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AISI LRFD METHOD FOR COLD-FORMED STEEL STRUCTURAL MEMBERS

by

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I. 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 (Refs. 3 and 5).

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 (Ref. 1). The limit states design method has been used in Canada and Europe for the design of steel structural members (Refs. 5 and 8). 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 Minnessota under the sponsorship of American Iron and Steel Institute. Initial results were presented in Refs. 10, 19 through 22, 24, and 25. Based on the 1986 Edition of the AISI ASD Specification, the revised LRFD specification for cold-formed steel structural members with commentary has been prepared for consideration of the American Iron and Steel Institute (Ref. 12). 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 Ref. 12.

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II. DESIGN PROCEDURE

A. 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
(Ref. 3), 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 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., post-buckling strength). These limit
states have been established through experience in practice or in the labo-
ratory, 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
(Refs. 2 and 27) (see also Refs. 18 and 28), 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 the loads, the analysis, the limit state model,
the material properties, the geometry, and the fabrication. Through expe-
rience 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 which 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:

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

where
           R_n = the nominal resistance
           ϕ = resistance factor
           γ_i = load factors
           Q_i = load effects

    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
which defines the strength. The resistance factor ϕ accounts for the un-
certainties 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 mo-
ment, axial force, shear force) determined from the specified minimum loads
by structural analysis, and γ_i are the corresponding load factors which ac-

count 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 achieve ideally 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.

B. Probabilistic Concepts

Factors of safety or load factors are provided against the uncertainties and variabilities which 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 Refs. 6, 7, 9, and 23. This reliability index can be expressed by the equation

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

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 for such a beam is

$$S_e F_y / FS = (L^2 s/8)(D_n + L_n) \qquad (3)$$

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 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 (Ref. 23)

$$R_m = R_n(P_m M_m F_m) \tag{4}$$

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

$$R_n = S_e F_y \tag{5}$$

that is, the ultimate 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 which define the variability of the resistance:

> P_m = the mean ratio of the experimentally determined ultimate moment to the predicted ultimate 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
> F_m = mean ratio of the section modulus to the Handbook (nominal) value

The coefficient of variation of R equals

$$V_R = \sqrt{V_P{}^2 + V_M{}^2 + V_F{}^2} \tag{6}$$

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 in Ref. 11 and is given below:

$P_m = 1.11$, $V_P = 0.09$; $M_m = 1.10$, $V_M = 0.10$; $F_m = 1.0$, $V_F = 0.05$ and thus $R_m = 1.22R_n$ and $V_R = 0.14$.

The mean load effect is equal to

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

and

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

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

Load statistics have been analyzed in Ref. 6, where it was shown that

$$D_m = 1.05D_n, \quad V_D = 0.1; \quad L_m = L_n, \quad V_L = 0.25.$$

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

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

$$V_Q = \frac{\sqrt{(1.05D_n/L_n)^2 V_D{}^2 + V_L{}^2}}{(1.05D_n/L_n + 1)} \tag{10}$$

Q_m and V_Q thus depend on the dead-to-live load ratio. Cold-formed 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 $Q_m = 1.21L_n(L^2 s/8)$ 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

$$\frac{R_m}{Q_m} = \frac{1.22 \times 2.0 \times L_n(L^2 s/8)}{1.21 L_n(L^2 s/8)} = 2.02$$

and, from Eq. 2:

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

Of itself $\beta = 2.79$ for beams having different compression flanges with partially and fully effective flanges and webs designed by the 1986 AISI Specification 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 (Ref. 9).

C. 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 are summarized in Ref. 23, where also many further papers are referenced which contain additional data. The determination of β for cold-formed steel elements or members is presented in Refs. 11, 25, and 19 through 22, where both the basic research data as well as the 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 re-analyzed in Refs. 6, 7, and 9 by using a) updated load

statistics and b) a more advanced level of probability analysis which was able to incorporate probability distributions which describe the true distributions more realistically. The details of this extensive reanalysis are presented in Refs. 6, 7, and 9 and also only the final conclusions from the analysis are summarized here:

1) The values of the reliability index β vary considerably for the different kinds of loading, the different types of construction, and the different types of members within a given material design specification. In order to achieve more consistent reliability, it was suggested in Ref. 9 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:

Basic case: Gravity loading, $\beta_o = 3.0$
For connections: $\beta_o = 4.5$
For wind loading: $\beta_o = 2.5$

These target reliability indices are the ones inherent in the load factors recommended in the ANSI A58.1-82 Load Code (Ref. 4).

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 shown above 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 A58.1-82 (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 (Ref. 1).

