Based on $V_Q = 0.21$ and the resistance factor of 0.95, the value of $\beta = 2.4$, which is close to the stated target value of $\beta_o = 2.5$.

Flexural Members

Strength for Bending Only

Bending strengths of flexural members are differentiated according to whether or not the member is laterally braced. If such members are laterally supported, then they are proportioned according to the nominal section strength. If they are laterally unbraced, then lateral-torsional buckling is one of the limit states.

Nominal Section Strength. The bending strength of beams with a compression flange having stiffened, partially stiffened, or unstiffened elements is based on the post-buckling strength of the member, and use is made in LRFD of the effective width concept in the same way as in the 1986 AISI specification. Several publications (AISI: "Commentary" 1986a; Pekoz 1986;

Winter 1970; Yu 1985) provide an extensive treatment of the background research.

The experimental bases for the postbuckling strengths of cold-formed beams were examined in two publications (Hsiao et al. 1988a; Pekoz 1986), where different cases were studied according to the types of compression flanges and the effectiveness of webs.

On the basis of the initiation of yielding, the nominal strength R_n is based on the nominal effective cross section and on the specified minimum yield point, i.e., $R_n = S_e F_y$.

The computed values of β for the selected values of $\phi_b = 0.95$, for sections with stiffened or partially stiffened compression flanges and 0.90 for sections with unstiffened compression flanges, and for a dead-to-live load ratio of 1/5 are listed in Hsiao et al. (1988b), which shows that the β values vary from 2.53 to 4.05.

Lateral Buckling Strength. There are not many test data on laterally unsupported cold-formed beams. The available test results were summarized (Hsiao et al. 1988a), and they were compared with predictions from AISI design formulas, theoretical formulas, and SSRC formulas. It showed that AISI design formulas for lateral buckling strength are conservative.

The statistical data used (Hsiao et al. 1988a) are listed in Hsiao et al. (1988b). Using the recommended resistance factor $\phi_b = 0.90$, the values of β vary from 2.35 to 3.8. It shoud be noted that the recommended design criteria use some simplified and conservative formulas, which are the same as the allowable stress-design rules included in the 1986 AISI specification.

Strength for Shear Only

The shear strength of beam webs is governed by either yielding or buckling, depending on the h/t ratio and the mechanical properties of steel. For beam webs having small h/t ratios, the shear strength is governed by shear yielding, i.e.

$$V_n = A_w \tau_y = \frac{A_w F_y}{\sqrt{3}} = 0.577 F_y h t \quad (27)$$

in which A_w = the area of the beam web computed by ($h \times t$), where h is the flat portion of the web, t is the web thickness, and τ_y is the yield point of steel in shear, which can be computed by $F_y/\sqrt{3}$.

For beam webs having large h/t ratios, the shear strength is governed by elastic shear buckling, i.e.

$$V_n = A_w \tau_{cr} = \frac{k_v \pi^2 E A_w}{12(1 - \mu^2)\left(\frac{h}{t}\right)^2} \quad (28)$$

in which τ_{cr} = the critical shear buckling stress in the elastic range; k_v = the shear buckling coefficient; E = the modulus of elasticity; and μ = Poisson's ratio. By using $\mu = 0.3$, the shear strength, V_n, can be determined as follows:

$$V_n = \frac{0.905 E k_v t^2}{h} \quad (29)$$

For beam webs having moderate h/t ratios, the shear strength is based on the inelastic buckling, i.e.

