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Load and resistance factor design of cold-formed steel reliability based criteria for cold-formed steel members

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LOAD AND RESISTANCE FACTOR DESIGN OF COLD-FORMED STEEL

RELIABILITY BASED CRITERIA FOR COLD-FORMED STEEL MEMBERS

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PREFACE

This report is based on a dissertation presented to the Faculty of the Graduate School of the University of Missouri-Rolla (UMR) in partial fulfillment of the requirements for the degree of Doctor of Philosophy in Civil Engineering.

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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 and other countries. In this approach, member forces, or moments, determined on the basis of working loads should not exceed the allowable values. The allowable value is used to prevent the possible structural failure by using an appropriate factor of safety selected primarily on the basis of engineering judgment and long-time experience.

Recently, in the United States, the concepts of risk and reliability analysis have been successfully applied to the Load and Resistance Factor Design criteria for steel buildings using hot-rolled shapes, and built-up members fabricated from steel plates.

In order to develop reliability based design criteria for cold-formed steel members, a joint research project entitled "Load and Resistance Factor Design (LRFD) of Cold-Formed Steel" was conducted at the University of Missouri-Rolla, Washington University, and the University of Minnesota. This study included the selection of a reliability analysis model; the evaluation of load factors; the calibration of the design provisions; the determination of resistance factors; the comparative study of design methods for cold-formed steel; and the preparation of the LRFD design manual for cold-formed steel. However, only the development of the reliability based design criteria, and the comparative study of design methods for cold-formed steel are discussed in this dissertation.

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I. INTRODUCTION

A. GENERAL REMARKS

The fundamental role of probability theory in safety and performance analysis is widely recognized in all branches of engineering. Probability theory provides a more accurate engineering representation of reality. Many leading civil engineers in different countries have studied the statistical nature of loads and material properties. It has been demonstrated that the uncertainty in applied forces and structural resistances implies uncertainties in structural performances, which can be analyzed rationally only with probability theory. The conclusion is that, if structural safety is to be placed in a position where it can be discussed quantitatively, it must be treated probabilistically[1].

In the design of steel buildings, the "Allowable Stress Design (ASD)" method has long been used for 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. The allowable value permitted in the specification is used to prevent the possible structural failure by using an appropriate factor of safety selected primarily on the basis of engineering judgment and long-time experience.

The use of cold-formed steel members in building construction began in about the 1850s in both the United States and Great Britain. However, such steel members were not widely used in buildings until around 1940.

In the United States, the first Specification for the Design of Light Gage Steel Structural Members was issued by the American Iron and Steel Institute (AISI) in 1946[2]. It was revised in 1956, 1960, 1962, 1968, 1980, and 1986. The background for the establishment of various design provisions of the Specification is extensively documented in the Commentary on the AISI Specification[3,4] and other references[5,6]. In the application of the AISI Specification, cold-formed steel structural members are currently designed on the basis of the allowable stress design method.

Recently, the concepts of risk and reliability analysis have been successfully applied to the design criteria for steel buildings using hot-rolled shapes and built-up members fabricated from steel plates, namely, Load and Resistance Factor Design criteria[7-19]. 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 same concept is known as "Limit States Design (LSD)" in other countries, and has been used in Canada and Europe for the design of steel structural members[20,21].

In order to develop the reliability based design criteria for cold-formed steel members, a joint research project entitled "Load and Resistance Factor Design (LRFD) of Cold-Formed Steel" was conducted at the University of Missouri-Rolla, Washington University, and the University of Minnesota under the sponsorship of American Iron and Steel Institute. Initial results were presented in several publications[22-32]. Based on the 1986 Edition of the AISI ASD Specification[33], additional research work was conducted and summarized in References 34-39. The revised LRFD specification for cold-formed steel structural members with commentary[38] has been prepared for consideration of the American Iron and Steel Institute. This proposed document contains six sections for designing cold-formed steel structural members and connections. The background information will be discussed in the subsequent chapters.

B. PURPOSE OF INVESTIGATION

The main objective of this investigation was to develop reliability based design criteria for cold-formed steel members. The LRFD format was chosen because of the following advantages:

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, therefore, these differences can be accounted for by use of multiple factors.
2) By using probability theory, designs can ideally achieve a more consistent reliability.

As the first step, the existing LRFD formats have been carefully reviewed in order to determine their suitability for cold-formed steel structures. The preliminary investigation has shown that it is possible to formulate a practical LRFD design method for cold-formed steel structural members.

To develop the LRFD criteria for cold-formed steel, the statistical data for mechanical properties, sectional properties, and the test-to-prediction ratios of structural members and connections were collected and evaluated. Based on the available data and engineering judgment, the representative values of target reliability index (target safety index) were selected. Following the selection of appropriate target reliability indices, the major load factors and load combinations were adopted from the ANSI Standard[40] and the resistance factors were determined on the basis of the mean-value first-order second-moment (FOSM) reliability analysis.

C. SCOPE OF INVESTIGATION

The LRFD criteria for cold-formed steel structural members were based on the limit states of strength and serviceability of thin-walled steel structures. The mean-value first-order second-moment probability analysis and advanced probability analysis were used as basic methods in the development of the LRFD criteria. Statistical data used for this work were obtained from the measured mechanical and sectional properties and from test-to-prediction ratios of the available experimental results.

As the first step of the investigation, numerous technical papers and research reports[1,7-18,41-56] relative to the theoretical concepts of the structural reliability have been collected and reviewed. Section II contains a summary of literature review. Also included in this section are the statistical data on material properties and sectional properties, determination of target reliability index (target safety index), and formulas for the determination of structural reliability.

Section III presents the determination of load factors and load combinations to be used in the LRFD criteria for cold-formed steel members. Load factors and load combinations recommended in the 1982 ANSI Code[40] were adopted and modified based on the consideration of special circumstances inherent in cold-formed steel structures.

For the purpose of facilitating the steps used in the calibration of various allowable stress design provisions of the AISI Specification, the calibration procedures have been formulated in Section IV. All the determination of resistance factors as well as reliability indices for various design provisions discussed in Section V were based on the formulas derived in this section.

Section V contains the development of LRFD design criteria for cold-formed steel members and connections. Section V.B presents the development of the LRFD criteria for tension members. The developments and calibrations of the LRFD criteria for flexural members, concentrically loaded compression members, combined axial load and bending are presented in Sections V.C through V.E. The development of the LRFD criteria for stiffeners and wall studs are given in Sections V.F and V.G, respectively. For welded and bolted connections, the developments and calibrations of the LRFD criteria are included in Sections V.H and V.I, respectively.

The calibrations of design provisions discussed in Section V are mainly based on the available test data. When tests for determining structural performance are needed, the calibration procedure must be modified to consider the influence due to the small number of tests. Section VI presents the evaluation procedure of the LRFD criteria on tests for special cases.

Section VII contains the comparative study of the design methods for cold-formed steel. The main purpose of this section is to study, and compare, the LRFD criteria for cold-formed steel with the existing allowable stress design (ASD) criteria included in the 1986 Specification for the Design of Cold-Formed Steel Structural Members[33]. This comparison involves studies of different variables used for the design of various types of structural members and discussions of different load-carrying capacities determined by these two methods.

Finally, a summary of this study is presented and a brief conclusion is drawn in Section VIII.

II. REVIEW OF LITERATURE

A. GENERAL

In the United States and England, two professional committees were appointed shortly after World War II by the American Society of Civil Engineers (ASCE) and Institution of Civil Engineers (ICE) to study the safety of structures. These investigations indicated that in structural design many design parameters such as material properties and loads should be considered as random variables instead of deterministic variables. Consequently, a better approach for structural safety can be achieved when these parameters are treated by using theories of probability and statistics together with engineering judgments. Based on their research findings, the ASCE and ICE committees issued two reports on this subject. The most well-established paper on the basic concept of the structural safety was presented by Freudental[57]. These reports led to the development of practical reliability-based design criteria.

B. HISTORICAL DEVELOPMENT OF RELIABILITY-BASED DESIGN CRITERIA

A large number of researchers have contributed to the development of reliability-based safety analysis and design procedures. The application of probability and statistics theories on structural design has increased rapidly since 1965. Based on the first-order probabilistic theory, the LRFD methods were developed by Cornell, Lind, Rosenblueth, Esteva, Ravindra, and Heaney[1,58-60] in the 1960s. An equivalent approach was also developed by Ang[61,62].

Many research projects were conducted in the seventies for the purpose of developing the LRFD criteria for structural design. Live load models were studied by Pier, Cornell, Corotis, and Doshi[63,64] and the live loads in office buildings were surveyed and analyzed by Mitchell, Woodgate, Ellingwood, and Culver[65,66]. Based on the data of Mitchell and Woodgate, and the model of Pier and Cornell, the statistics of live loads were documented by McGuire and Cornell[67]. In Reference 68, wind and snow loads were studied by Ravindra, Cornell, and Galambos. The material properties to be used in the LRFD criteria were reported for reinforced concrete and hot-rolled steel in References 69 and 70, respectively. The LRFD criteria for reinforced concrete beams were also included in Reference 69. By using the study of reinforced concrete presented in Reference 69, Ellingwood developed the LRFD criteria for reinforced concrete beam-columns[71]. In Reference 1, the LRFD formats for reinforced concrete were proposed by Cornell.

For steel structures using hot-rolled shapes, the development of the LRFD criteria were summarized in numerous publications. References 15 and 72 through 76 describe the development of the LRFD criteria for tension members, beams, beam-columns, plate girders, composite beams, and connectors. Based on the advanced reliability analysis method, load factors and load combinations were developed by Ellingwood, Galambos, MacGregor, and Cornell[16-18] for use in the LRFD criteria; regardless of the type of structures or materials because the applied loads did not vary[40]. Recently, the LRFD criteria for steel buildings using hot-rolled shapes and built-up members fabricated from steel plates have been developed and used[19] in the United States. These aforementioned research findings served as the basis for the LRFD criteria for cold-formed steel.

C. DESIGN FORMAT OF LOAD AND RESISTANCE FACTOR DESIGN CRITERIA

The current method of designing cold-formed steel structural members, as presented in the 1986 AISI Specification[33], is based on the Allowable Stress Design method. In this approach, the allowable load or moment is determined by dividing the nominal load or moment at a specified limit state by a factor of safety. The factor of safety is based on an engineering judgment and past experience to ensure the safety of the structure.

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 laboratory, and they have been thoroughly investigated through analytical and experimental research. The background for defining the limit states is extensively documented in the Commentary on the AISI Specification[3,4] and other references[5,6], and a continuing research effort provides further improvement in understanding them.

In ASD, 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 experience it has been established that the present factors of safety provide a satisfactory design.

The allowable stress design method employs only one factor of safety for a limit state. The use of multiple load factors in the Load and Resistance Factor Design provides a refinement in the design which can better account for the different degrees of the uncertainties and variabilities of the design parameters. The design format of LRFD is expressed by the following criterion:

$$\phi R_n \geq \Sigma \gamma_i Q_i \qquad (2.1)$$

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 uncertainties and variabilities inherent in the R_n, and 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 which 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, designs can ideally achieve a more consistent reliability. Thus LRFD provides the basis for a more rational, and refined design method, than is possible with the Allowable Stress Design method.

D. SELECTION OF RELIABILITY ANALYSIS MODEL

The conceptual framework for reliability-based design is provided by the reliability theory described by Freudenthal, Ang, Cornell, and others[45,77]. A mathematical model is first defined as follows which relates the resistance and load variables for the limit state of interest:

$$g(X_1, X_2, \ldots, X_n) = 0 \tag{2.2}$$

where

X_i = resistance or load variable

Failure occurs when $g < 0$ for any ultimate, or serviceability, limit state of interest. The safety is assured by assigning a small probability P_F to the event that the limit state will be reached, i.e.,

$$P_F = \int \ldots \int f_X(X_1, X_2, \ldots, X_n) dX_1 dX_2 \ldots dX_n \tag{2.3}$$

in which f_X is the joint probability density function for X_1, $X_2, \ldots$, and the integration is performed over the region where $g < 0$.

In structural reliability analysis, the joint probability density function f_X is seldom known precisely due to a general scarcity of data. In some cases, only the first- and second-order moments, i.e., mean and variance, may be known for individual variable. It is usually impractical to evaluate Eq. (2.3) numerically. These difficulties led to the development of the mean-value first-order second-moment reliability analysis model and the advanced reliability analysis model.

1. <u>Mean-Value First-Order Second-Moment Reliability Analysis Model</u>. Based on the first-order second-moment probabilistic theory, the random variables involved in reliability analysis can be characterized by their first and second moments. While any continuous mathematical form of the limit state equation is possible, it must be linearized at some point for the purpose of performing the reliability analysis. Linearization of the failure criterion defined by Eq. (2.2) leads to

$$Z \approx g(X_1^*, X_2^*, \ldots, X_n^*) + \Sigma(X_i - X_i^*)(\partial g/\partial X_i)_{X^*} \tag{2.4}$$

where $(X_1^*, X_2^*, \ldots, X_n^*)$ is the linearizing point. The reliability analysis then is performed with respect to this linearized version of Eq. (2.2). The key consideration is the selection of an appropriate linearizing point.

In the mean-value first-order second-moment reliability analysis model, the point $(X_1^*, X_2^*, \ldots, X_n^*)$ was set equal to the mean values $(\bar{X}_1, \bar{X}_2, \ldots, \bar{X}_n)$. Assuming the X-variables to be statistically uncorrelated, the mean and standard deviation in Z are approximated by[16,45]

$$\bar{Z} \approx g(\bar{X}_1, \bar{X}_2, \ldots, \bar{X}_n) \tag{2.5}$$

$$\sigma_Z \approx [\Sigma(\partial g/\partial X_i)_{\bar{X}_i}^{\;2} \sigma_{X_i}^{\;2}]^{1/2} \tag{2.6}$$

The reliability index (safety index) is defined by[16]

$$\beta = \bar{Z}/\sigma_Z \tag{2.7}$$

where β is a measure of the probability for function g less than zero.

Structural design consists of comparing nominal load effects Q to nominal resistance R, both Q and R are random parameters (see Figure 1).