2) The following load factors and load combinations were developed in Refs. 6 and 7 to give essentially the same β's as the target β_o's, and are recommended for use with the 1982 ANSI Load Code (Ref. 4) for all materials, including cold-formed steel:

1. $1.4D_n$
2. $1.2D_n + 1.6L_n + 0.5(L_{rn} \text{ or } S_n \text{ or } R_n)$
3. $1.2D_n + 1.6(L_{rn} \text{ or } S_n \text{ or } R_n) + (0.5L_n \text{ or } 0.8W_n)$
4. $1.2D_n + 1.3W_n + 0.5L_n + 0.5(L_{rn} \text{ or } S_n \text{ or } R_n)$
5. $1.2D_n + 1.5E_n + (0.5L_n \text{ or } 0.2S_n)$
6. $0.9D_n - (1.3W_n \text{ or } 1.5E_n)$

where

D_n = nominal dead load

E_n = nominal earthquake load
L_n = nominal live load due to occupancy;
    weight of wet concrete for composite construction
L_{rn} = nominal roof live load
R_n = nominal roof rain load
S_n = nominal snow load
W_n = nominal wind load

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:

a) For roof and floor construction

$$1.2D_n+1.6C_{wn}+1.4C_n$$

where

C_{wn} = nominal weight of wet concrete during construction
C_n = nominal construction load, including equipment, workmen and formwork, but excluding the weight of the wet concrete.

This criterion has been included in the Commentary to provide safe construction practices for cold-formed steel decks and panels which otherwise could be damaged during erection.

b) For roof and wall construction, it is recommended that 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 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 are to be made with the appropriate unfactored loads.

The load factors and load combinations given above 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 given above 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 will differ from the derived targets. This means that

Table 1
Resistance Factors
(Section A5.1.5 of the Proposed LRFD Specification)

Type of Strength	Resistance Factor, ϕ
(a) Stiffeners	
Transverse stiffeners	0.85
Shear stiffeners[*]	0.90
(b) Tension members	0.95
(c) Flexural members	
Bending strength	
For sections with stiffened compression flanges	0.95
For sections with unstiffened compression flanges	0.90
Laterally unbraced beams	0.90
Web design	
Shear strength[*]	0.90
Web crippling	
For single unreinforced webs	0.75
For I-sections	0.80
(d) Concentrically loaded compression members	0.85
(e) Combined axial load and bending	
ϕ_c	0.85
ϕ	
For using Section C3.1.1	0.90-0.95
For using Section C3.1.2	0.90
(f) Cylindrical tubular members	
Bending strength	0.95
Axial compression	0.85
(g) Wall studs and wall stud assemblies	
Wall studs in compression	0.85
Wall studs in bending	
For sections with stiffened compression flanges	0.95
For sections with unstiffened compression flanges	0.90

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

$$\phi R_n = c(1.2D_n + 1.6L_n) = (1.2D_n/L_n + 1.6)cL_n \qquad (11)$$

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

By assuming $D_n/L_n = 1/5$, Eqs. 11 and 9 can be rewritten as follows:

$$R_n = 1.84(cL_n/\phi) \qquad (12)$$

$$Q_m = (1.05D_n/L_n + 1)cL_n = 1.21cL_n \qquad (13)$$

Therefore,

$$R_m/Q_m = (1.521/\phi)(R_m/R_n) \qquad (14)$$

The ϕ factors can be computed from Eq. 14 and the following equation by using $V_Q = 0.21$:

$$\text{Target } \beta_o = \frac{\ln(R_m/Q_m)}{\sqrt{V_R^2 + V_Q^2}} \qquad (15)$$

III. DESIGN OF MEMBERS

A. Yield Point

The following statistical data (mean values and coefficients of variation) on material and cross-sectional properties were develpoed in Refs. 19 and 20 for use in the derivation of the resistance factors ϕ:

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

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

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

$$F_m = 1.00; \quad V_F = 0.05$$

The subscript m refers to mean values. The symbol V stands for coefficient of variation. The symbols M and F are, respectively, the ratio of the mean-to-the nominal material property or cross-sectional property; and F_y, F_{ya}, and F_u are, 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.

B. Tension Members

The resistance factor of $\phi = 0.95$ used for tension member design was derived from the procedure described in Section II.A of this paper and a selected β_o value of approximately 2.5. In the determination of the resistance factor, the following formulas were used for R_m and R_n:

$$R_m = A_n(F_y)_m \qquad (16)$$

$$R_n = A_nF_y \qquad (17)$$

$$\text{i.e. } R_m/R_n = (F_y)_m/F_y \qquad (18)$$

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

$$V_R = \sqrt{V_M{}^2 + V_F{}^2 + V_P{}^2} = 0.11 \qquad (19)$$

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

C. Flexural Members

i) 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 the limit state is lateral-torsional buckling.

a) Nominal Section Strength

The bending strength of beams with a compression flange having 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. Refs. 2, 18, 27, and 28 provide an extensive treatment of the background research.

The experimental bases for the post-buckling strengths of cold-formed beams were examined in Refs. 11 and 18, 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 $\phi = 0.95$ and 0.90 for sections with stiffened compression flanges and unstiffened compression flanges, respectively, and for a dead-to-live load ratio of 1/5 for different cases are listed in Table 2. It can be seen that the β values vary from 2.53 to 4.05. In Table 2, the values of M_m, V_M, F_m and V_F are the values presented in Sec. III.A of this paper for the material strength.

b) Lateral Buckling Strength

There are not many test data on laterally unsupported cold-formed beams. The available test results are summarized in Ref. 11, and they are compared with predictions from AISI design formulas, theoretical formulas and SSRC formulas.