$$V_n = 0.64t^2\sqrt{k_v F_y E} \quad (30)$$

In view of the fact that the appropriate test data on shear are not available (Supornsilaphachai et al. 1979), the ϕ_v factors were derived from the condition that the nominal resistance for the LRFD method is the same as the nominal resistance for the allowable stress-design method. Thus

$$(R_n)_{\mathrm{LRFD}} = (R_n)_{\mathrm{ASD}} \quad (31)$$

Since

$$(R_n)_{\mathrm{LRFD}} \geq \frac{c(1.2D_n + 1.6L_n)}{\phi_v} \quad (32)$$

$$(R_n)_{\mathrm{ASD}} \geq c(FS)(D_n + L_n) \quad (33)$$

the resistance factors can be computed from the following formula:

$$\phi_v = \frac{1.2D_n + 1.6L_n}{(FS)(D_n + L_n)} = \frac{\left(1.2\dfrac{D_n}{L_n} + 1.6\right)}{(FS)\left(\dfrac{D_n}{L_n} + 1\right)} \quad (34)$$

By using a dead-to-live load ratio of $D_n/L_n = 1/5$, the ϕ_v factors computed from Eq. 34 are listed in Hsiao et al. (1988b) for three different ranges of h/t ratios. It shows that the ϕ_v factors vary from 0.90 to 1.0. The factors of safety are adopted from the AISI specification for allowable stress design. It should be noted that the use of a small safety factor of 1.44 for yielding in shear is justified by long standing use and by the minor consequences of incipient yielding in shear compared with those associated with yielding in tension and compression.

Web Crippling Strength

The nominal concentrated load or reaction, P_n, is determined by the allowable load given in the AISI ASD specification (1986b) multiplied by the appropriate factor of safety. In this regard, a factor of safety of 1.85 is used for shapes having single unreinforced webs, and a factor of safety of 2.0 is used for I beams or similar sections in the LRFD specification.

On the basis of the statistical analysis of the available test data on web crippling, the values of P_m, M_m, F_m, V_P, V_M, and V_F were computed and selected. These values are presented in Hsiao et al. (1988b). By using β_o = 2.5, the resistance factors ϕ_w = 0.75 and 0.80 were selected for single unreinforced webs and I sections, respectively. The values of β corresponding to these values of ϕ_w vary from 2.36 to 3.80.

Combined Bending and Web Crippling Strength

A total of 551 tests were calibrated for combined bending and web crippling strength. Based on ϕ_w = 0.75 for single unreinforced webs and ϕ_w = 0.80 for I sections, the value of safety indices vary from 2.45 to 3.27 as given by Hsiao et al. (1988b).

Concentrically Loaded Compression Members

The available experimental data on cold-formed steel concentrically loaded compression members were evaluated (Hsiao et al. 1988a). The test results were compared to the predictions based on the same mathematical models on which the AISI ASD specification was based. The design provisions in these LRFD criteria are also based on the same mathematical models.

Column capacities using these LRFD criteria are based on the same prediction models as were employed in the formulation of the AISI ASD specification. A total of 264 tests were examined; 14 different cases were studied according to the types of columns, the types of compression flanges, and the type of failure modes. The resistance factor $\phi_c = 0.85$ was selected on the basis of the statistical data (Hsiao et al. 1988a). The corresponding safety indices vary from 2.39 to 3.34. A summary of the information is given by Hsiao et al. (1988b).

Combined Axial Load and Bending

The LRFD specification provides the similar interaction equations as the 1986 edition of the AISI ASD specification except that design strengths are used instead of allowable strengths.

A total of 144 tests were calibrated for combined axial load and bending. Nine different cases were studied according to the types of sections, stable conditions, and loading conditions. Based on $\phi_c = 0.85$, $\phi_b = 0.95$ or 0.90 for nominal section strength, and $\phi_b = 0.90$ for lateral buckling strength. The values of safety indices vary from 2.7 to 3.34, as given by Hsiao et al. (1988b).