A limit state is violated if $g = (R-Q) < 0$ and the possibility of this event ever occurring (probability of failure), P_F, is given as follow:

$$P_F = P(\text{failure}) = P(R-Q < 0) \qquad (2.8)$$

If the exact probability distributions of R and Q were known, then Eq. (2.8) could be exactly determined for any design. In general, the distributions of Q and R are not known, it is convenient to prescribe the distribution of ln(R/Q) to be normal, then the probability of failure, P_F, can be expressed as follow:

$$P_F = P(g < 0) = P[\ln(R/Q) < 0] \qquad (2.9)$$

Standarize the variable ln(R/Q), Eq. (2.9) can be rewritten as follows:

$$P_F = P\left[\frac{\ln(R/Q)-[\ln(R/Q)]_m}{\sigma_{\ln(R/Q)}} < -\frac{[\ln(R/Q)]_m}{\sigma_{\ln(R/Q)}}\right]$$

$$= P\left[U < -\frac{[\ln(R/Q)]_m}{\sigma_{\ln(R/Q)}}\right]$$

$$= F_U\left[-\frac{[\ln(R/Q)]_m}{\sigma_{\ln(R/Q)}}\right] \qquad (2.10)$$

where

U = standard variable with a zero mean and a unit standard deviation

$$= \frac{\ln(R/Q) - [\ln(R/Q)]_m}{\sigma_{\ln(R/Q)}} \qquad (2.11)$$

F_U = cumulative lognormal distribution

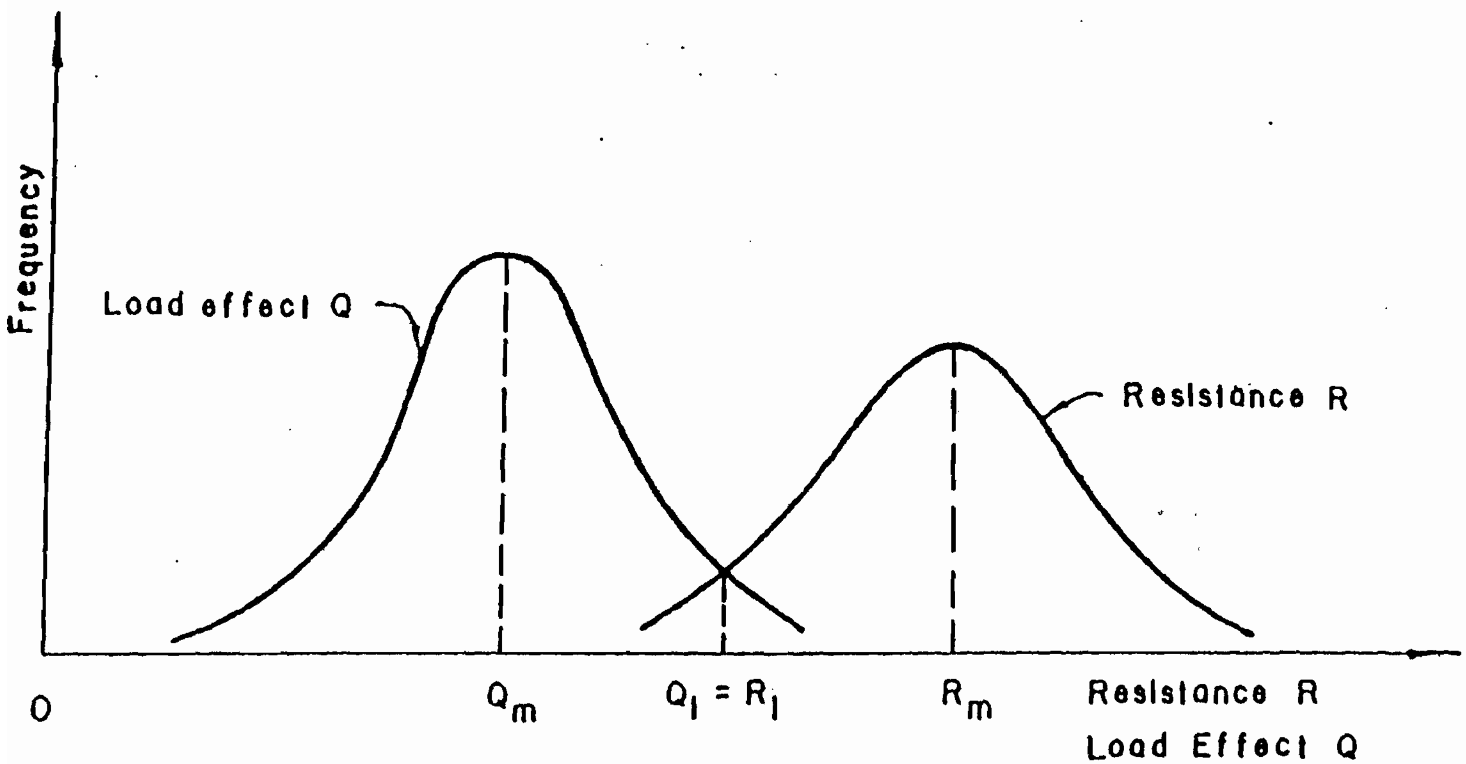

Figure 1 Definition of the Randomness of Q and R[28]

From Eq. (2.10) it can be seen that

$$\bar{Z} = [\ln(R/Q)]_m \tag{2.12}$$

$$\sigma_Z = \sigma_{\ln(R/Q)} \tag{2.13}$$

Applying Eqs. (2.7), (2.12), and (2.13), Eq. (2.10) can be rewritten as follow:

$$P_F = F_U(-\beta) \tag{2.14}$$

where

$$\beta = \frac{[\ln(R/Q)]_m}{\sigma_{\ln(R/Q)}} \tag{2.15}$$

Using Eqs. (2.5) and (2.6), Eqs. (2.12) and (2.13) can be rewritten as Eqs. (2.16) and (2.17), respectively.

$$\bar{Z} = [\ln(R/Q)]_m \approx \ln(R_m/Q_m) \tag{2.16}$$

$$\begin{aligned}\sigma_Z &= \sigma_{\ln(R/Q)} \\ &\approx \sqrt{(\sigma_R^2/R_m^2)+(\sigma_Q^2/Q_m^2)} \\ &\approx \sqrt{V_R^2+V_Q^2}\end{aligned} \tag{2.17}$$

Therefore, Eq. (2.15) can be rewritten as follow:

$$\beta = \frac{\ln(R_m/Q_m)}{\sqrt{V_R^2+V_Q^2}} \tag{2.18}$$

In the above equation, β is a relative measure of the safety for design. The higher the reliability index β, the smaller the probability of failure. By using the reliability index, the probability of failure is simply obtained from the cumulative lognormal distribution as shown in Eq. (2.14). From Figure 2 it can be seen that the calculated probability of failure, P_F, based on Eqs. (2.14) and (2.18) is the area under the normal curve beyond β standard deviations from the mean. This model provides a basis for quantitatively measuring structural reliability.

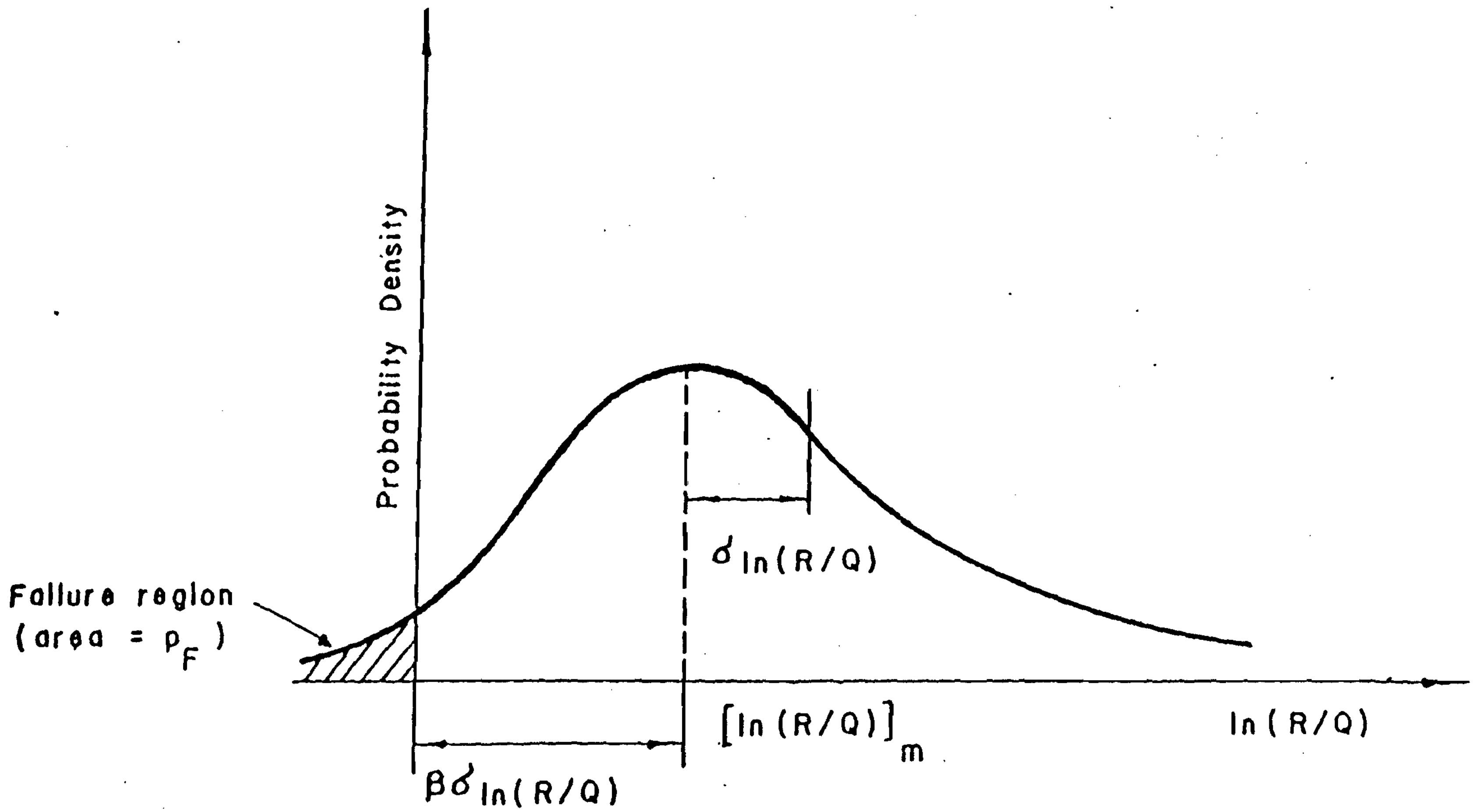

Figure 2 Definition of the Reliability Index β[28]

2. <u>Advanced Reliability Analysis Model</u>. The mean-value first-order second-moment reliability analysis model has some shortcomings[16]. First, because the function g is linearized at the mean values of the X-variables, errors might be induced when function g is nonlinear. Second, the mean value model fails to be invariant to different mechanically equivalent formulations of the same problem. This means that β depends on how the limit state is formulated. This is a problem not only for nonlinear forms of function g but even for certain linear forms, e.g., when the loads (or load effects) counteract one another. The lack of invariance arises because the linear expansions are taken at the mean value point. These problems may be avoided by linearizing function g at some point on the failure surface[50-52]. This is because function g and its partial derivations in Eq. (2.4) are independent of the problem formulated only on the surface g = 0. The advanced reliability analysis model shown herein is based on this approach.

With the limit state and its variables as given in Eq. (2.2), the variables X_i are first transformed to reduced variables with zero mean and unit variance as follows:

$$x_i = \frac{X_i - \overline{X}_i}{\sigma_{X_i}} \qquad (2.19)$$

In the space of reduced coordinates x_i, the limit state is

$$g_1(x_1, x_2, \ldots, x_n) = 0 \qquad (2.20)$$

with failure occurring when $g_1 < 0$. This is shown in Figures 3 and 4.

The reliability index β is defined as the shortest distance between the surface $g_1 = 0$ and the origin[16]. The point $(x_1^*, x_2^*, \ldots, x_n^*)$ on $g_1 = 0$ which corresponds to this shortest distance is referred to as the checking point (design point)[16,57]. The shortest distance between the surface $g_1 = 0$ and the origin can be determined by using Lagrange multiplier method as follow:

$$L(x_1,x_2,\ldots,x_n,\lambda) = d+\lambda g_1(x_1,x_2,\ldots,x_n) \qquad (2.21)$$

where

d = distance between a point $(x_1, x_2, \ldots, x_n)$ and origin

$$= \sqrt{x_1^2+x_2^2+\ldots+x_n^2} \qquad (2.22)$$

Equation (2.21) should satisfy Eq. (2.20) and the following equation:

$$\partial L/\partial x_i = 0 \qquad i = 1,2,\ldots,n \qquad (2.23)$$

i.e.,

$$(x_i/d)+\lambda(\partial g_1/\partial x_i) = 0 \qquad i = 1,2,\ldots,n \qquad (2.24)$$

For the purpose of simplification, assume only two variables x_1 and x_2 are involved. The following equations can be obtained from Eq. (2.24):

$$x_1/\sqrt{x_1^2+x_2^2}+\lambda(\partial g_1/\partial x_1) = 0 \qquad (2.25)$$

$$x_2/\sqrt{x_1^2+x_2^2}+\lambda(\partial g_1/\partial x_2) = 0 \qquad (2.26)$$

From Eqs. (2.25) and (2.26), λ can be determined as shown in Eq. (2.27).

$$\lambda = - x_1/[(\partial g_1/\partial x_1)\sqrt{x_1^2+x_2^2}]$$
$$= - x_2/[(\partial g_1/\partial x_2)\sqrt{x_1^2+x_2^2}] \qquad (2.27)$$

in which, x_2 can be expressed in terms of x_1 as follows:

$$x_2 = x_1(\partial g_1/\partial x_2)/(\partial g_1/\partial x_1) \qquad (2.28)$$

Applying Eqs. (2.2), (2.19), and (2.28), x_1 can be obtained as follows:

$$x_1 = - (\bar{X}_1+\bar{X}_2)(\partial g_1/\partial x_1)/[\sigma_{X_1}(\partial g_1/\partial x_1)+\sigma_{X_2}(\partial g_1/\partial x_2)] \qquad (2.29)$$

Using Eqs. (2.28) and (2.29), x_2 can be obtained as follows:

$$x_2 = - (\bar{X}_1+\bar{X}_2)(\partial g_1/\partial x_2)/[\sigma_{X_1}(\partial g_1/\partial x_1)+\sigma_{X_2}(\partial g_1/\partial x_2)] \qquad (2.30)$$

From Eqs. (2.22), (2.29), and (2.30), the shortest distance between the surface $g_1 = 0$ and the origin, d_{min}, can be determined as follows:

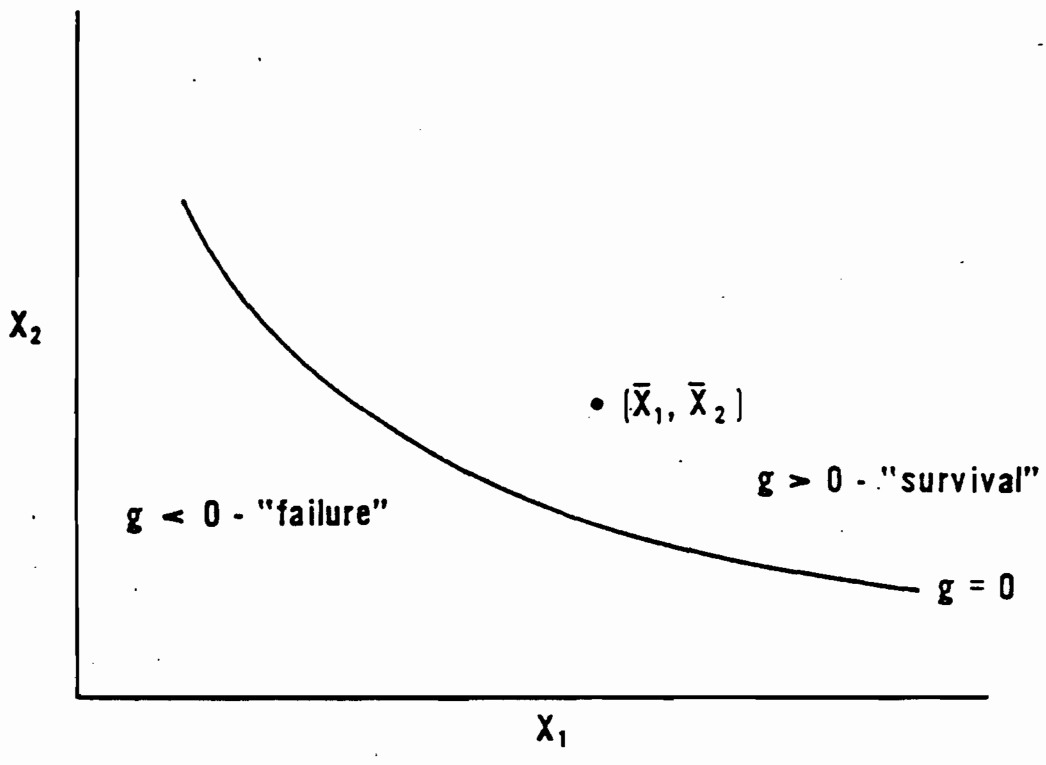

Figure 3 Formulation of Reliability Analysis in Original Variable Coordinates[16]