The statistical data used in Ref. 11 are listed in Table 3. The symbol P is the ratio of the tested capacity to the predicted value, M is the ratio of the actual to the specified value of the modulus of elasticity, and F is the ratio of the actual to the nominal sectional properties.

Using the recommended resistance factor $\phi = 0.90$, the values of β vary from 2.35 to 3.8. It should 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.

Table 2
Computed Safety Index β for Section Bending Strength of Beams
Based on Initiation of Yielding

Case	No. of Tests	M_m	V_M	F_m	V_F	P_m	V_P	β
Stiffened Compression Flanges ($\phi = 0.95$)								
FF. FW.	8	1.10	0.10	1.0	0.05	1.11	0.04	2.76
PF. FW.	30	1.10	0.10	1.0	0.05	1.11	0.09	2.65
PF. PW.	5	1.10	0.10	1.0	0.05	1.08	0.09	2.53
Unstiffened Compression Flanges ($\phi = 0.90$)								
FF. FW.	3	1.10	0.10	1.0	0.05	1.43	0.04	4.05
PF. FW.	40	1.10	0.10	1.0	0.05	1.12	0.14	2.67
PF. PW.	10	1.10	0.10	1.0	0.05	1.03	0.06	2.66

Note: FF. = Fully effective flanges
      PF. = Partially effective flanges
      FW. = Fully effective webs
      PW. = Partially effective webs
      For details, see Ref. 11.

Table 3
Computed Safety Index β for Lateral Buckling Strength of Bending
(ϕ = 0.90)

Case	No. of Tests	M_m	V_M	F_m	V_F	P_m	V_P	β
1	47	1.0	0.06	1.0	0.05	2.52	0.31	3.79
2	47	1.0	0.06	1.0	0.05	1.24	0.19	2.48
3	47	1.0	0.06	1.0	0.05	1.18	0.19	2.35
4	47	1.0	0.06	1.0	0.05	1.80	0.22	3.53
5	47	1.0	0.06	1.0	0.05	1.88	0.21	3.80

Note: Case 1 = AISI approach
Case 2 = Theoretical approach with J = 0.0026 in.[4]
Case 3 = SSRC approach with J = 0.0026 in.[4]
Case 4 = Theoretical approach with J = 0.0008213 in.[4]
Case 5 = SSRC approach with J = 0.0008213 in.[4]

Table 4

Range of h/t Ratio	F.S. for Allowable Load Design	ϕ Factor computed by Eq. 27	Recommended ϕ Factor
$h/t \leq \sqrt{Ek_v/F_y}$	1.44	1.06	1.00
$\sqrt{Ek_v/F_y} \leq h/t \leq 1.415\sqrt{Ek_v/F_y}$	1.67	0.92	0.90
$h/t > 1.415\sqrt{Ek_v/F_y}$	1.71	0.90	0.90

ii) 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.:

$$V_n = A_w \tau_y = A_w F_y/\sqrt{3} = 0.577F_y ht \qquad (20)$$

in which A_w is the area of the beam web computed by (hxt), 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.:

$$V_n = A_w \tau_{cr} = \frac{k_v \pi^2 E A_w}{12(1-\mu^2)(h/t)^2} \qquad (21)$$

in which τ_{cr} is the critical shear buckling stress in the elastic range, k_v is the shear buckling coefficient, E is the modulus of elasticity, μ is the Poisson's ratio, h is the web depth, and t is the web thickness. By using $\mu = 0.3$, the shear strength, V_n, can be determined as follows:

$$V_n = 0.905Ek_v t^3/h \qquad (22)$$

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

$$V_n = 0.64t^2\sqrt{k_v F_y E} \qquad (23)$$

In view of the fact that the appropriate test data on shear are not available, the ϕ 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,

$$(R_n)_{LRFD} = (R_n)_{ASD} \qquad (24)$$

Since $\qquad (R_n)_{LRFD} \geq c(1.2D_n+1.6L_n)/\phi \qquad (25)$

$$(R_n)_{ASD} \geq c(F.S.)(D_n+L_n) \qquad (26)$$

the resistance factors can be computed from the following formula:

$$\phi = \frac{1.2D_n+1.6L_n}{(F.S.)(D_n+L_n)}$$

$$= \frac{1.2(D_n/L_n)+1.6}{(F.S.)(D_n/L_n+1)} \qquad (27)$$

By using a dead-to-live load ratio of $D_n/L_n = 1/5$, the ϕ factors computed from the above equation are listed in Table 4 for three different ranges of

h/t ratios. 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.

iii) Web Crippling Strength

   The nominal ultimate concentrated load or reaction, P_n, is determined by
the allowable load given in Section C3.4 of the AISI ASD Specification times
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 Table 5 (see Table 76 of Ref. 11).
By using β_o = 2.5, the resistance factors ϕ = 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 are also given in Table 5.

iv) Combined Bending and Web Crippling Strength

   A total of 551 tests were calibrated for combined bending and web crip-
pling strength. Six different cases were studied. 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 in Table 6.