Comparisons of ASD and LRFD Criteria

For the purpose of comparison, the design equation used for the LRFD criteria is based on dead and live loads as follows:

$$\phi R_n \geq 1.2D_n + 1.6L_n \quad (35)$$

The unfactored load combination $(D_n + L_n)$, or allowable load, can be computed from the nominal resistance R_n, the resistance factor ϕ, and a given D_n/L_n ratio as follows:

$$\phi R_n \geq \left(1.2\frac{D_n}{L_n} + 1.6\right)L_n \quad (36)$$

$$\phi R_n \geq \left(1.2\frac{D_n}{L_n} + 1.6\right)\left[\frac{D_n + L_n}{\left(\frac{D_n}{L_n} + 1\right)}\right] \quad (37)$$

Therefore

$$D_n + L_n \leq \frac{\phi R_n\left(\frac{D_n}{L_n} + 1\right)}{\left(1.2\frac{D_n}{L_n} + 1.6\right)} \quad (38)$$

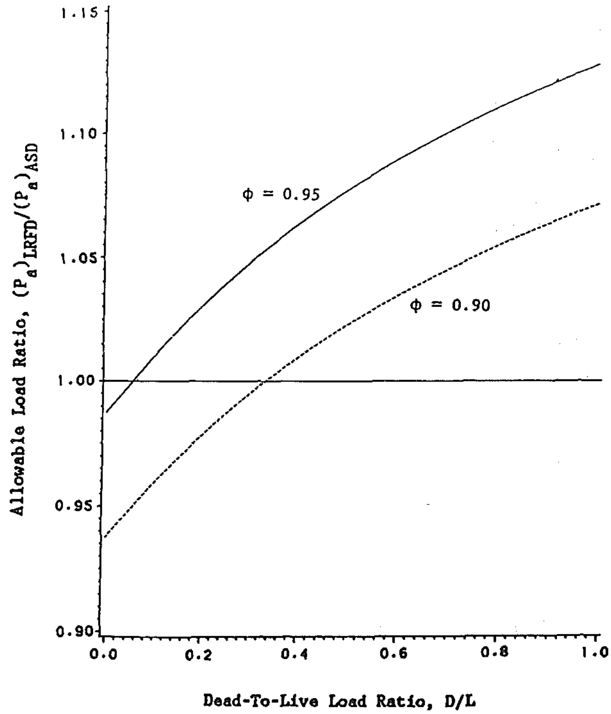

FIG. 1. Allowable Load Ratio versus D/L Ratio for Tension Members and Bending Strength of Beams

From Eq. 38, the factor of safety (FS) against the nominal resistance used in the LRFD criteria is as follows:

$$(FS)_{\mathrm{LRFD}} = \frac{\left(1.2\dfrac{D_n}{L_n} + 1.6\right)}{\phi\left(\dfrac{D_n}{L_n} + 1\right)} \quad (39)$$

The allowable load for ASD is based on a factor of safety (FS) of the nominal resistance, as shown in Eq. 40

$$D_n + L_n \leq \frac{R_n}{(FS)_{\mathrm{ASD}}} \quad (40)$$

Therefore, based on Eqs. 38 and 40, the allowable load ratio is as follows:

$$\frac{(P_a)_{\mathrm{LRFD}}}{(P_a)_{\mathrm{ASD}}} = \phi(FS)_{\mathrm{ASD}}\frac{\left(\dfrac{D_n}{L_n} + 1\right)}{\left(1.2\dfrac{D_n}{L_n} + 1.6\right)} \quad (41)$$