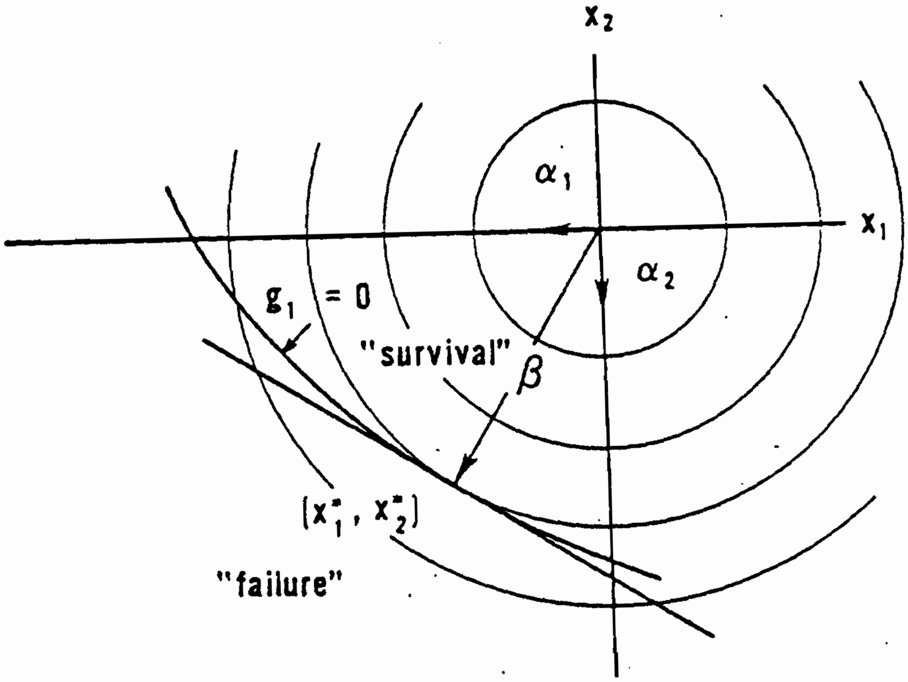

Figure 4 Formulation of Reliability Analysis in Reduced Variable Coordinates[16]

$$d_{min} = \sqrt{x_1^2+x_2^2}$$

$$= (\bar{X}_1+\bar{X}_2)\sqrt{\frac{(\partial g_1/\partial x_1)^2+(\partial g_1/\partial x_2)^2}{[\sigma_{X_1}(\partial g_1/\partial x_1)+\sigma_{X_2}(\partial g_1/\partial x_2)]^2}} \qquad (2.31)$$

From Eqs. (2.2), (2.19), and (2.20), it can be seen that

$$g_1(x_1,x_2) = x_1\sigma_{X_1}+\bar{X}_1+x_2\sigma_{X_2}+\bar{X}_2 = 0 \qquad (2.32)$$

Therefore, Eq. (2.31) can be rewritten as follows:

$$d_{min} = (\bar{X}_1+\bar{X}_2)\sqrt{\frac{(\partial g_1/\partial x_1)^2+(\partial g_1/\partial x_2)^2}{[(\partial g_1/\partial x_1)^2+(\partial g_1/\partial x_2)^2]^2}}$$

$$= \frac{\bar{X}_1+\bar{X}_2}{\sqrt{(\partial g_1/\partial x_1)^2+(\partial g_1/\partial x_2)^2}}$$

$$= \frac{-(x_1\sigma_{X_1}+x_2\sigma_{X_2})}{\sqrt{(\partial g_1/\partial x_1)^2+(\partial g_1/\partial x_2)^2}}$$

$$= \frac{-[x_1(\partial g_1/\partial x_1)+x_2(\partial g_1/\partial x_2)]}{\sqrt{(\partial g_1/\partial x_1)^2+(\partial g_1/\partial x_2)^2}} \qquad (2.33)$$

If the above equation is expressed as a general form and substituting checking point $(x_1^*, x_2^*,\ldots, x_n^*)$ into Eq. (2.33), β can be expressed as follows:

$$\beta = -\frac{\Sigma(\partial g_1/\partial x_i)_{x^*}x_i^*}{\sqrt{\Sigma(\partial g_1/\partial x_i)_{x^*}^2}} = -\Sigma\alpha_i x_i^* \qquad (2.34)$$

where

α_i = direction cosine

$$= (\partial g_1/\partial x_i)_{x^*}/\sqrt{\Sigma(\partial g_1/\partial x_i)_{x^*}^2} \qquad (2.35)$$

At the checking point, the distance d_{min} is perpendicular to the failure surface. Therefore

$$x_i^* = -\alpha_i\beta \qquad (2.36)$$

The checking point can be determined by solving Eqs. (2.34), (2.35), (2.36), (2.37), and searching for the direction cosines α_i which minimize β. Equation (2.37) is obtained by substituting checking point (x_1^*, x_2^*,..., x_n^*) into Eq. (2.20) as shown below.

$$g_1(x_1^*, x_2^*, \ldots, x_n^*) = 0 \tag{2.37}$$

In the original variable space, the checking point variables can be determined by using Eqs. (2.2), (2.19), and (2.36) as follows:

$$X_i^* = \bar{X}_i(1-\alpha_i\beta V_i) \tag{2.38}$$

$$g(X_1^*, X_2^*, \ldots, X_n^*) = 0 \tag{2.39}$$

If necessary, load and resistance factors for the design corresponding to a prescribed reliability index β may be determined through the use of the following equation:

$$F_{\gamma i} = X_i^*/X_{ni} \tag{2.40}$$

where

$F_{\gamma i}$ = load or resistance factor

X_{ni} = nominal value of the load or resistance

In the aforementioned derivations, the random variables X_i are assumed to be normally distributed. In fact, some structural problems involve random variables which are non-normal. In order to use the equations derived above, it is necessary to transform the non-normal variables into equivalent normal variables. For the purpose of determining the mean and standard deviation of the equivalent normal variables such that at the value X_i^*, the cumulative probability and probability density of the actual and approximating normal variable are equal, the following equations were recommended in References 16 and 78:

$$\sigma_{X_i}{}^N = \frac{\phi\{\Phi^{-1}[F_{X_i}(X_i^*)]\}}{f_{X_i}(X_i^*)} \quad (2.41)$$

$$\bar{X}_i{}^N = X_i^* - \Phi^{-1}[F_{X_i}(X_i^*)]\sigma_{X_i}{}^N \quad (2.42)$$

where

$\sigma_{X_i}{}^N$ = standard deviation of the equivalent normal variable

$\bar{X}_i{}^N$ = mean value of the equivalent normal variable

ϕ = density function for the standard normal variable

Φ^{-1} = inverse function of standard normal probability distribution

F_{X_i} = non-normal distribution function

f_{X_i} = non-normal density function

Having determined $\bar{X}_i{}^N$ and $\sigma_{X_i}{}^N$ of the equivalent normal distributions, the solution proceeds exactly as described above.

As an example of using advanced reliability analysis model, the two-variable problem considered in the previous section is shown as follows:

$$g = \ln(R/Q) = \ln(R) - \ln(Q) = 0 \quad (2.43)$$

in which both ln(R) and ln(Q) have normal distributions. Making the transformations,

$$r = \{\ln(R) - [\ln(R)]_m\}/\sigma_{\ln(R)} \quad (2.44)$$

$$q = \{\ln(Q) - [\ln(Q)]_m\}/\sigma_{\ln(Q)} \quad (2.45)$$

The failure criterion becomes

$$r\sigma_{\ln(R)} + [\ln(R)]_m - q\sigma_{\ln(Q)} - [\ln(Q)]_m = 0 \quad (2.46)$$

The failure criteria in the original ln(R), ln(Q) and reduced (r,q) coordinate systems are shown in Figures 5 and 6. The checking point variables can be determined using Eqs. (2.35), (2.36), and (2.46):

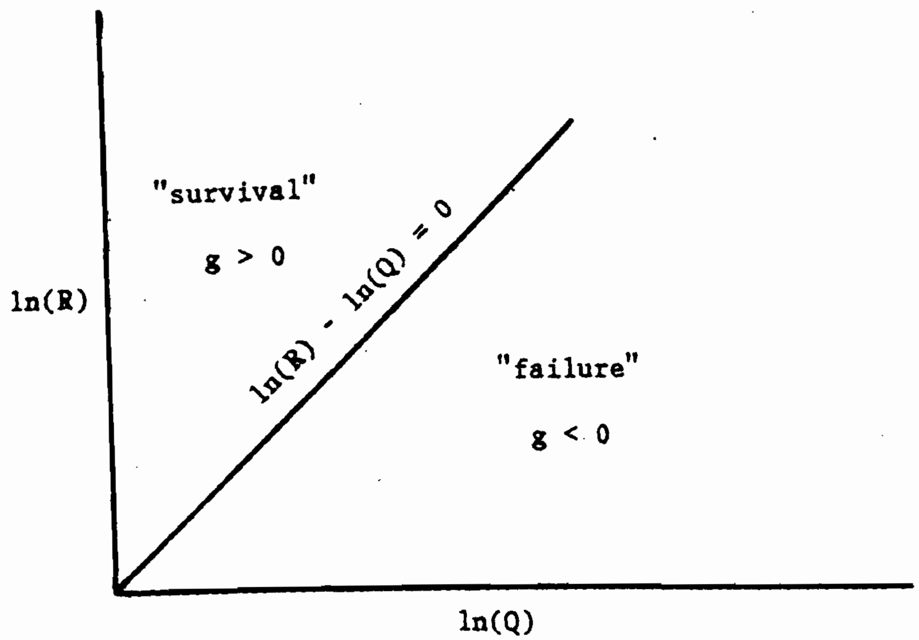

Figure 5 Reliability Calculation for Linear Two-Variable Problem in Original Variable Coordinates

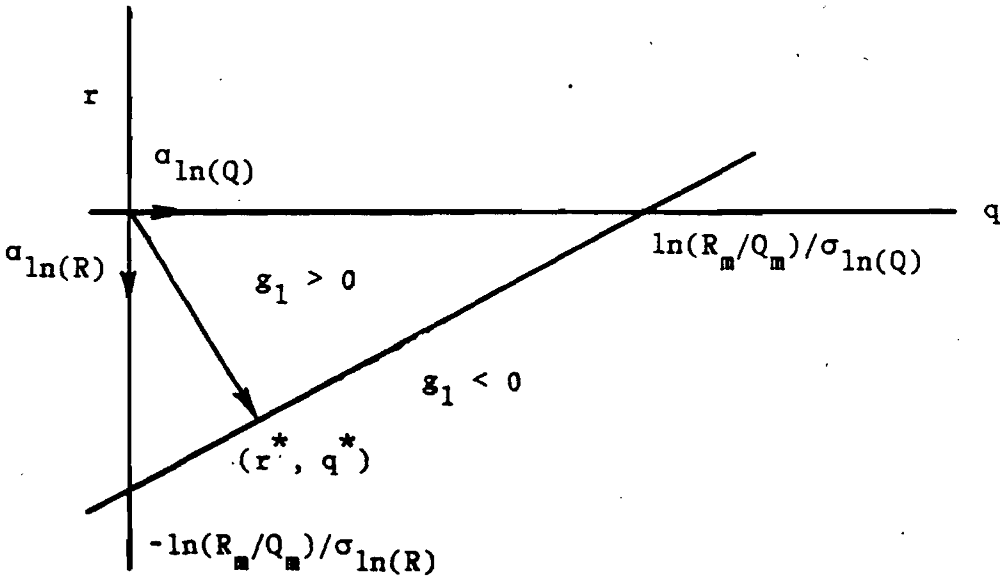

Figure 6 Reliability Calculation for Linear Two-Variable Problem in Reduced Variable Coordinates

$$r^{*} = -\alpha_{\ln(R)}\beta = -\beta\left[\frac{\sigma_{\ln(R)}}{\sqrt{\sigma_{\ln(R)}^{2}+\sigma_{\ln(Q)}^{2}}}\right] \tag{2.47}$$

$$q^{*} = -\alpha_{\ln(Q)}\beta = \beta\left[\frac{\sigma_{\ln(Q)}}{\sqrt{\sigma_{\ln(R)}^{2}+\sigma_{\ln(Q)}^{2}}}\right] \tag{2.48}$$

Substituting Eqs. (2.47) and (2.48) into Eq. (2.46),

$$-\beta\left[\frac{\sigma_{\ln(R)}^{2}}{\sqrt{\sigma_{\ln(R)}^{2}+\sigma_{\ln(Q)}^{2}}}\right]+[\ln(R)]_{m}$$
$$-\beta\left[\frac{\sigma_{\ln(Q)}^{2}}{\sqrt{\sigma_{\ln(R)}^{2}+\sigma_{\ln(Q)}^{2}}}\right]-[\ln(Q)]_{m} = 0 \tag{2.49}$$

Rearrange Eq. (2.49),

$$\beta = \frac{[\ln(R)]_{m} - [\ln(Q)]_{m}}{\sqrt{\sigma_{\ln(R)}^{2}+\sigma_{\ln(Q)}^{2}}} \tag{2.50}$$

Further simplifications for Eq. (2.50) can be made as follows:

$$[\ln(R)]_{m}-[\ln(Q)]_{m} \approx \ln(R_{m})-\ln(Q_{m})$$
$$\approx \ln(R_{m}/Q_{m}) \tag{2.51}$$

and

$$\sigma_{\ln(R)}^{2} = \{\partial[\ln(R)]/\partial R\}_{m}^{2}\sigma_{R}^{2} = \sigma_{R}^{2}/R_{m}^{2} = V_{R}^{2} \tag{2.52}$$
$$\sigma_{\ln(Q)}^{2} = \{\partial[\ln(Q)]/\partial Q\}_{m}^{2}\sigma_{Q}^{2} = \sigma_{Q}^{2}/Q_{m}^{2} = V_{Q}^{2} \tag{2.53}$$

The simplified result of Eq. (2.50) is identical with Eq. (2.18).

3. Selection of Model. As mentioned above, the advanced reliability analysis model is able to incorporate probability distributions which describe the true distributions more realistically, and is relatively straightforward in handling counteracting loads. Therefore, the advanced reliability analysis model was used for the development of the load factors and load combinations recommended in the 1982 ANSI Code[40]. These load factors and load combinations are appropriate for all types of building

materials, and therefore, they are adopted for the use of LRFD criteria for cold-formed steel members. However, necessary modifications are made to account for the special circumstances inherent in cold-formed steel structures.

Regarding to the determination of resistance factors, there are two important conclusions made in References 16 through 18:

1) The load factors and load combinations recommended in the 1982 ANSI Code do not prevent material specification writing groups from selecting their own ϕ factors together with their own desired values of β.