D. Concentrically Loaded Compression Members

   The available experimental data on cold-formed steel concentrically
loaded compression members were evaluated in Ref 11. 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 pre-
diction models as were employed in the formulation of the AISI ASD Specifi-
cation. 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
failure modes. The resistance factor ϕ_c = 0.85 was selected on the basis
of the statistical data given in Ref. 11. The corresponding safety indices
vary from 2.39 to 3.34. A summary of the information is given in Table 7.

   The safety indices were determined from Eq. 2 for a D_n/L_n ratio of 1/5.
Different ϕ factors could have been used for different cases.

Table 5
Computed Safety Index β for Web Crippling Strength of Beams

Case	No. of Tests	M_m	V_M	F_m	V_F	P_m	V_P	β
Single, Unreinforced Webs ($\phi = 0.75$)								
1(SF)	68	1.10	0.10	1.0	0.05	1.00	0.12	3.01
1(UF)	30	1.10	0.10	1.0	0.05	1.00	0.16	2.80
2(UMR)	54	1.10	0.10	1.0	0.05	0.99	0.11	3.02
2(CA)	38	1.10	0.10	1.0	0.05	0.86	0.14	2.36
2(SUM)	92	1.10	0.10	1.0	0.05	0.94	0.14	2.67
3(UMR)	26	1.10	0.10	1.0	0.05	0.99	0.09	3.11
3(CA)	63	1.10	0.10	1.0	0.05	1.72	0.26	3.80
3(SUM)	89	1.10	0.10	1.0	0.05	1.51	0.34	2.95
4(UMR)	26	1.10	0.10	1.0	0.05	0.98	0.10	3.03
4(CA)	70	1.10	0.10	1.0	0.05	1.04	0.26	2.39
4(SUM)	96	1.10	0.10	1.0	0.05	1.02	0.23	2.49
I-Sections ($\phi = 0.80$)								
1	72	1.10	0.10	1.0	0.05	1.10	0.19	2.74
2	27	1.10	0.10	1.0	0.05	0.96	0.13	2.57
3	53	1.10	0.10	1.0	0.05	1.01	0.13	2.76
4	62	1.10	0.10	1.0	0.05	1.02	0.11	2.89

Note: Case 1 = End one-flange loading
　　　Case 2 = Interior one-flange loading
　　　Case 3 = End two-flange loading
　　　Case 4 = Interior two-flange loading
　　　SF　　 = Stiffened flanges
　　　UF　　 = Unstiffened flanges
　　　UMR　 = UMR and Cornell tests only
　　　CA　　 = Canadian tests only
　　　SUM　 = Combine UMR and Canadian tests together

Table 6
Computed Safety Index β for Combined Bending and Web Crippling

Case	No. of Tests	M_m	V_M	F_m	V_F	P_m	V_P	β
Single, Unreinforced Webs (Interior one-flange loading) (Based on $\phi_w = 0.75$)								
1	74	1.10	0.10	1.0	0.05	1.01	0.07	3.27
2	202	1.10	0.10	1.0	0.05	0.87	0.13	2.45
3	103	1.10	0.10	1.0	0.05	0.95	0.10	2.91
4	66	1.10	0.10	1.0	0.05	1.03	0.18	2.79
5	445	1.10	0.10	1.0	0.05	0.94	0.14	2.68
I-Sections (Interior one-flange loading) (Based on $\phi_w = 0.80$)								
1	106	1.10	0.10	1.0	0.05	1.06	0.12	2.99

Note: Case 1 = UMR and Cornell tests only
      Case 2 = Canadian brake-formed section tests only
      Case 3 = Canadian roll-formed section tests only
      Case 4 = Hoglund's tests only
      Case 5 = Combine all tests together

Table 7
Computed Safety Index β for Concentrically Loaded Compression Member
(φ = 0.85)

Case	No. of Tests	M_m	V_M	F_m	V_F	P_m	V_P	β
1	5	1.10	0.10	1.0	0.05	1.15	0.10	3.13
2	24	1.10	0.10	1.0	0.05	1.05	0.08	2.89
3	15	1.10	0.10	1.0	0.05	1.06	0.07	2.93
4	3	1.10	0.10	1.0	0.05	1.11	0.08	3.11
5	28	1.10	0.10	1.0	0.05	1.05	0.11	2.76
6	25	1.10	0.10	1.0	0.05	1.22	0.22	2.72
7	9	1.00	0.06	1.0	0.05	0.96	0.04	2.39
8	41	1.10	0.10	1.0	0.05	1.20	0.10	3.34
9	18	1.10	0.10	1.0	0.05	1.03	0.08	2.81
10	12	1.10	0.11	1.0	0.05	1.06	0.11	2.77
11	8	1.00	0.06	1.0	0.05	1.15	0.11	2.92
12	30	1.10	0.10	1.0	0.05	1.08	0.15	2.68
13	14	1.10	0.10	1.0	0.05	1.08	0.08	3.00
14	32	1.10	0.10	1.0	0.05	1.08	0.11	2.89