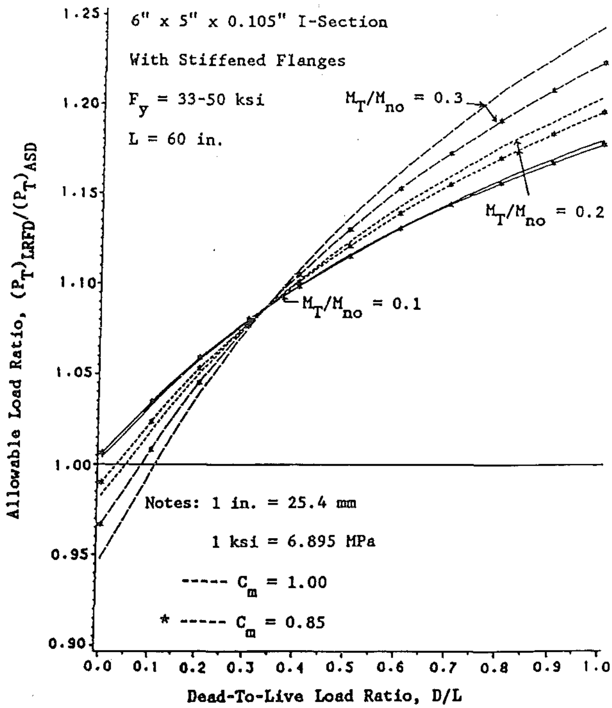

FIG. 2. Allowable Load Ratio versus D/L Ratio for Doubly Symmetric Beam Columns

Eq. 41 was used in this study to compare the AISI specification for allowable stress design and the proposed LRFD specification. This equation would only be applicable to structural members with only one type of load. It does not apply to the combined loading where design formulas are interaction equations. Fig. 1 shows the allowable load ratio versus dead-to-live load ratio for tension members and bending strength of beams. The solid curve represents $\phi = 0.95$, which is the ϕ factor for tension members and bending strength of beams with stiffened or partially stiffened compression flanges while the dotted curve represents $\phi = 0.90$, which is the ϕ factor for bending strength of beams with unstiffened compression flanges and lateral buckling of beams. For other structural members, the curves are similar as those curves in Fig. 1, except that they are shifted up or down depending on the factor of safety and ϕ factor. Detailed information can be found in the comparative study of design methods for cold-formed steel (Hsiao et al. 1988c). As shown in Fig. 1, the LRFD criteria are slightly conservative for small dead-to-live load ratios. For large dead-to-live load ratios, the LRFD criteria would result in a more economic design than ASD.

When a structural member has to be designed for a combination of loads or load effects, interaction equations are used in the proposed LRFD specification. Due to the complexity of the design equations for combined loads, the following specific examples were chosen for comparison.

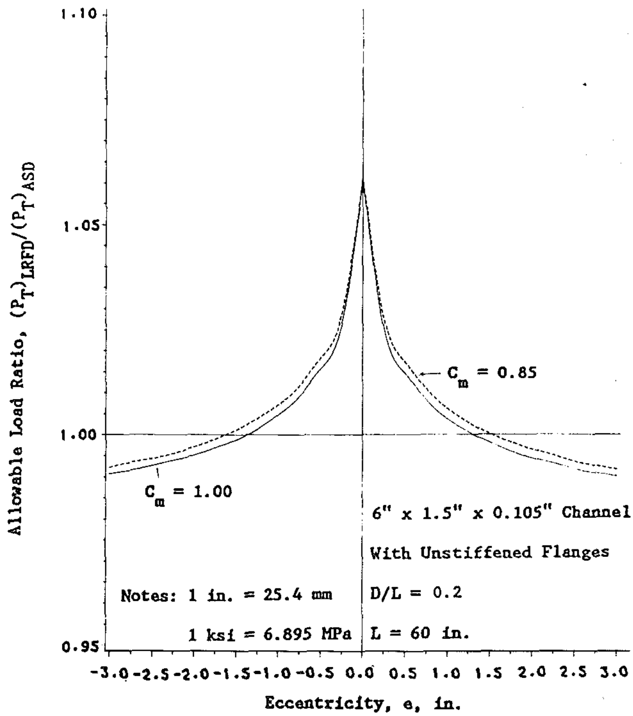

FIG. 3. Allowable Load Ratio versus D/L Ratio for Singly Symmetric Beam Columns

For doubly symmetric beam columns, an I section with equal end moments bending about the x-axis is used for comparison. Since the end moments are independent of the axial load, the ratio of the unfactored applied moment to the nominal moment capacity based on section strength, M_T/M_{no}, was considered to be a parameter in the equations for determining the allowable loads.