2) The mean-value first-order second-moment reliability analysis model gave results similar to those obtained from the advanced reliability analysis model.

Based on these conclusions, and a consideration of simplicity, it was decided that mean-value first-order second-moment reliability analysis model be used for the determination of resistance factors used in the LRFD criteria for cold-formed steel members. It should be noted that this is also the basis for the 1986 AISC LRFD Specification[19].

E. STATISTICAL DATA ON MATERIAL AND SECTIONAL PROPERTIES

As seen from Eq. (2.18), mean resistance R_m, coefficient of variation of resistance V_R, mean load effect Q_m, and coefficient of variation of load effect V_Q are needed in the structural reliability analysis. The determination of Q_m and V_Q is discussed in Section IV while the statistical data needed to determine R_m and V_R are discussed in this section.

The resistance of a structural member is assumed to be the following form:

$$R = R_n MFP \tag{2.54}$$

in which R_n is the nominal resistance of the structural elements, M, F, and P are dimensionless random variables reflecting the uncertainties in the material properties (i.e., F_y, F_u, etc.), the geometry of the cross-section (i.e., S_x, A, etc.), and the design assumptions.

The random variable M is called the "material factor", which is determined by the ratio of a tested mechanical property to a specified value. It is considered as a random variable because of the variation of mechanical properties of the materials. The fabrication factor F is a random variable which accounts for the uncertainties caused by initial imperfections, tolerances, and variations of geometric properties. The professional factor P is a random variable that reflects the uncertainties in the determination of the resistance. These uncertainties are induced by the use of approximations in the simplification, and idealization, of complicated design formulas.

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

$$R_m = R_n M_m F_m P_m \tag{2.55}$$

$$V_R = \sqrt{V_M^2+V_F^2+V_P^2+V_M^2V_F^2+V_F^2V_P^2+V_P^2V_M^2+V_M^2V_F^2V_P^2} \tag{2.56}$$

in which M_m, F_m, and P_m are mean values of M, F, and P, respectively; V_M, V_F, and V_P are coefficients of variation of M, F, and P, respectively.

Because of the small quantities of V_M, V_F, and V_P, Eq. (2.56) can be simplified as

$$V_R = \sqrt{V_M^2 + V_F^2 + V_P^2} \tag{2.57}$$

From Eqs. (2.55) and (2.57) it can be seen that the statistical data needed to determine R_m and V_R are M_m, V_M, F_m, V_F, P_m and V_P. P_m and V_P can be determined by comparing the tested failure loads and the predicted ultimate loads calculated from the selected design provisions. The P_m and V_P values for various design provisions for cold-formed steel members are discussed in section V.

For M_m and V_M, statistical data on yield point of virgin steels used for cold-formed steel members were studied by Rang[22]. The following mean values and coefficients of variation were recommended:

For yield point of virgin materials

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

For average yield point of steels considering cold-work effect

$(F_{ya})_m = 1.10F_{ya}$, $\quad V_{F_{ya}} = 0.11$

For ultimate strength of virgin steels

$(F_u)_m = 1.10F_u$, $\quad V_{F_u} = 0.08$

For modulus of elasticity

$E_m = 1.00E$, $\quad V_E = 0.06$

Consequently, the following mean values and coefficients of variation were selected as M_m and V_M:

For yield point of virgin materials

$M_m = 1.10$, $\quad V_M = 0.10$

For average yield point of steels considering cold-work effect

$M_m = 1.10$, $\quad V_M = 0.11$

For ultimate strength of virgin steels

$M_m = 1.10$, $\quad V_M = 0.08$

For modulus of elasticity

$M_m = 1.00,$ $V_M = 0.06$

For cold-formed steel structural members, the effect of cross-sectional dimensions (thickness of material, flange width, overall depth, dimensions of stiffeners, and inside bend radius, etc.) on the section modulus was also studied by Rang[23]. Based on the findings reported in Reference 23, the following mean value and coefficient of variation were selected as F_m and V_F for the design of cold-formed steel:

$F_m = 1.00,$ $V_F = 0.05$

F. DETERMINATION OF TARGET RELIABILITY INDICES

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 Reference 15, 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 References 22 through 27 and 34, 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 analyzed in References 16 through 18 by using the advanced reliability analysis model. It was found that 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 References 16 through 18 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 ASD design for all materials of construction. These target reliabilities, β_o, for use in LRFD are:

For gravity loading:	$\beta_o = 3.0$
For connections:	$\beta_o = 4.5$
For wind loading:	$\beta_o = 2.5$
For earthquake loading:	$\beta_o = 1.75$
For counteracting loading:	$\beta_o = 2.0$

These target reliability indices are inherent in the load factors recommended in the ANSI A58.1-82 Load Code[40].

For the reliability index inherent in cold-formed steel structural members, the studies in Reference 16 indicate that cold-formed steel has typically a low dead-to-live load ratio (around 0.2) and β is around 2.5. Also, in Reference 35 it was shown that cold-formed simply supported braced steel beams with stiffened flanges designed according to the 1986 AISI allowable stress design specification, or to any previous version of this specification, can provide a reliability index $\beta = 2.8$ for the representative dead-to-live load ratio of 1/5. 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[19].

G. FORMULAS FOR THE DETERMINATION OF STRUCTURAL RELIABILITY

The reliability evaluation discussed above is mainly for structural elements and members. The relationship between reliability index β and the probability of failure P_F was shown in Eq. (2.14). Equation (2.14) was based on the assumption that the probability distribution of (R/Q) is lognormal. In the cases that the probability distribution of (R/Q) is not known, the following relationship between the reliability index and the probability of failure was derived by Rosenblueth in Reference 79:

$$P_F = 460 \times 10^{-1.869\beta} \tag{2.58}$$

A structure is built-up by several components or elements. Naturally, its capacity will be a function of the capacities of the individual components, and thus, the probability of failure of the system will depend on its components. Except for very simple systems, such as structures that are statically determinate, or that are composed of identical components in parallel, the evaluation of the probability of failure of the entire system is generally quite involved. A great deal of work has been done[80-88], however, major difficulties in evaluating structural system reliabilities still remain, and no general formula has been recommended[88].

The studies in References 86 and 88 indicate that the only meaningful solution to the structural reliability problem for real structures is the upper and lower bounds of structural system reliability. These bounds are usually found to be quite widely spread for larger structural systems.

The upper and lower bounds for system reliability were first formulated in References 80, 81, and 85 as follows:

If all failure modes are statistically independent

$$P_f = P(\text{structural failure}) = 1 - \prod_i (1-P_{Fi}) \tag{2.59}$$

If all failure modes are perfectly correlated

$$P_f = P(\text{structural failure}) = \max_i P_{Fi} \tag{2.60}$$

where

P_{Fi} = probability of failure of component or element

Eq. (2.59) and (2.60) represent the upper and lower bounds of the probability of structural failure, respectively.

These bounds are called "First-Order Bounds" and it has been demonstrated that the bounds were too broad, therefore, a so-called "Ditlevsen Second-Order Bound" has been proposed to give a narrower range between the upper and lower bounds. These bounds are represented by the following formulas in References 82 and 88:

$$P_f > P_{F1} + \text{MAX}\left\{ \sum_{i=2}^{k} \left[P_{Fi} - \sum_{j=1}^{i=1} p(E_i E_j)\right]; \; 0\right\} \tag{2.61}$$

and

$$P_f < \sum_{i=1}^{k} P_{Fi} - \sum_{i=2}^{k} \underset{j<i}{\mathrm{MAX}}[p(E_iE_j)] \tag{2.62}$$

where

k = number of failure modes

$p(E_iE_j)$ = joint probability of exceeding limit states i and j

This joint probability is determined according to the method given in Reference 82 as follows:

$$p(E_iE_j) < p(A)+p(B) \tag{2.63}$$

$$p(E_iE_j) > \mathrm{MAX}\ [p(A),p(B)] \tag{2.64}$$

where Eq. (2.63) is used with Eq. (2.61), and Eq. (2.64) is used with Eq. (2.62), and

$$p(A) = \Phi(-\beta_i)\Phi(-X) \tag{2.65}$$

$$p(B) = \Phi(-\beta_j)\Phi(-Y) \tag{2.66}$$

where

$$X = \frac{\beta_j-\rho\beta_i}{(1-\rho^2)^{1/2}} \tag{2.67}$$

$$Y = \frac{\beta_i-\rho\beta_j}{(1-\rho^2)^{1/2}} \tag{2.68}$$

Φ = cumulative function of normal distribution

ρ = coefficient of correlation between two limit state functions

The coefficient of correlation between two linear limit state functions

$$Y_1 = \sum_{i=1}^{n} a_iX_i \tag{2.69}$$

and

$$Y_2 = \sum_{i=1}^{n} b_i X_i \qquad (2.70)$$

is equal to

$$\rho_{Y_1,Y_2} = COV(Y_1,Y_2)/(\sigma_{Y_1}\sigma_{Y_2}) \qquad (2.71)$$

where

$$COV(Y_1,Y_2) = \sum_{i=1}^{n} a_i b_i VAR(X_i) + \sum_{i=j}^{n} \sum^{n} a_i b_j COV(X_i X_j) \qquad (2.72)$$

The term $VAR(X_i) \doteq \sigma_{X_i}{}^2$ is the variance of X_i and $COV(X_i X_j)$ is the covariance of X_i, X_j, defined as follows:

$$COV(X_i X_j) = \rho_{ij}\sigma_{X_i}\sigma_{X_j} \qquad (2.73)$$

where

ρ_{ij} = coefficient of correlation between X_i and X_j

Once the lower and upper bounds of the probability of system failure are determined, the corresponding upper and lower bounds of system reliability indices can be determined from Eq. (2.14) as follows:

$$\beta = -\Phi^{-1}(P_f) \qquad (2.74)$$

where

Φ^{-1} = inverse function of cumulative lognormal distribution

or from Eq. (2.58) as follows:

$$\beta = [\log_{10}(460/P_f)]/1.869 \qquad (2.75)$$

It should be noted that Eq. (2.75) will give a result similar to Eq. (2.74). For example, for $P_f = 0.006$, Eq. (2.74) gives $\beta = 2.5$ while Eq. (2.75) gives $\beta = 2.6$.

III. LOAD FACTORS AND LOAD COMBINATIONS

A. GENERAL

Load factors, and load combinations, used in the LRFD criteria for cold-formed steel structural members and connections are basically adopted from the 1982 ANSI Code, in which load factors, and load combinations, are recommended for all types of materials including cold-formed steel. However, certain modifications and additional load factors and load combinations are needed to account for the special circumstances inherent in cold-formed steel structures. Background information used in determining the load factors, and load combinations, are discussed in this section.

B. MEAN-VALUE FIRST-ORDER SECOND-MOMENT METHOD

In order to determine load and resistance factors using mean-value first-order second-moment method, Eq. (2.18) must be transformed into the following form:

$$R_m \geq \theta Q_m \tag{3.1}$$

in which, θ is called central safety factor and defined as follow:

$$\theta = \exp(\beta\sqrt{V_R^2+V_Q^2}) \tag{3.2}$$

In Eq. (3.2), the central safety factor includes the uncertainties of both resistance and load effects. In order to separate this factor into independent factors, which deal only with the uncertainties of resistance or the uncertainties of load effects, the variation of the square root term is replaced by a straight line. Lind proposed the following linear approximation which is good for the range of V_R/V_Q between 1/3 and 3.0[59]:

$$\sqrt{V_R^2+V_Q^2} = \bar{\alpha}(V_R+V_Q) \tag{3.3}$$

in which $\bar{\alpha}$ is equal to 0.75. By substituting this linear approximation into Eq. (3.2), the central safety factor can be rewritten as:

$$\bar{\theta} = \exp[\bar{\alpha}\beta(V_R+V_Q)] \tag{3.4}$$

The design criterion becomes

$$\frac{R_m}{\exp(\bar{\alpha}\beta V_R)} \geq Q_m[\exp(\bar{\alpha}\beta V_Q)] \tag{3.5}$$

By trial and error, the following representative approximation for the dead and live load effects was selected by Galambos and Ravindra[7]:

$$\frac{R_m}{\exp(\alpha\beta V_R)} \geq \exp(\alpha\beta V_A)[c_D D_m(1+\alpha\beta\sqrt{V_C^2+V_D^2})]+ \exp(\alpha\beta V_A)[c_L L_m(1+\alpha\beta\sqrt{V_B^2+V_L^2})] \tag{3.6}$$

in which D_m, L_m and V_D, V_L are the mean values of the dead and live load intensities and their corresponding coefficients of variation. c_D and c_L are the deterministic influence factors used to transform the dead and live load intensities to the dead and live load effects. V_B and V_C are the coefficients of variation of random variables B and C which reflect the uncertainties in idealizing the design live and dead load from the actual values. A is a random variable reflecting the uncertainties in structural analysis and V_A is its coefficient of variation. α is a constant equal to 0.55. This linearization factor is obtained from minimization of the variation of the central safety factor over the possible range of all parameters. Equation (3.6) is in the same format as Eq. (2.1), therefore, resistance factor and load factors can be determined.

Equation (3.6) is for dead and live load combination only, other loads and load combinations are discussed in Reference 68.

Because of the shortcomings mentioned in Section II, this method was not used for the development of the load factors and load combinations recommended in the 1982 ANSI Code.

C. ADVANCED RELIABILITY ANALYSIS METHOD

For the advanced reliability analysis method, the procedures for determining load and resistance factors is discussed in Section II in great detail. Based on this method and the target reliability indices listed in Section II.F, the following load factors and load combinations were developed in References 16 through 18 and are recommended for use with the 1982 ANSI Load Code for all materials, including cold-formed steel:

1) $1.4D_n$

2) $1.2D_n+1.6L_n+0.5(L_{rn}$ or S_n or $R_{rn})$

3) $1.2D_n+1.6(L_{rn}$ or S_n or $R_{rn})+(0.5L_n$ or $0.8W_n)$

4) $1.2D_n+1.3W_n+0.5L_n+0.5(L_{rn}$ or S_n or $R_{rn})$

5) $1.2D_n+1.5E_n+(0.5L_n$ or $0.2S_n)$

6) $0.9D_n-(1.3W_n$ 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_{rn} = nominal roof rain load

S_n = nominal snow load

W_n = nominal wind load

D. LOAD FACTORS AND LOAD COMBINATIONS USED IN THE LRFD CRITERIA FOR COLD-FORMED STEEL

Based on the 1982 ANSI Code, the following load factors and load combinations are recommended in the LRFD criteria for cold-formed steel:

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

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 is $(1.4D_n+L_n)$ instead of the ANSI value of $1.4D_n$. This 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 composite 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 combination provides safe construction practice for cold-formed steel decks and panels which otherwise may be damaged during construction. The load factor used for the weight of wet concrete is 1.6 because the wet concrete 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 33% strength increase for concentrated loads.

It should be noted that for the third case of load combinations, the load factor used for the nominal roof live load, L_{rn}, in the LRFD criteria for cold-formed steel 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.

b) 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. The reliability index of a wall panel under wind load alone is approximately 1.5 with this reduction factor. With this reduction factor designs comparable to current practice are obtained.

IV. PROCEDURES FOR CALIBRATION OF DESIGN PROVISIONS

A. GENERAL

For the purpose of facilitating the steps used in the calibration of various provisions of the AISI Specification, the calibration procedures were formulated based on the mean-value first-order second-moment reliability analysis model and are summarized in this section.