Note: Case 1 = Stub columns having unstiffened flanges with fully
                effective widths
      Case 2 = Stub columns having unstiffened flanges with
                partially effective widths
      Case 3 = Thin plates with partially effective widths
      Case 4 = Stub columns having stiffened compression flanges
                with fully effective flanges and webs
      Case 5 = Stub columns having stiffened compression flanges
                with partially effective flanges and fully effective
                webs
      Case 6 = Stub columns having stiffened compression flanges
                with partially effective flanges and partially
                effective webs
      Case 7 = Long columns having unstiffened compression flanges
                subjected to elastic flexural buckling
      Case 8 = Long columns having unstiffened compression flanges
                subjected to inelastic flexural buckling
      Case 9 = Long columns having stiffened compression flanges
                subjected to inelastic flexural buckling
      Case 10 = Long columns subjected to inelastic flexural
                buckling (include cold-work)
      Case 11 = Long columns subjected to elastic torsional-flexural
                buckling
      Case 12 = Long columns subjected to inelastic torsional-
                flexural buckling
      Case 13 = Stub columns with circular perforations
      Case 14 = Long columns with circular perforations

E. Combined Axial Load and Bending

The LRFD Specification provide the same interaction equations as the 1986 Edition of the AISI ASD Specification.

A total of 144 tests were calibrated for combined axial load and bending. Nine different cases were studied according to the types of sections, the stable conditions and the loading conditions. Based on $\phi_c = 0.85$, $\phi = 0.95$ or 0.90 for nominal section strength, and $\phi = 0.90$ for lateral buckling strength, the values of safety indices vary from 2.7 to 3.34 as given in Table 8.

IV. COMPARISONS OF ASD AND LRFD CRITERIA

The design equation used for the LRFD criteria is based on dead and live loads as follows:

$$\phi R_n \geq 1.2D_n + 1.6L_n \tag{28}$$

where

D_n = nominal dead load
L_n = nominal live load

For the purpose of comparison, 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:

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

$$\phi R_n \geq (1.2D_n/L_n + 1.6)[(D_n + L_n)/(D_n/L_n + 1)]$$

Therefore,

$$D_n + L_n \leq \frac{R_n}{(1.2D_n/L_n + 1.6)/[\phi(D_n/L_n + 1)]} \tag{29}$$

From Eq. 29, the factor of safety against the nominal resistance used in the LRFD criteria is as follows:

$$(F.S.)_{LRFD} = (1.2D_n/L_n + 1.6)/[\phi(D_n/L_n + 1)] \tag{30}$$

The allowable load for ASD is based on a factor of safety of the nominal resistance as shown in Eq. 31.

$$D_n + L_n \leq R_n/(F.S.)_{ASD} \tag{31}$$

Therefore, based on Eqs. 29 and 31, the allowable load ratio is as follows:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = \phi(F.S.)_{ASD} \frac{D_n/L_n + 1}{1.2D_n/L_n + 1.6} \tag{32}$$

Table 8
Computed Safety Index β for Combined Axial Load and Bending
(based on $\phi_c = 0.85$)

Case	No. of Tests	M_m	V_M	F_m	V_F	P_m	V_P	β
1	18	1.05	0.10	1.0	0.05	1.04	0.07	2.70
2	13	1.05	0.10	1.0	0.05	1.05	0.08	2.72
3	33	1.05	0.10	1.0	0.05	1.10	0.09	2.86
4	18	1.05	0.10	1.0	0.05	1.15	0.10	2.96
5	6	1.05	0.10	1.0	0.05	1.16	0.13	2.87
6	17	1.05	0.10	1.0	0.05	1.12	0.09	2.92
7	10	1.05	0.10	1.0	0.05	1.23	0.08	3.34
8	17	1.05	0.10	1.0	0.05	1.09	0.08	2.86
9	12	1.05	0.10	1.0	0.05	1.11	0.11	2.79

Note: Case 1 = Locally stable beam-columns, hat sections of Pekoz
and Winter (1967)(Ref. 17)
Case 2 = Locally unstable beam-columns, lipped channel
sections of Thomasson (1978)(Ref. 26)
Case 3 = Locally unstable beam-columns, lipped channel
sections of Loughlan (1979)(Ref. 14)
Case 4 = Locally unstable beam-columns, lipped channel
sections of Mulligan and Pekoz (1983)(Ref. 16)
Case 5 = Locally stable beam-columns, lipped channel sections
of Loh and Pekoz (1985)(Ref. 15) with $e_x \neq 0$ and
$e_y = 0$
Case 6 = Locally stable beam-columns, lipped channel sections
of Loh and Pekoz (1985) with $e_x = 0$ and $e_y \neq 0$
Case 7 = Locally stable beam-columns, lipped channel sections
of Loh and Pekoz (1985) with $e_x \neq 0$ and $e_y \neq 0$
Case 8 = Locally unstable beam-columns, lipped channel
sections of Loh and Pekoz (1985) with $e_x = 0$ and $e_y \neq 0$
Case 9 = Locally unstable beam-columns, lipped channel
sections of Loh and Pekoz (1985) with $e_x \neq 0$ and $e_y \neq 0$