Fig. 2 shows the allowable load ratio versus dead-to-live load ratio for various end moment ratios. This figure is based on the flexural failure at the midlength of the beam column, which governs the design for this case. The curves without star symbols are for C_m = 1.0. The curves with star symbols in Fig. 2 are for the same I section, except that the coefficient, C_m, is 0.85. The value of 0.85 is used for unbraced beam columns and beam columns with restrained ends subject to transverse loading between its supports. For small end moment ratios, the C_m value has a negligible effect on the allowable load ratio. The effect of C_m on the allowable load ratio increases as the end moment ratio increases, as shown in Fig. 2.

For singly symmetric beam columns, the direction of the moment and location of the axial load can be important. Therefore, the eccentricity of applied load is used as a parameter instead of end moment ratio.

Fig. 3 shows the allowable load ratio versus eccentricity for the 5 ft (1.524 m) long channel with D/L = 0.2 and F_y = 33 ksi (228 MPa). The curves

shown in the figure are obtained for C_m values of 1.0 and 0.85. The bottom solid line represents the curve for $C_m = 1.0$. It can be seen that the smaller the eccentricity, the larger the allowable load ratio. This relationship holds for both positive and negative eccentricities. The top line in Fig. 3 represents the curve for $C_m = 0.85$. It can be seen that the C_m value has negligible effect on the allowable load ratio.

Additional information can be found in the comparative study of design methods for cold-formed steel (Hsiao et al. 1988c).

Summary and Conclusions

The AISI specification for load and resistance factor design of cold-formed steel structural members has been developed. This paper presents a brief discussion of the reasoning behind, and the justification for, various provisions being proposed for designing tension members, beams, columns, and beam columns. Additional publications are cited in the paper for future reference.

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Appendix II. Notation

The following symbols are used in this paper:

A_n = net area;
A_w = area of beam web;
C_n = nominal construction load;
C_{wn} = nominal weight of wet concrete during construction;
c = deterministic influence coefficient translating load intensities to load effect;
D_m = mean dead load;
D_n = nominal dead load;
E = modulus of elasticity;
E_n = nominal earthquake load;

F_m = mean ratio of actual section modulus to nominal value;
F_y = specific yield point;
F_{ya} = average yield point;
F_u = specified minimum tensile strength;
G = shear modulus;
h = depth of the flat portion of web measured along plane of web;
I_y = moment of inertia about y-axis;
J = torsional constant;
k_v = shear buckling coefficient;
L = unbraced length;
L_m = mean live load;
L_n = nominal live load;
L_{rn} = nominal roof live load;
M_a = allowable moment;
M_m = mean ratio of yield point to minimum specified value;
M_{no} = nominal moment capacity based on section strength;
M_T = applied unfactored bending moment at each end of member;
P_a = allowable load;
P_m = mean ratio of the experimentally determined ultimate load to the predicted ultimate load of test specimens;
P_T = total unfactored load;
Q = load effect;
Q_m = mean load effect;
R = resistance;
R_m = mean value of resistance;
R_n = nominal resistance;
R_{rn} = nominal roof rain load;
S_e = elastic section modulus based on effective section;
S_n = nominal snow load;
s = beam spacing;
t = thickness;
V = coefficient of variation;
V_n = nominal shear strength;
W_n = nominal wind load;
β = reliability index;
β_o = target reliability index;
γ = load factor;
μ = Poisson's ratio;
σ = standard deviation;
τ_{cr} = critical shear buckling stress;
τ_y = yield point in shear;
ϕ = resistance factor;
ϕ_b = resistance factor for bending strength;
ϕ_c = resistance factor for concentrically loaded compression member;
ϕ_t = resistance factor for tension member;
ϕ_v = resistance factor for shear strength; and
ϕ_w = resistance factor for web crippling strength.

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