As indicated in Reference 17, the load combination of $1.2D_n+1.6L_n$ governs many design cases, hence, the calibration procedures derived from this important load combination can be used for most of the design provisions. However, when the design provision is primary for uplift loading, the calibration procedures for uplift loading are needed. Therefore, both $1.2D_n+1.6L_n$ and $1.17W_n-0.9D_n$ (counteracting loads with a reduction factor of 0.9 applied to the load factor for the nominal wind load) are used in developing calibration procedures. All the determinations of resistance factors as well as reliability indices for various design provisions discussed in Section V are based on the formulas derived herein.

B. CALIBRATION PROCEDURES FOR GENERAL CASES

The load effects, Q, for a combination of dead and live loads is assumed to be the following form:

$$Q = c_D CD+c_L BL \tag{4.1}$$

where D and L are random variables representing the dead and live load intensities, respectively, c_D and c_L are deterministic influence coefficients, B and C are random variables reflecting the uncertainties in the transformation of loads into the load effects. Based on the first-order probabilistic theory, the mean load effects, Q_m, is

$$Q_m = c_D C_m D_m + c_L B_m L_m \tag{4.2}$$

in which, B_m, C_m, D_m, and L_m are the mean values of the random variables B, C, D, and L.

Consequently, the coefficient of variation of the load effects, V_Q, can be determined as follows[23]:

$$V_Q = \frac{\sqrt{c_D{}^2 C_m{}^2 D_m{}^2 (V_C{}^2 + V_D{}^2) + c_L{}^2 B_m{}^2 L_m{}^2 (V_B{}^2 + V_L{}^2)}}{c_D C_m D_m + c_L B_m L_m} \tag{4.3}$$

where V_B and V_C are the coefficients of variation for random variables B and C, respectively, V_D and V_L are the coefficients of variation for dead and live loads.

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

$$Q_m = c(D_m + L_m) \tag{4.4}$$

$$V_Q = \frac{\sqrt{(D_m V_D)^2 + (L_m V_L)^2}}{D_m + L_m} \tag{4.5}$$

Load statistics have been analyzed in Reference 16, where it was shown that $D_m = 1.05D_n$, $V_D = 0.1$, $L_m = L_n$, $V_L = 0.25$. The mean live load intensity equals to the code live load intensity if the tributary area is small enough so that no live load reduction is required. Substitution of the load statistics into Eqs. (4.4) and (4.5) gives

$$Q_m = c(1.05D_n/L_n + 1)L_n \tag{4.6}$$

$$V_Q = \frac{\sqrt{(1.05D_n/L_n)^2 V_D{}^2 + V_L{}^2}}{(1.05D_n/L_n + 1)} \tag{4.7}$$

Thus, Q_m and V_Q depend on the dead-to-live load ratio. Cold-formed steel members typically have relatively small D_n/L_n ratios. For the purposes of determining the reliability of the LRFD criteria, it will be assumed that $D_n/L_n = 1/5$, and so $V_Q = 0.21$.

In this study, the ϕ factors are determined for the load combination of $1.2D_n+1.6L_n$ to approximately provide a target β_o of 2.5 for members and 3.5 for connections, respectively. For practical reasons, it is desirable to have only a few different resistance factors. Therefore the actual values of β will differ from the derived targets. This means that

$$\phi R_n = c(1.2D_n+1.6L_n) = (1.2D_n/L_n+1.6)cL_n \tag{4.8}$$

By assuming $D_n/L_n = 1/5$, Eqs. (4.8) and (4.6) can be rewritten as follows:

$$R_n = 1.84(cL_n/\phi) \tag{4.9}$$

or

$$CL_n = \phi R_n/1.84 \tag{4.10}$$

$$Q_m = (1.05D_n/L_n+1)cL_n = 1.21cL_n = \phi R_n/1.521 \tag{4.11}$$

Therefore,

$$\frac{R_m}{Q_m} = \frac{1.521}{\phi}\,\frac{R_m}{R_n} \tag{4.12}$$

The application of Eqs. (2.18), (2.55), (2.57) and (4.12) gives

$$\beta = \frac{\ln(1.521M_mF_mP_m/\phi)}{\sqrt{V_M^2+V_F^2+V_P^2+V_Q^2}} \tag{4.13}$$

or

$$\phi = 1.521(M_mF_mP_m)\exp(-\beta\sqrt{V_M^2+V_F^2+V_P^2+V_Q^2}) \tag{4.14}$$

C. CALIBRATION PROCEDURES FOR COUNTERACTING LOADS

In this part of study, the ϕ factors are determined for the load combination of $1.17W_n-0.9D_n$ to approximately provide a target β_o of 1.5 for counteracting loads with a reduction factor of 0.9 applied to the load factor for the nominal wind load. The reasons for using a low target β_o are discussed in Section III. Based on this type of load combination, the following equations can be established:

$$\phi R_n = c(1.17W_n-0.9D_n) = (1.17-0.9D_n/W_n)cW_n \tag{4.15}$$

$$Q_m = c(W_m-D_m) \tag{4.16}$$

$$V_Q = \frac{\sqrt{(W_mV_W)^2+(D_mV_D)^2}}{W_m - D_m} \tag{4.17}$$

where W_m is the mean wind load intensity, and V_W is the corresponding coefficient of variation.

Load statistics have been analyzed in Reference 16, where it was shown that

$$D_m = 1.05D_n,\ V_D = 0.1;\ W_m = 0.78W_n,\ V_W = 0.37$$

The substitution of the load statistics into Eqs. (4.16) and (4.17) gives

$$Q_m = c(0.78W_n-1.05D_n) = (0.78-1.05D_n/W_n)cW_n \tag{4.18}$$

$$V_Q = \frac{\sqrt{(0.78\text{x}0.37)^2+(1.05D_n/W_n\text{x}0.1)^2}}{0.78 - 1.05D_n/W_n} \tag{4.19}$$

By assuming $D_n/W_n = 0.1$, Eqs. (4.15), (4.18), and (4.19) can be rewritten as follows:

$$\phi R_n = 1.08cW_n \tag{4.20}$$

$$Q_m = 0.675cW_n = 0.675(\phi R_n/1.08) = 0.625\phi R_n \tag{4.21}$$

$$V_Q = 0.43 \tag{4.22}$$

The application of Eqs. (2.18), (2.55), (2.57), and (4.21) gives

$$\beta = \frac{\ln(1.6M_m F_m P_m/\Phi)}{\sqrt{V_M^2+V_F^2+V_P^2+V_Q^2}} \tag{4.23}$$

or

$$\Phi = 1.6(M_m F_m P_m)\exp(-\beta\sqrt{V_M^2+V_F^2+V_P^2+V_Q^2}) \tag{4.24}$$

V. DEVELOPMENT OF LRFD DESIGN CRITERIA FOR COLD-FORMED STEEL MEMBERS AND CONNECTIONS

A. GENERAL

As mentioned in Section II.C, the design format of the LRFD criteria is given by Eq. (2.1). The right side of Eq. (2.1) represents the required strength which is computed by structural analysis based upon assumed loads, load factors, and load combinations. The development of load factors and load combinations is discussed in Section III. The left side of Eq. (2.1) represents the design strength provided by the selected components. The objective of this section is to develop the design strengths for various LRFD design provisions for cold-formed steel.

In the determination of design strength of a structural component, two major values are involved, namely, ϕ factor and nominal resistance R_n. In this section, the nominal resistances used for various design provisions in the LRFD criteria for cold-formed steel are either based on the nominal strengths specified in the 1986 AISI Specification or derived from the allowable strengths specified in the 1986 AISI Specification. The resistance factors and corresponding reliability indices are determined by calibrating various design provisions.

B. TENSION MEMBERS

For the design of axially loaded tension members, the nominal tensile strength, T_n, of a cold-formed steel member specified in the 1986 AISI Specification is used in the LRFD criteria for cold-formed steel members.

1. Design Requirements. For axially loaded tension members, the nominal tensile strength, T_n, shall be determined as follows:

$$T_n = A_n F_y \tag{5.1}$$

where

A_n = net area of the cross section

F_y = design yield stress

2. Development of the LRFD Criteria. The resistance factor ϕ_t = 0.95 is recommended for tension member design. It is derived from the procedure described in Section III, and a selected β_o value of 2.5. In the determination of the resistance factor, the following formulas are used for R_m and R_n:

$$R_m = A_n (F_y)_m \tag{5.2}$$

$$R_n = A_n F_y \tag{5.3}$$

i.e.,

$$R_m/R_n = (F_y)_m/F_y \tag{5.4}$$

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 II.E. By using $V_M = 0.10$, $V_F = 0.05$ and $V_P = 0$, the coefficient of variation V_R is:

$$V_R = \sqrt{V_M^2 + V_F^2 + V_P^2} = 0.11 \tag{5.5}$$

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 $\beta_o = 2.5$.

C. FLEXURAL MEMBERS

In the design of cold-formed steel flexural members, considerations should be given to the following design limit states:

. Strength for bending only

. Strength for shear only

. Strength for combined bending and shear

. Web crippling strength

. Combined bending and web crippling strength

Due to the lack of appropriate test data, ϕ factors for shear strength, and strength for combined bending and shear, are determined using a procedure which does not require test data. The determination of ϕ factors for other design provisions are based on the procedures derived in Section IV.

1. 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. For C- or Z-section with the tension flange attached to deck or sheathing, and with the compression flange laterally unbraced, the bending capacity is less than that of a fully braced member, but greater than that of an unbraced member. The nominal bending strengths M_n specified in the 1986 AISI Specification are used in the LRFD criteria for cold-formed steel members.

a. Design Requirements.

i. Nominal Section Strength. For nominal section strength of flexural members, the nominal bending strength, M_n, shall be calculated either on the basis of initiation of yielding in the effective section (Procedure I) or on the basis of the inelastic reserve capacity (Procedure II) as applicable.

Procedure I - Based on Initiation of Yielding

Effective yield moment based on section strength, M_n, shall be determined as follows:

$$M_n = S_e F_y \quad (5.6)$$

where

F_y = design yield stress

S_e = elastic section modulus of the effective section calculated with the extreme compression or tension fiber at F_y

Procedure II - Based on Inelastic Reserve Capacity

The inelastic flexural reserve capacity may be used when the following conditions are met:

- The member is not subject to twisting or to lateral, torsional, or torsional-flexural buckling.
- The effect of cold-forming is not included in determining the yield point F_y.
- The ratio of the depth of the compressed portion of the web to its thickness does not exceed λ_1.
- The shear force does not exceed $0.35F_y$ times the web area, h x t.
- The angle between any web and the vertical does not exceed 30 degrees.

The nominal flexural strength, M_n, shall not exceed either $1.25S_eF_y$ determined according to Procedure I or that causing a maximum compression strain of $C_y e_y$ (no limit is placed on the maximum tensile strain).

where

e_y = yield strain = F_y/E

E = modulus of elasticity

C_y = compression strain factor determined as follows:

(a) Stiffened compression elements without intermediate stiffeners

$C_y = 3$ for $w/t \leq \lambda_1$

$C_y = 3-2[(w/t-\lambda_1)/(\lambda_2-\lambda_1)]$ for $\lambda_1 < w/t < \lambda_2$

$C_y = 1$ for $w/t \geq \lambda_2$

where

$$\lambda_1 = 1.11/\sqrt{F_y/E} \quad (5.7)$$

$$\lambda_2 = 1.28/\sqrt{F_y/E} \quad (5.8)$$

(b) Unstiffened compression elements

$C_y = 1$

(c) Multiple-stiffened compression elements and compression elements with edge stiffeners

$C_y = 1$

When applicable, effective design widths shall be used in calculating section properties. M_n shall be calculated considering equilibrium of stresses, assuming an ideally elastic-plastic stress-strain curve which is the same in tension as in compression, assuming small deformation and assuming that plane sections remain plane during bending.

The effective widths, b, of compression elements are determined as follows:

(1) For uniformly compressed stiffened elements, the effective widths, b, shall be determined from the following formulas:

$$b = w \quad \text{when } \lambda \leq 0.673 \quad (5.9)$$

$$b = \rho w \quad \text{when } \lambda > 0.673 \quad (5.10)$$

where

w = flat width as shown in Figure 7

$\rho = (1-0.22/\lambda)/\lambda$ (5.11)

$\lambda = (1.052/\sqrt{k})(w/t)(\sqrt{f/E})$ (5.12)

k = 4.0 for stiffened elements supported by a web on each longitudinal edge

(2) For uniformly compressed stiffened elements with circular holes, the effective widths, b, shall be determined as follows:

For $0.50 \geq d_h/w \geq 0$, and $w/t \leq 70$

center-to-center spacing of holes > 0.50w and $3d_h$,

$b = w - d_h$ when $\lambda \leq 0.673$ (5.13)

$b = w[1-0.22/\lambda-(0.8d_h)/w]/\lambda$ when $\lambda > 0.673$ (5.14)

where

d_h = diameter of holes

(3) For webs and stiffened elements with stress gradient, the effective widths, b_1 and b_2, shall be determined from the following formulas:

$b_1 = b_e/(3-\Psi)$ (5.15)

For $\Psi \leq -0.236$

$b_2 = b_e/2$ (5.16)

b_1+b_2 shall not exceed the compression portion of the web calculated on the basis of effective section

For $\Psi > -0.236$

$b_2 = b_e - b_1$ (5.17)

where

b_e = effective width b determined in accordance with Case (1) with f_1 substituted for f and with k determined as follows:

$$k = 4+2(1-\Psi)^3+2(1-\Psi) \quad (5.18)$$

$\Psi = f_2/f_1$

f_1, f_2 = stresses shown in Figure 8 calculated on the basis of effective section. f_1 is compression (+) and f_2 can be either tension (-) or compression. In case f_1 and f_2 are both compression, $f_1 \geq f_2$

(4) For uniformly compressed unstiffened elements, the effective widths, b, shall be determined in accordance with Case (1) with the exception that k shall be taken as 0.43 (see Figure 9).

(5) For unstiffened elements and edge stiffeners with stress gradient, the effective widths, b, shall be determined in accordance with Case (1) with $f = f_3$ as in Figure 10 in the element and $k = 0.43$.

(6) For uniformly compressed elements with an intermediate stiffener (see Figure 11):

Case I: $b_o/t \leq S$

$$I_a = 0 \text{ (no intermediate stiffener needed)} \quad (5.19)$$

$$b = w \quad (5.20)$$

$$A_s = A_s' \quad (5.21)$$

Case II: $S < b_o/t < 3S$

$$I_a/t^4 = [50(b_o/t)/S]-50 \quad (5.22)$$

b and A_s shall be calculated according to Case (1) where

$$k = 3(I_s/I_a)^{1/2}+1 \leq 4 \quad (5.23)$$

$$A_s = A_s'(I_s/I_a) \leq A_s' \quad (5.24)$$

Case III. $b_o/t \geq 3S$

$$I_a/t^4 = [128(b_o/t)/S]-285 \quad (5.25)$$

b and A_s are calculated according to Case (1) where

$k = 3(I_s/I_a)^{1/3}+1 \leq 4$ (5.26)

$A_s = A_s'(I_s/I_a) \leq A_s'$ (5.27)

(7) For uniformly compressed elements with an edge stiffener (see Figure 10):

Case I: $w/t \leq S/3$

$I_a = 0$ (no edge stiffener needed) (5.28)

$b = w$ (5.29)

$d_s = d_s'$ for simple lip stiffener (5.30)

$A_s = A_s'$ for other stiffener shapes (5.31)

Case II: $S/3<w/t<S$

$I_a/t^4 = 399\{[(w/t)/S]-0.33\}^3$ (5.32)

$n = 1/2$

$C_2 = I_s/I_a \leq 1$ (5.33)

$C_1 = 2-C_2$ (5.34)

b shall be calculated according to Case (1) where

$k = [4.82-5(D/w)](I_s/I_a)^n+0.43 \leq 5.25-5(D/w)$ (5.35)

for $0.8 \geq D/w > 0.25$

$k = 3.57(I_s/I_a)^n+0.43 \leq 4.0$ (5.36)

for $(D/w) \leq 0.25$

$d_s = d_s'(I_s/I_a) \leq d_s'$ (5.37)

for simple lip stiffener

$A_s = A_s'(I_s/I_a) \leq A_s'$ (5.38)

for other stiffener shapes

Case III: $w/t \geq S$

$I_a/t^4 = [115(w/t)/S]+5$ (5.39)

C_1, C_2, b, k, d_s, A_s are calculated per Case II with $n=1/3$.