Equation 32 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 inter-action equations. Figure 1 shows the allowable load ratio versus dead-to-live load ratio for tension members and bending strength of beams with stiffened compression flanges. Figure 2 shows the allowable moment ratio versus dead-to-live load ratio for bending strength of beams with unstiffened compression flanges and lateral buckling of beams. For other structural members, the curves are similar as Figures 1 and 2 except that they are shifted up or down depending on the factor of safety and φ factor. Detailed information can be found in Ref. 13. As shown in Figures 1 and 2, the LRFD criteria is 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 Spec-ification. Due to the complexity of the design equations for combined loads, specific examples were chosen for comparison.

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 con-sidered to be a parameter in the equations for determining the allowable loads.

Figure 3 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 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 Figure 3 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 sup-ports. 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 in-creases as the end moment ratio increases as shown in Figure 3. It can be seen that for D/L < 1/3, the allowable load ratios computed for $C_m = 0.85$ are larger than those for $C_m = 1.0$.

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

Figure 4 shows the allowable load ratio versus eccentricity for the 5 ft long channel with D/L = 0.5 and F_y = 33 ksi. 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 relationshop holds for both posi-tive and negative eccentricities. The top line in Figure 4 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 Ref. 13.

V.   SUMMARY

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.

VI.   ACKNOWLEDGEMENTS

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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 the actual section modulus to the 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 the web measured along the plane of the 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 the yield point to the minimum specified value
M_{no} = nominal moment capacity based on section strength
M_T = applied unfactored bending moment at each end of the 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
S_e = elastic section modulus based on the 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
ϕ = resistance factor

ϕ_c = resistance factor for concentrically loaded compression member
ϕ_w = resistance factor for web crippling strength
μ = Poisson's ratio
σ = standard deviation
τ_{cr} = critical shear buckling stress
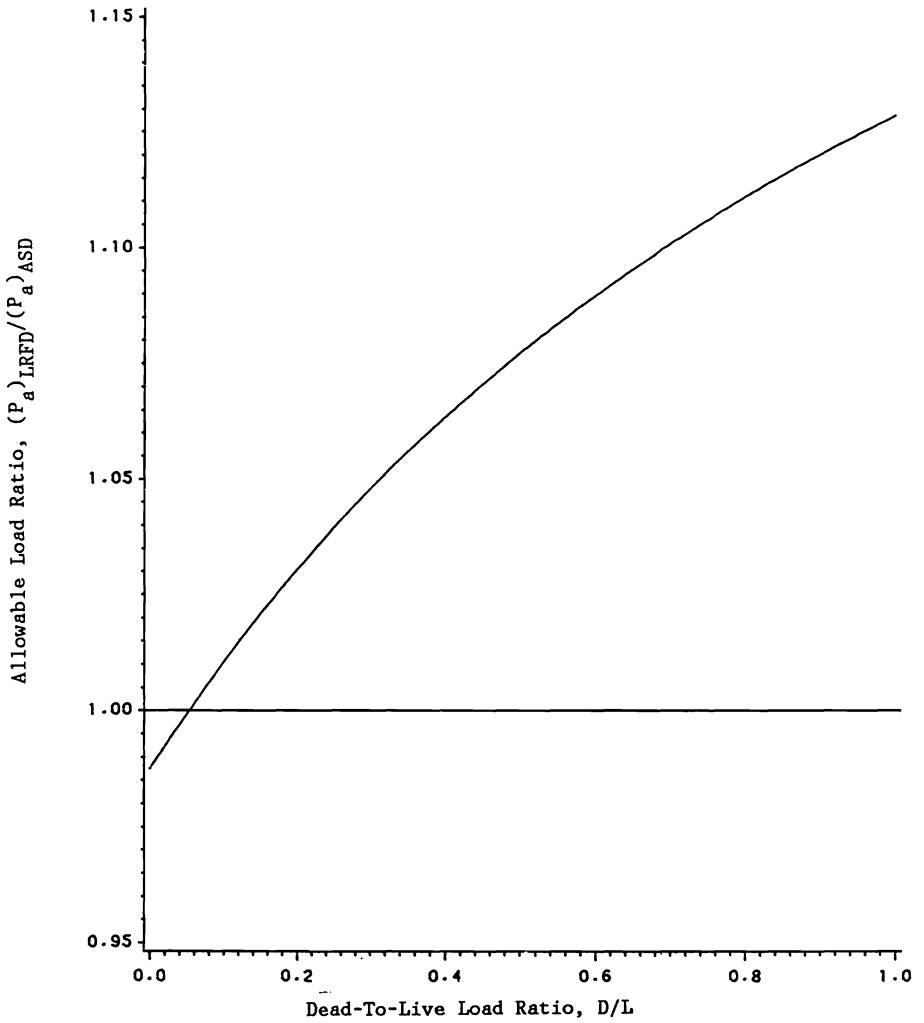τ_y = yield point in shear