In Cases (6) and (7) of this section,

S = $1.28\sqrt{E/f}$

k = buckling coefficient

b_o = dimension defined in Figure 11

d, w, D = dimensions defined in Figure 10

d_s = reduced effective width of the stiffener (see Figure 10)

d'_s = effective width of the stiffener (see Figure 10)

C_1, C_2 = coefficients defined in Figures 10 and 11

A_s = reduced area of the stiffener as specified in this section. A_s is to be used in computing the overall effective section properties. The centroid of the stiffener is to be considered located at the centroid of the full area of the stiffener, and the moment of inertia of the stiffener about its own centroidal axis shall be that of the full section of the stiffener.

I_a = adequate moment of inertia of stiffener, so that each component element will behave as a stiffened element.

I_s, A'_s = moment of inertia of the full stiffener about its own centroidal axis parallel to the element to be stiffened and the effective area of the stiffener, respectively. For edge stiffeners the round corner between the stiffener and the element to be stiffened shall not be considered as a part of the stiffener.

For the stiffener shown in Figure 10,

I_s = $(d^3 t \sin^2\theta)/12$

A'_s = $d'_s t$

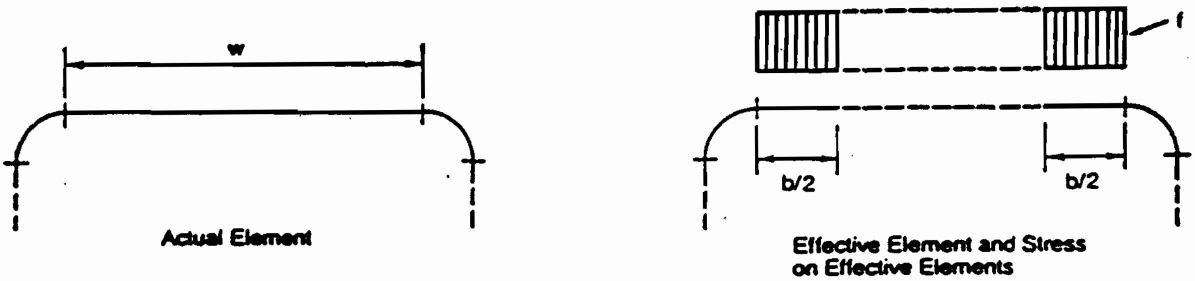

Figure 7 Stiffened Elements with Uniform Compression[33]

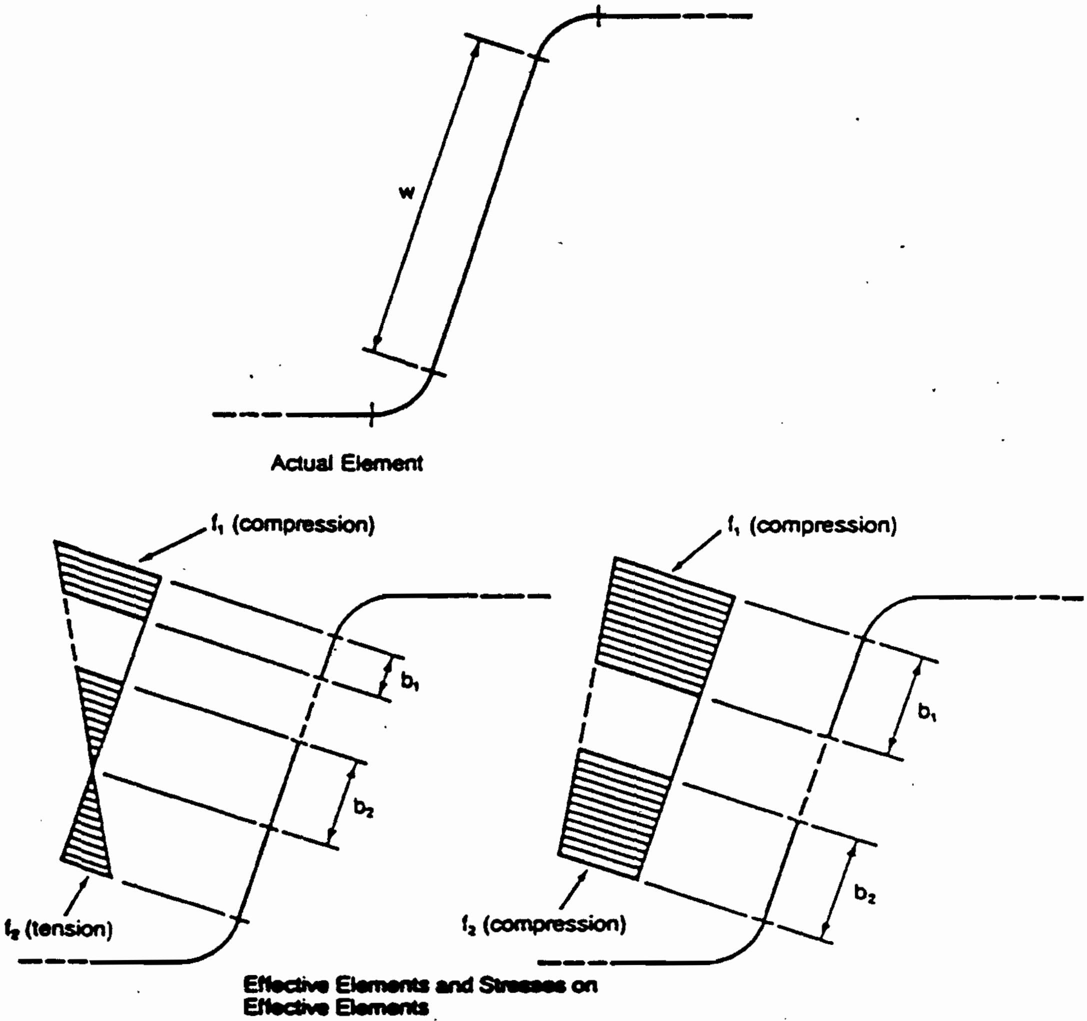

Figure 8 Stiffened Elements with Stress Gradient and Webs[33]

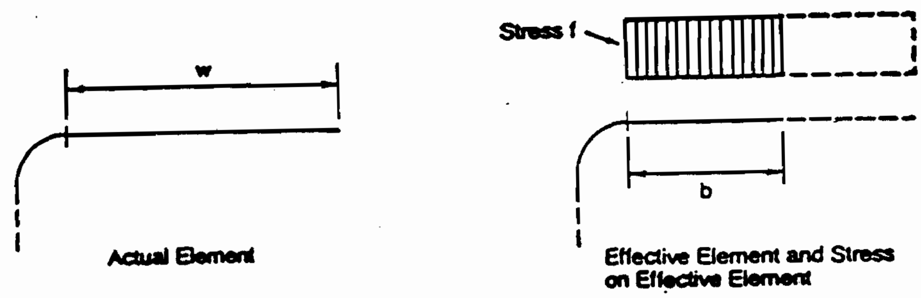

Figure 9 Unstiffened Elements with Uniform Compression[33]

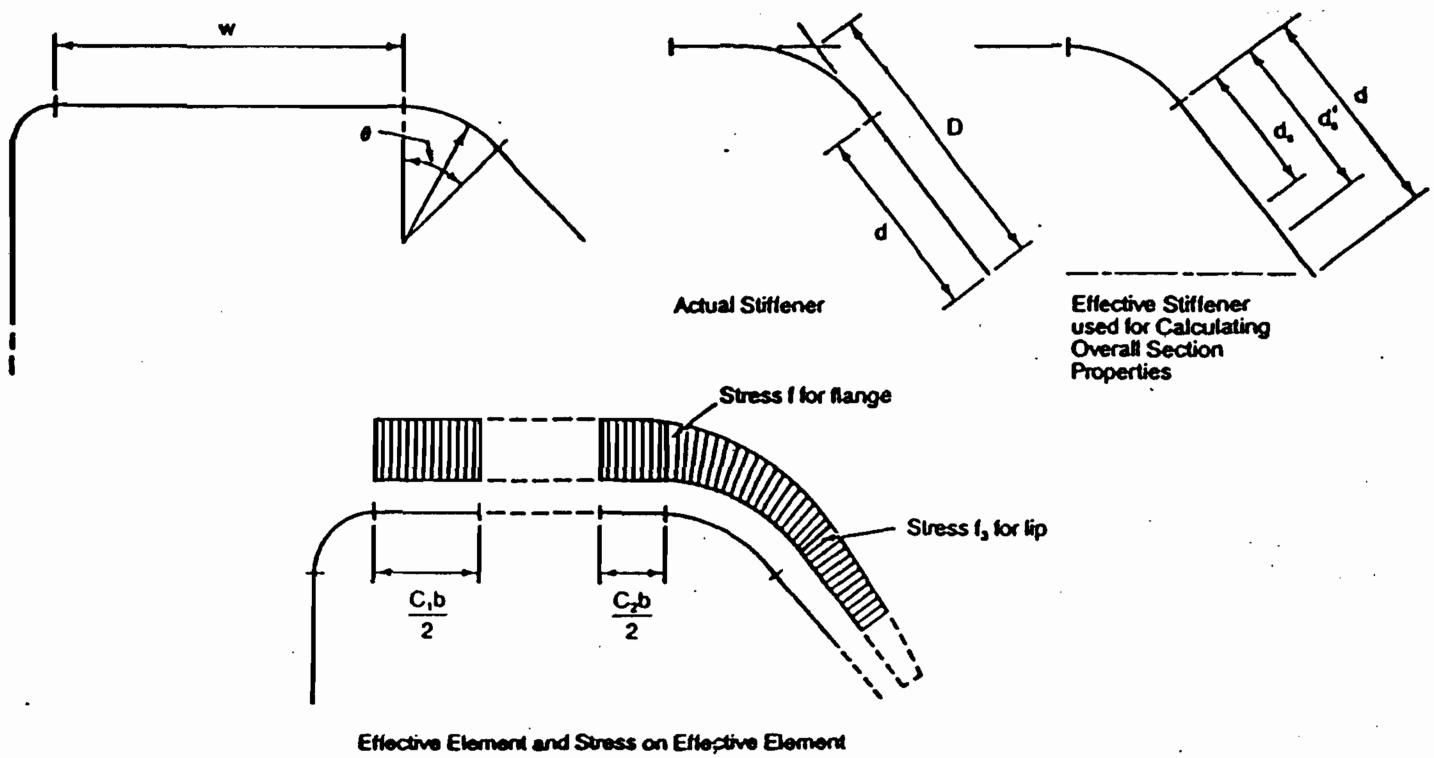

Figure 10 Elements with Edge Stiffener[33]

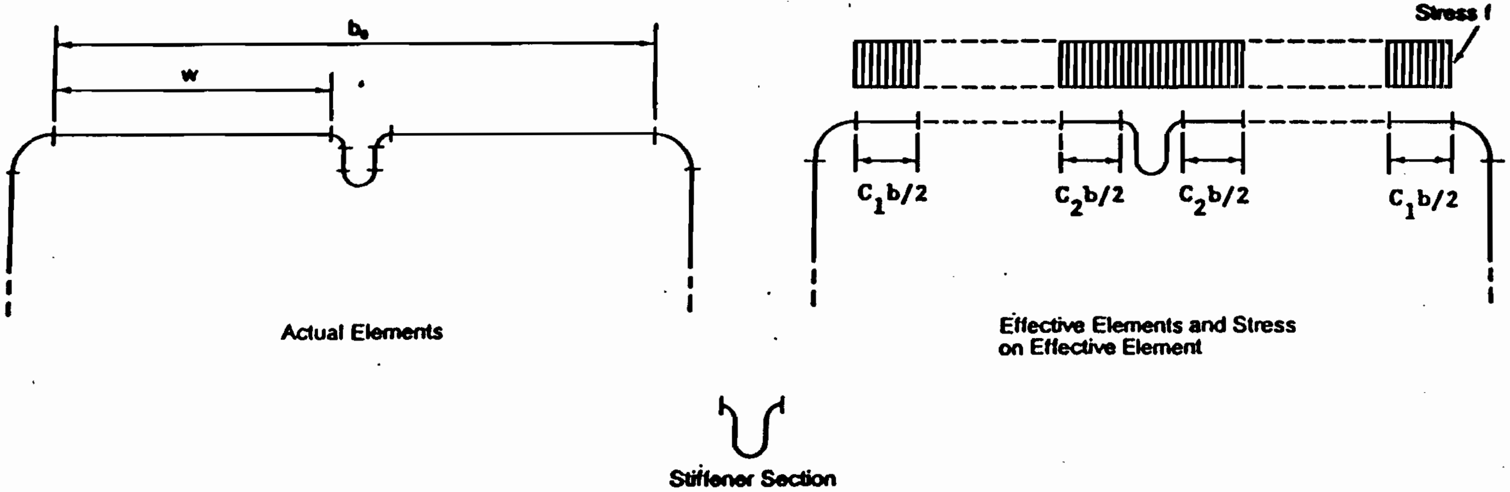

Figure 11 Elements with Intermediate Stiffener[33]

(8) For the determination of the effective width, the intermediate stiffener of an edge stiffened element or the stiffeners of a stiffened element with more than one stiffener shall be disregarded unless each intermediate stiffener has the minimum I_s as follows:

$$I_{min} = [3.66\sqrt{(w/t)^2-(0.136E)/F_y}]t^4 \qquad (5.40)$$

but not less than $18.4t^4$

where

w/t = width-thickness ratio of the larger stiffened sub-element

I_s = moment of inertia of the full stiffener about its own centroid axis parallel to the element to be stiffened

ii. Lateral Buckling Strength. Cold-formed steel flexural members, when loaded in the plane of the web, may twist and deflect laterally as well as vertically if adequate braces are not provided. For the laterally unbraced segments of doubly- or singly-symmetric sections subject to lateral buckling, M_n shall be determined as follows:

$$M_n = S_c(M_c/S_f) \qquad (5.41)$$

where

S_f = elastic section modulus of the full unreduced section for the extreme compression fiber

S_c = elastic section modulus of the effective section calculated at a stress M_c/S_f in the extreme compression fiber

M_c = critical moment calculated according to (a) or (b) below:

(a) For I- or Z-section bent about the centroidal axis (x-axis) perpendicular to the web:

For $M_e \geq 2.78M_y$

$$M_c = M_y \qquad (5.42)$$

For $2.78M_y > M_e > 0.56M_y$

$$M_c = (10/9)M_y(1-10M_y/36M_e) \tag{5.43}$$

For $M_e \leq 0.56M_y$

$$M_c = M_e \tag{5.44}$$

where

M_y = moment causing initial yield at the extreme compression fiber of the full section

$$= S_f F_y \tag{5.45}$$

M_e = elastic critical moment determined either as defined in (b) below or as follows:

$$= \pi^2 E C_b (d I_{yc}/L^2) \text{ for doubly-symmetric I-sections} \tag{5.46}$$

$$= \pi^2 E C_b (d I_{yc}/2L^2) \text{ for point-symmetric Z-sections} \tag{5.47}$$

L = unbraced length of the member

I_{yc} = moment of inertia of the compression portion of a section about the gravity axis of the entire section parallel to the web, using the full unreduced section

Other terms are defined in (b) below.

(b) For singly-symmetric sections (x-axis is assumed to be the axis of symmetry):

For $M_e > 0.5M_y$

$$M_c = M_y(1-M_y/4M_e) \tag{5.48}$$

For $M_e \leq 0.5M_y$

$$M_c = M_e \tag{5.49}$$

where

M_y is as defined in (a) above

M_e = elastic critical moment

M_e = $C_b r_o A\sqrt{\sigma_{ey}\sigma_t}$ for bending about the symmetry axis (x-axis is the axis of symmetry oriented such that the shear center has a negative x-coordinate.) Alternatively, M_e can be calculated using the formula for doubly-symmetric I-sections given in (a) above (5.50)

M_e = $C_s A\sigma_{ex}[j+C_s\sqrt{j^2+r_0{}^2(\sigma_t/\sigma_{ex})}]/C_{TF}$ for bending about centroidal axis perpendicular to the symmetry axis (5.51)

C_s = +1 for moment causing compression on the shear center side of the centroid

C_s = -1 for moment causing tension on the shear center side of the centroid

$$\sigma_{ex} = \pi^2E/(K_xL_x/r_x)^2 \quad (5.52)$$

$$\sigma_{ey} = \pi^2E/(K_yL_y/r_y)^2 \quad (5.53)$$

$$\sigma_t = 1/(Ar_o{}^2)[GJ+\pi^2EC_w/(K_tL_t)^2] \quad (5.54)$$

A = full cross-sectional area

C_b = bending coefficient which can conservatively be taken as unity, or calculated from

$$C_b = 1.75+1.05(M_1/M_2)+0.3(M_1/M_2)^2 \le 2.3$$

where

M_1 is the smaller and M_2 the larger bending moment at the ends of the unbraced length, taken about the strong axis of the member, and where M_1/M_2, the ratio of end moments, is positive when M_1 and M_2 have the same sign (reverse curvature bending) and negative when they are of opposite sign (single curvature bending). When the bending moment at any point within an unbraced length is larger than that at both ends of this length, and for members subject to combined axial load and bending moment, C_b shall be taken as unity.

E = modulus of elasticity

d = depth of section

C_{TF} = $0.6-0.4(M_1/M_2)$

where

M_1 is the smaller and M_2 the larger bending moment at the ends of the unbraced length, and where M_1/M_2, the ratio of end moments, is positive when M_1 and M_2 have the same sign (reverse curvature bending) and negative when they are of opposite sign (single curvature bending). When the bending moment at any point within an unbraced length is larger than that at both ends of this length, and for members subject to combined axial load and bending moment, C_b shall be taken as unity.

r_o = polar radius of gyration of the cross section about the shear center

$$= \sqrt{r_x^2+r_y^2+x_o^2} \qquad (5.55)$$

r_x, r_y = radii of gyration of the cross section about the centroidal principal axes

G = shear modulus

K_x, K_y, K_t = effective length factors for bending about the x- and y-axes, and for twisting

L_x, L_y, L_t = unbraced length of compression member for bending about the x- and y-axes, and for twisting

x_o = distance from the shear center to the centroid along the principal x-axis, taken as negative

J = St. Venant torsion constant of the cross section

C_w = torsional warping constant of the cross section

$$j = 1/(2I_y)(\int_A x^3 dA + \int_A xy^2 dA) - x_o \qquad (5.56)$$

iii. Beams Having One Flange Through-Fastened to Deck or Sheathing. For a C- or Z-section loaded in a plane parallel to the web and with the tension flange attached to deck or sheathing and with compression flange laterally unbraced, the nominal flexural strength, M_n, shall be calculated as follows:

$$M_n = RS_eF_y \qquad (5.57)$$

where

R = 0.40 for simple span C-sections

= 0.50 for simple span Z-sections

= 0.60 for continuous span C-sections

= 0.70 for continuous span Z-sections

The reduction factor, R, shall be limited to roof systems meeting the following conditions:

- Member depth shall be less than 11.5 inches
- The flanges shall be edge stiffened compression elements
- $60 \leq$ depth/thickness ≤ 170
- $2.8 \leq$ depth/flange width ≤ 4.5
- $16 \leq$ flat width/thickness of flange ≤ 43
- Lap length in each direction (distance from center of support to end of lap) shall not be less than:

  1.5d for Z-sections

3.0d for C-sections

- Member span length shall be no greater than 33 feet
- For continuous span systems, the longest member span shall not be more than 20% greater than the shortest span
- Both flanges shall be prevented from moving laterally at the supports
- Roof or wall panels shall be steel sheets, minimum of 0.019" coated thickness, having a minimum rib depth of 1 in., spaced 12 in. on centers and attached in a manner to effectively inhibit relative movement between the panel and purlin flange
- Insulation shall be glass fiber blanket 0 to 6 inches thick compressed between the member and panel in a manner consistent with the fastener being used
- Fastener type shall be minimum No. 12 self-drilling or self-tapping sheet metal screws or 3/16 - in. rivets, washers 1/2 in. diameter
- Fasteners shall not be standoff type screws
- Fasteners shall be spaced not greater than 12 in. on centers and placed near the center of the beam flange

If variables fall outside any of the above stated limits, the user must perform full-scale tests, or apply another rational analysis procedure. In any case, the user may perform tests, as an alternate to the procedure described above.

b. <u>Development of the LRFD Criteria</u>.

i. <u>Nominal Section Strength</u>. According to the design requirements discussed above, section strength shall be calculated either on the basis of initiation of yielding using effective section (Procedure I) or on the basis of the inelastic reserve capacity (Procedure II) as applicable. The calibration for nominal section strength deals only with Procedure I. In the calibration, the tested ultimate moments for beams, M_{test}, were obtained from References 89 through 95; the predicted values of M_{pred} were computed according to the design formulas mentioned above. The tested and predicted ultimate moments were listed in Tables 1 through 6 of Reference 34. On the basis of the statistical data for material properties and the dimensional properties listed in Section II.E, it was decided that the following values be used in this study: $M_m = 1.10$, $V_M = 0.10$, $F_m = 1.0$ and $V_F = 0.05$. Based on these values, the safety indices were computed and summarized in Table I. It can be seen that six different cases have been studied according to the types of the compression flanges. The results indicate that by using $\phi_b = 0.95$ for stiffened or partially stiffened compression flanges and $\phi_b = 0.9$ for unstiffened compression flanges, the values of β vary from 2.53 to 4.08 which are satisfactory to target β of 2.5.

ii. Lateral Buckling Strength. A total of 74 tests on lateral buckling of cold-formed steel beams were reported in Reference 96. Among these tests, the dimensions and cross-sectional properties of the 47 relatively long I-beams which failed in elastic buckling are as follows:

Thickness (t): 0.0598 in.

Depth (d): 4 in.

Width (2B): 2 in.

Area: 0.705 in.^{2}

Moment of inertia about x-axis (I_x): 1.515 in.^{4}

Moment of inertia about y-axis (I_y): 0.0806 $in.^4$

Torsional constant (J): 0.00260 $in.^4$

Radius of gyration about y-axis (r_y): 0.338 in.

Member lengths (L): vary from 61.5 in. to 138 in.

It shall be noted that the torsional constant provided in Reference 96 was based on a web considered to be a one piece element instead of two separate pieces. The latter is used for the AISI approach (J=0.00082 $in.^4$). Since the connection for the web was not clearly shown in Reference 96, both values were used in this calibration.

In addition to the AISI design formula, the theoretical approach and the Structural Stability Research Council (SSRC) approach[97] were used in this calibration.

The theoretical critical moment, M_{cr}, can be determined by the following formula:

$$M_{cr} = \frac{\pi^2 E}{L^2} \sqrt{I_y C_w} \sqrt{1 + \frac{GJL^2}{\pi^2 E C_w}} \tag{5.58}$$

For the SSRC approach, the buckling load, P_p, for a beam subjected to a concentrated load at the mid-span can be predicted by using the following equation:

$$P_p = \frac{1}{L^3}[2\pi^2 E C_b I_y d][\sqrt{1 + C_2^2 + \frac{4GJL^2}{\pi^2 E I_y d^2}} - C_2] \tag{5.59}$$

in which the values of C_b and C_2 are taken as 1.35 and 0.55, respectively.

The tested failure loads, P_t, and the predicted loads, P_p, were listed in Table 7 of Reference 34. The mean values and the coefficients of variation for the tested-to-predicted load ratios, P_t/P_p, for five different cases were listed in Table 8 of Reference 34.

Since all test specimens used in this calibration failed in the elastic range, only the modulus of elasticity was considered in the uncertainties of material properties. Therefore, $M_m = 1.00$ and $V_M = 0.06$. The mean value of the fabrication factor F_m was assumed to be unity with a coefficient of variation $V_F = 0.05$. Based on these values, the safety indices were computed and summarized in Table II. Five different cases have been studied with $\phi_b = 0.90$, and the values of β vary from 2.35 to 3.80. It can be seen that the β values obtained by using J = 0.00082 in.^{4} (AISI consideration) for all three approachs are satisfactory to the target β of 2.5.

iii. Beams Having One Flange Through-Fastened to Deck or Sheathing. In the calibration for beams having one flange through-fastened to deck or sheathing, the ϕ_b factor is determined for the load combination of $1.17W_n-0.9D_n$ to approximately provide a target β_o of 1.5 for counteracting loads with a reduction factor of 0.9 applied to the load factor for the nominal wind load. The reasons for using a low target β_o are discussed in Section III. In the calibration, the tested ultimate moments for beams, M_{test}, were obtained from References 98 through 102; the predicted values of M_{pred} were computed according to the design formulas mentioned above. The tested and predicted ultimate moments are listed in Tables III through VI. On the basis of the statistical data for material properties and the dimensional properties listed in Section II.E, it was decided that the following values be used in this study: $M_m = 1.10$, $V_M = 0.10$, $F_m = 1.0$ and $V_F = 0.05$. Based on these values, the computed values of β for the selected value of $\phi_b = 0.90$ for different cases are listed in Table

VII. It can be seen that the β values vary from 1.50 to 1.60 which are satisfactory for the target value of 1.5.

2. 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.:

$$V_n = A_w \tau_y = A_w F_y / \sqrt{3} = 0.577 F_y ht \tag{5.60}$$

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.:

$$V_n = A_w \tau_{cr} = \frac{k_v \pi^2 E A_w}{12(1-\mu^2)(h/t)^2} \tag{5.61}$$

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:

$$V_n = 0.905 E k_v t^3/h \tag{5.62}$$

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

$$V_n = 0.64 t^2 \sqrt{k_v F_y E} \tag{5.63}$$

These formulas are used in the LRFD criteria for cold-formed steel members.

Table I

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 or Partially Stiffened Compression Flanges (ϕ_b = 0.95)								
FF. FW.	8	1.10	0.10	1.0	0.05	1.10543	0.03928	2.76
PF. FW.	30	1.10	0.10	1.0	0.05	1.11400	0.08889	2.65
PF. PW.	5	1.10	0.10	1.0	0.05	1.08162	0.09157	2.53
Unstiffened Compression Flanges (ϕ_b = 0.90)								
FF. FW.	3	1.10	0.10	1.0	0.05	1.43330	0.04337	4.05
PF. FW.	40	1.10	0.10	1.0	0.05	1.12384	0.13923	2.67
PF. PW.	10	1.10	0.10	1.0	0.05	1.03162	0.05538	2.66

Note: FF. = Fully effective flanges
PF. = Partially effective flanges
FW. = Fully effective webs
PW. = Partially effective webs

Table II

Computed Safety Index β for Lateral Buckling Strength of Bending

(ϕ_b = 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	2.5213	0.30955	3.79
2	47	1.0	0.06	1.0	0.05	1.2359	0.19494	2.48
3	47	1.0	0.06	1.0	0.05	1.1800	0.19000	2.35
4	47	1.0	0.06	1.0	0.05	1.7951	0.21994	3.53
5	47	1.0	0.06	1.0	0.05	1.8782	0.20534	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 III

Comparison of Tested and Predicted Ultimate Moments of Cold-Formed Steel Beams Having One Flange Through-Fastened to Deck or Sheathing

(Simple Span C-Sections, R = 0.40)

Specimen	L (ft.)	F_y (ksi)	M_{pred} (ft.-kips)	M_{test} (ft.-kips)	$\frac{M_{test}}{M_{pred}}$	Reference
1	20.0	55.0	3.500	6.361	1.8174	98
2	20.0	55.3	5.004	5.867	1.1725	98
3	20.0	55.3	5.132	5.158	1.0051	98
4	20.0	64.5	5.600	5.110	0.9125	99
5	20.0	64.5	9.356	10.198	1.0900	99

Number of specimens	N = 5
Mean	P_m = 1.1995
Coefficient of variation	V_P = 0.2991

Table IV

Comparison of Tested and Predicted Ultimate Moments of Cold-Formed Steel Beams

Having One Flange Through-Fastened to Deck or Sheathing

(Simple Span Z-Sections, R = 0.50)

Specimen	L (ft.)	F_y (ksi)	$M_{pred.}$ (ft.-kips)	M_{test} (ft.-kips)	$\frac{M_{test}}{M_{pred}}$	Reference
1	20.0	66.0	4.630	4.537	0.9799	98
2	20.0	61.5	4.210	4.151	0.9860	98
3	20.0	56.9	4.565	4.812	1.0541	98
4	20.0	64.6	5.865	5.912	1.0080	98
5	20.0	64.7	6.495	6.898	1.0620	98
6	20.0	63.8	7.910	8.290	1.0480	98
7	20.0	64.0	8.350	8.133	0.9740	98
8	20.0	56.1	9.805	10.727	1.0940	98
9	20.0	65.9	11.225	9.205	0.8200	98
10	20.0	57.4	6.210	5.949	0.9580	98
11	20.0	57.3	5.295	5.697	1.0759	98
12	20.0	52.9	10.150	13.337	1.3140	98
13	20.0	57.6	12.220	10.900	0.8920	98
14	20.0	64.5	7.045	6.341	0.9001	99
15	20.0	64.5	11.660	11.963	1.0260	99

Table IV (Continued)

Number of specimens	$N = 15$
Mean	$P_m = 1.0128$
Coefficient of variation	$V_P = 0.1112$

Table V

Comparison of Tested and Predicted Ultimate Moments of Cold-Formed Steel Beams

Having One Flange Through-Fastened to Deck or Sheathing

(Continuous Span C-Sections, R = 0.60)

Specimen	L (ft.)	F_y (ksi)	M_{pred} (ft.-kips)	M_{test} (ft.-kips)	$\frac{M_{test}}{M_{pred}}$	Reference
1	24.0	56.5	5.652	5.897	1.0433	101
2	24.0	54.6	3.822	4.516	1.1816	101
3	24.0	58.3	9.156	9.995	1.0916	101
4	20.0	65.3	5.604	4.997	0.8917	101
5	30.0	58.3	9.480	9.717	1.0250	101