Figure 1.   Allowable Load Ratio vs. D/L Ratio for Tension Members and
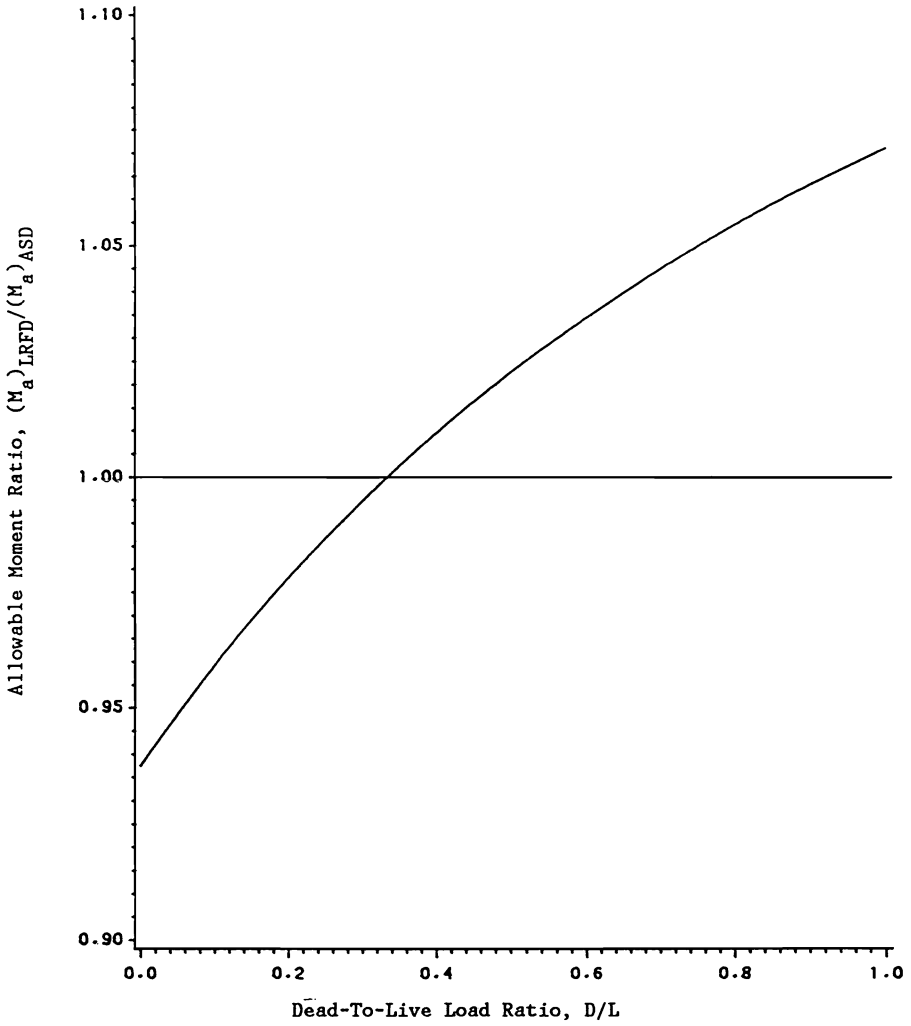Bending Strength of Beams With Stiffened Compression flanges

Figure 2. Allowable Moment Ratio vs. D/L Ratio for Bending Strength
of Beams With Unstiffened Compression Flanges and Lateral
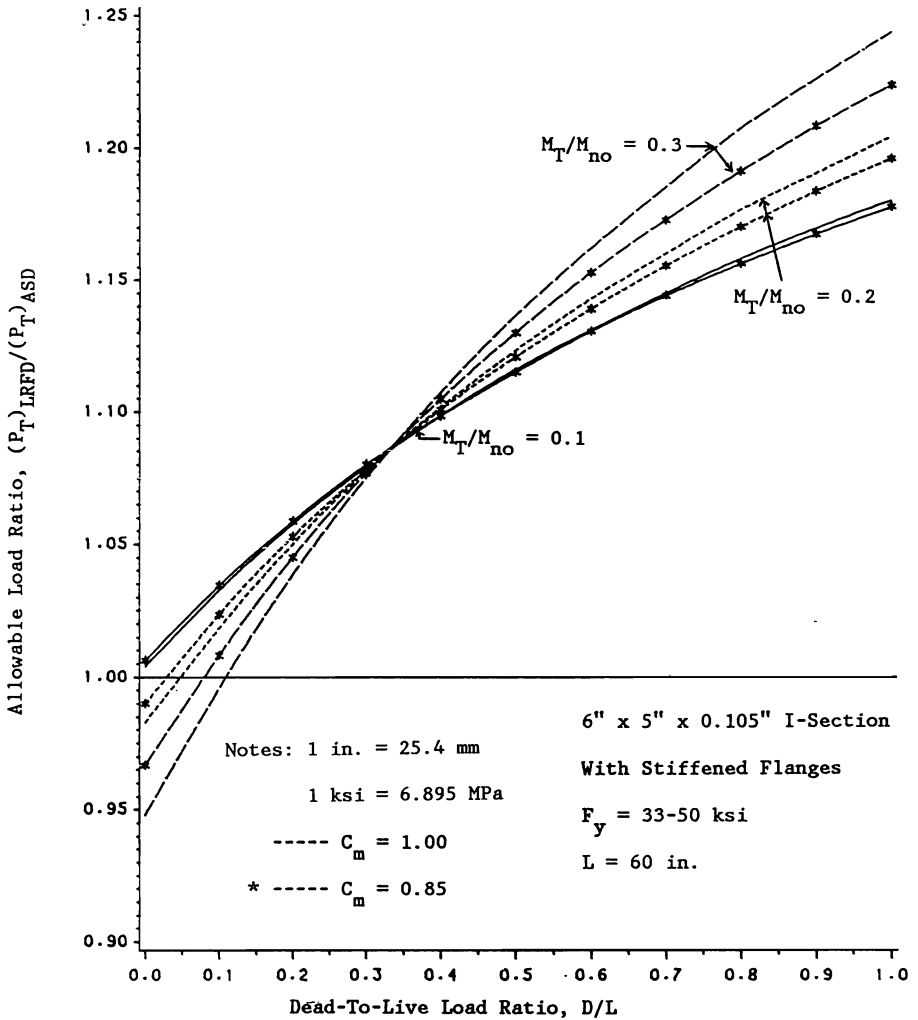Buckling of Beams

Figure 3. Allowable Load Ratio vs. D/L Ratio for Doubly Symmetric Beam-Columns
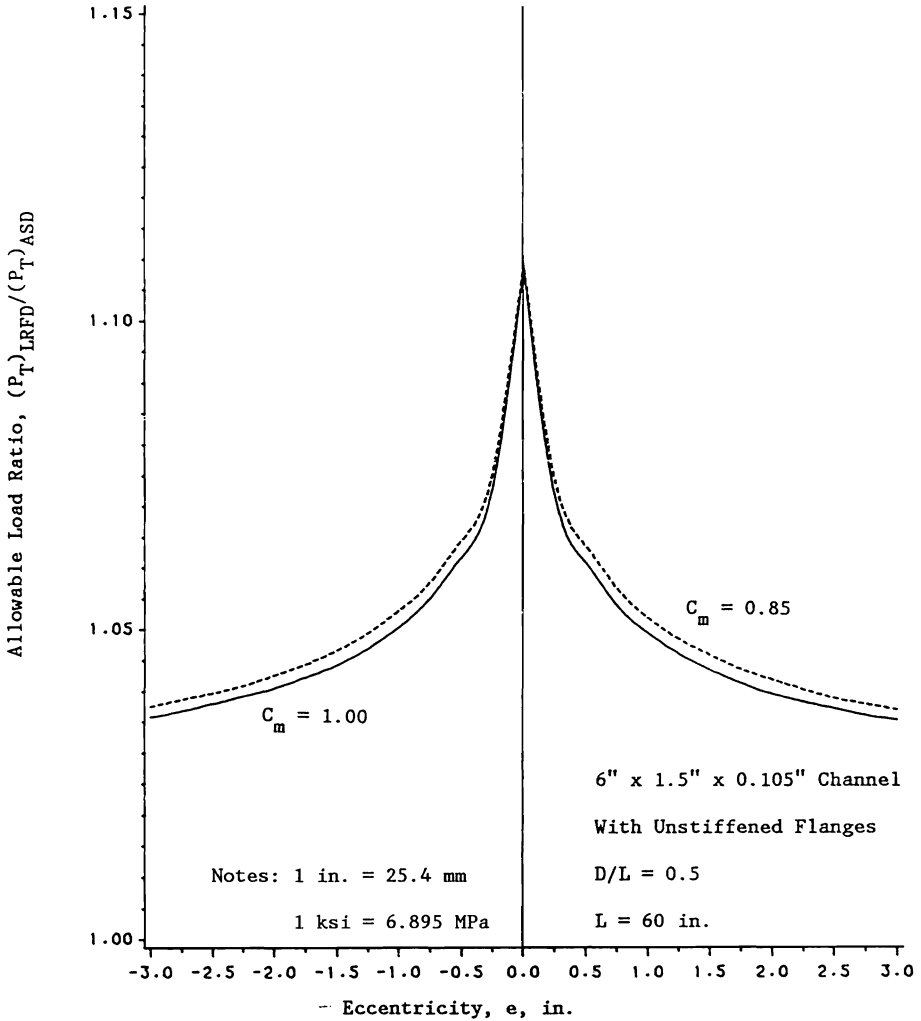
Figure 4.   Allowable Load Ratio vs. D/L Ratio for Singly Symmetric
            Beam-Columns