Number of specimens	N = 5
Mean	P_m = 1.0466
Coefficient of variation	V_P = 0.1010

Table VI

Comparison of Tested and Predicted Ultimate Moments of Cold-Formed Steel Beams

Having One Flange Through-Fastened to Deck or Sheathing

(Continuous Span Z-Sections, R = 0.70)

Specimen	L (ft.)	F_y (ksi)	M_{pred} (ft.-kips)	M_{test} (ft.-kips)	$\frac{M_{test}}{M_{pred}}$	Reference
1	30.0	63.1	9.310	9.935	1.0671	101
2	30.0	58.1	14.945	16.845	1.1271	101
3	30.0	61.6	9.135	9.788	1.0715	101
4	30.0	58.2	14.630	15.696	1.0729	101
5	20.0	61.3	8.890	8.966	1.0085	101
6	20.0	55.2	3.724	3.485	0.9358	101
7	20.0	57.4	3.920	3.970	1.0128	101
8	20.0	56.1	3.920	3.875	0.9885	101
9	20.0	55.8	3.829	3.807	0.9943	101
10	24.0	60.2	9.583	8.337	0.8700	101
11	24.0	57.8	9.177	8.889	0.9686	101
12	24.0	61.5	13.132	13.170	1.0029	101
13	24.0	58.5	6.069	6.182	1.0186	101
14	20.0	63.9	8.491	7.715	0.9086	101

Table VI (Continued)

Number of specimens	$N = 14$
Mean	$P_m = 1.0034$
Coefficient of variation	$V_P = 0.0689$

Table VII

Computed Safety Index β for Beams Having One Flange

Through-Fastened to Deck or Sheathing

(ϕ_b = 0.90)

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	1.1995	0.2991	1.60
2	15	1.10	0.10	1.0	0.05	1.0128	0.1112	1.50
3	5	1.10	0.10	1.0	0.05	1.0466	0.1010	1.58
4	14	1.10	0.10	1.0	0.05	1.0034	0.0689	1.51

Note: Case 1 = Simple span C-sections
Case 2 = Simple span Z-sections
Case 3 = Continuous span C-sections
Case 4 = Continuous span Z-sections

a. Design Requirements. The nominal shear strength, V_n, at any section shall be calculated as follows:

(a) For $h/t \leq \sqrt{Ek_v/F_y}$

V_n = Eq. (5.60)

(b) For $\sqrt{Ek_v/F_y} < h/t \leq 1.415\sqrt{Ek_v/F_y}$

V_n = Eq. (5.63)

(c) For $h/t > 1.415\sqrt{Ek_v/F_y}$

V_n = Eq. (5.62)

where

t = web thickness

h = depth of the flat portion of the web measured along the plane of the web

k_v = shear buckling coefficient determined as follows:

1. For unreinforced webs, $k_v = 5.34$
2. For beam webs with transverse stiffeners satisfying the requirements

when $a/h \leq 1.0$

$$k_v = 4.00+5.34/(a/h)^2 \tag{5.64}$$

when $a/h > 1.0$

$$k_v = 5.34+4.00/(a/h)^2 \tag{5.65}$$

where

a = the shear panel length for unreinforced web element

= distance between transverse stiffeners for web elements

For a web consisting of two or more sheets, each sheet shall be considered as a separate element carrying its share of the shear force.

b. Development of the LRFD Criteria. In the calibration of shear strength of beam webs, the results of the shear tests reported in Reference 103 were reviewed and considered. Because the connection arrangement used in the tests developed a considerable amount of tension field action, these test results were not used for the calibration.

Since the appropriate test data on shear are not available, the ϕ_v factors used in the LRFD criteria for cold-formed steel members 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,

$$(R_n)_{LRFD} = (R_n)_{ASD} \tag{5.66}$$

Since

$$(R_n)_{LRFD} \geq c(1.2D_n+1.6L_n)/\phi_v \tag{5.67}$$

$$(R_n)_{ASD} \geq c(F.S.)(D_n+L_n) \tag{5.68}$$

the resistance factors can be computed from the following formula:

$$\phi_v = \frac{1.2D_n+1.6L_n}{(F.S.)(D_n+L_n)} = \frac{1.2(D_n/L_n)+1.6}{(F.S.)(D_n/L_n+1)} \tag{5.69}$$

By using a dead-to-live load ratio of $D_n/L_n = 1/5$, the ϕ_v factors computed from the above equation are listed in Table VIII 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

Table VIII

Computed and Recommended ϕ_v Factors for Shear Strength of the Webs

Range of h/t Ratio	F.S. for Allowable Load Design	ϕ_v Factor computed by Eq. (5.69)	Recommended ϕ_v Factor
$h/t \le \sqrt{Ek_v/F_y}$	1.44	1.06	1.00
$\sqrt{Ek_v/F_y} \le h/t \le 1.415\sqrt{Ek_v/F_y}$	1.67	0.92	0.90
$h/t > 1.415\sqrt{Ek_v/F_y}$	1.71	0.90	0.90

yielding in shear, compared with those associated with yielding in tension and compression.

3. Strength for Combined Bending and Shear. For cantilever beams and continuous beams, high bending stresses often combined with high shear stresses at the supports. In the design of such members, interaction equations are used to prevent buckling of flat webs due to the combination of bending and shear stresses. The interaction equations used in the LRFD criteria for cold-formed steel are based on the interaction equations included in the 1986 AISI Specification.

a. Design Requirements. For beams with unreinforced webs, the moment, M, and the shear, V, shall satisfy the following interaction equation:

$$(M/M_n)^2+(V/V_n)^2 \leq 1.0 \tag{5.70}$$

For beams with transverse web stiffeners, the moment, M, and the shear, V, shall not exceed M_n and V_n, respectively. When $M/M_n > 0.5$ and $V/V_n > 0.7$, then M and V shall satisfy the following interaction equation:

$$0.6(M/M_n)+(V/V_n) \leq 1.3 \tag{5.71}$$

In the above equations:

M_n = nominal flexural strength when bending alone exists excluding lateral buckling strength

V_n = nominal shear strength when shear alone exists

b. Development of the LRFD Criteria. Due to the lack of sufficient test results of cold-formed steel members subjected to combined bending and shear, the calibration of this design provision is not possible. However, the results obtained from the calibration of combined bending and web crippling strength, and the calibration of combined axial load and bending, indicate that appropriate resistance factors obtained from the calibration of bending strength and shear strength can be used for the design of members subjected to the combination of bending and shear.

4. Web Crippling Strength. For flexural members, the unreinforced webs may cripple due to the high local intensity of load or reaction. In preventing this problem, four different loading conditions for both I-sections and shapes having single webs are considered in the 1986 AISI Specification:

. End one-flange loading

. Interior one-flange loading

. End two-flange loading

. Interior two-flange loading

Equations for determining allowable concentrated load or reaction for different cases are given in the 1986 AISI Specification.

The nominal concentrated loads or reactions, P_n, used in the LRFD criteria for cold-formed steel are determined by the allowable loads given in the 1986 AISI Specification times the appropriate factor of safety. In this regard, a factor of safety of 1.85 is used for single unreinforced webs, and a factor of safety of 2.0 is used for I-beams or similar sections.

a. Design Requirements. Table IX is used to determine the nominal web crippling strength, P_n, of webs of flexural members subject to concentrated loads or reactions, or the components thereof, acting perpendicular to the longitudinal axis of the member and in the plane of the

Table IX

Nominal Web Crippling Strength, P_n

		Shapes Having Single Webs		I-Sections or Similar Sections(1)
		Stiffened or Partially Stiffened Flanges	Unstiffened Flanges	Stiffened, Partially Stiffened and Unstiffened Flanges
Opposing loads Spaced>1.5h(2)	End Reaction(3)	Eq. (5.72)	Eq. (5.73)	Eq. (5.74)
	Interior Reaction(4)	Eq. (5.75)	Eq. (5.75)	Eq. (5.76)
Opposing Loads Spaced ≤1.5h(5)	End Reaction(3)	Eq. (5.77)	Eq. (5.77)	Eq. (5.78)
	Interior Reaction(4)	Eq. (5.79)	Eq. (5.79)	Eq. (5.80)

Footnotes and Equation References to the above table:

(1) I-sections made of two channels connected back to back or similar sections which provide a high degree of restraint against rotation of the web (such as I-sections made by welding two angles to a channel).

(2) At locations of one concentrated load or reaction acting either on the top or bottom flange, when the clear distance between the bearing edges of this and adjacent opposite concentrated loads or reactions is greater than 1.5h.

(3) For end reactions of beams or concentrated loads on the end of cantilevers when the distance from the edge of the bearing to the end of the beam is less than 1.5h.

(4) For reactions and concentrated loads when the distance from the edge of bearing to the end of the beam is equal to or greater than 1.5h.

(5) At locations of two opposite concentrated loads or of a concentrated load and an opposite reaction acting simultaneously on the top and bottom flanges, when the clear distance between their adjacent bearing edges is equal to or less than 1.5h.

web under consideration, and causing compressive stresses in the web. This table is used for unreinforced flat webs of flexural members having a flat width ratio, h/t, equal to or less than 200. Webs of flexural members for which h/t is greater than 200 shall be provided with adequate means of transmitting concentrated loads and/or reactions directly into the webs.

The formulas in Table IX apply to beams when $R/t \le 6$ and to deck when $R/t \le 7$, $N/t \le 210$ and $N/h \le 3.5$.

P_n represents the nominal strength for concentrated load or reaction for one solid web connecting top and bottom flanges. For two or more webs, P_n shall be computed for each individual web and the results added to obtain the nominal load or reaction for the multiple web.

For built-up I-sections, or similar sections, the distance between the web connector and beam flange shall be kept as small as practical.

Equations for Table IX:

$$P_n = t^2kC_3C_4C_\theta[331-0.61(h/t)][1+0.01(N/t)] \qquad (5.72)$$

$$P_n = t^2kC_3C_4C_\theta[217-0.28(h/t)][1+0.01(N/t)] \qquad (5.73)$$

when $N/t > 60$, the factor $[1+0.01(N/t)]$ may be increased to $[0.71+0.015(N/t)]$

$$P_n = t^2F_yC_6(10+1.25\sqrt{N/t}) \qquad (5.74)$$

$$P_n = t^2kC_1C_2C_\theta[538-0.74(h/t)][1+0.007(N/t)] \qquad (5.75)$$

when $N/t > 60$, the factor $[1+0.007(N/t)]$ may be increased to $[0.75+0.011(N/t)]$

$$P_n = t^2F_yC_5(0.88+0.12m)(15+3.25\sqrt{N/t}) \qquad (5.76)$$

$$P_n = t^2kC_3C_4C_\theta[244-0.57(h/t)][1+0.01(N/t)] \qquad (5.77)$$

$$P_n = t^2F_yC_8(0.64+0.31m)(10+1.25\sqrt{N/t}) \qquad (5.78)$$

$$P_n = t^2 k C_1 C_2 C_\theta [771-2.26(h/t)][1+0.0013(N/t)] \quad (5.79)$$

$$P_n = t^2 F_y C_7 (0.82+0.15m)(15+3.25\sqrt{N/t}) \quad (5.80)$$

In the above-referenced formulas,

P_n = nominal strength for concentrated load or reaction per web

$$C_1 = (1.22-0.22k) \quad (5.81)$$

$$C_2 = (1.06-0.06R/t) \le 1.0 \quad (5.82)$$

$$C_3 = (1.33-0.33k) \quad (5.83)$$

$$C_4 = (1.15-0.15R/t) \le 1.0 \text{ but not less than } 0.50 \quad (5.84)$$

$$C_5 = (1.49-0.53k) \ge 0.6 \quad (5.85)$$

$$C_6 = 1+(h/t)/750, \text{ when } h/t \le 150 \quad (5.86)$$

$$= 1.20, \text{ when } h/t > 150 \quad (5.87)$$

$$C_7 = 1/k, \text{ when } h/t \le 66.5 \quad (5.88)$$

$$= [1.10-(h/t)/665](1/k), \text{ when } h/t > 66.5 \quad (5.89)$$

$$C_8 = [0.98-(h/t)/865](1/k) \quad (5.90)$$

$$C_\theta = 0.7+0.3(\theta/90)^2 \quad (5.91)$$

F_y = design yield stress of the web, ksi

h = depth of the flat portion of the web measured along the plane of the web

$$k = F_y/33 \quad (5.92)$$

$$m = t/0.075 \quad (5.93)$$

t = web thickness, inches

N = actual length of bearing, inches. For the case of two equal and opposite concentrated loads distributed over unequal bearing lengths, the smaller value of N shall be taken

R = inside bend radius

θ = angle between the plane of the web and the plane of the bearing surface ≥ 45°, but not more than 90°

b. Development of the LRFD Criteria. In this investigation, the design formulas listed in Table IX were calibrated by using the results of 589 tests, which included 375 tests for beams having single unreinforced webs and 214 tests for I-beam sections.

Based on the test data obtained from References 104 through 106, and the predicted web crippling loads, P_{pred}, computed from the equations listed above, the professional factors were determined by using the ratios of P_{test}/P_{pred} as listed in Tables 9 through 20 of Reference 34. The meam values and coefficients of variation of the professional factors (P_m and V_P) were also included in Tables 9 through 20 of Reference 34.

By using the above mentioned values, and the values of M_m, V_M, F_m, and V_F listed in Table X, the values of the safety index for 15 different cases were determined by using $\phi_w = 0.75$ and 0.80 for single unreinforced webs and I-sections, respectively. All of the computed values of the safety index are listed in Table X. From this table, it can be seen that the safety indexes β vary from 2.36 to 3.80.

5. Combined Bending and Web Crippling Strength. For practical applications, there are some cases that combined bending and web crippling strength must be considered. For instance, a high bending moment may occur at the location of the applied concentrated load in simple beams; for continuous beams, the reactions at supports may be combined with high bending moment. In the design of such members, interaction formulas are used. The interaction formulas used in the LRFD criteria for cold-formed steel are derived from the interaction formulas included in the 1986 AISI Specification:

(1) For shapes having single unreinforced webs, the interaction formula is derived by using a safety factor of 1.85 for web crippling load and a safety factor of 1.67 for bending moment.

(2) For shapes having multiple unreinforced webs, the interaction formula is derived by using a safety factor of 2.0 for web crippling load and a safety factor of 1.67 for bending moment.

a. Design Requirements. Unreinforced flat webs of shapes subjected to a combination of bending and concentrated load or reaction shall be designed to meet the following requirements:

(a) For shapes having single unreinforced webs:

$$1.07(P/P_n)+(M/M_n) \leq 1.42 \qquad (5.94)$$

Exception: At the interior supports of continuous spans, the above formula is not applicable to deck or beams with two or more single webs, provided the compression edges of adjacent webs are laterally supported in the negative moment region by continuous or intermittently connected flange elements, rigid cladding, or lateral bracing, and the spacing between adjacent webs does not exceed 10 inches.

(b) For shapes having multiple unreinforced webs such as I-sections made of two channels connected back-to-back, or similar sections which provide a high degree of restraint against rotation of the web (such as I-sections made by welding two angles to a channel);

$$0.82(P/P_n)+(M/M_n) \leq 1.32 \qquad (5.95)$$

Exception: When $h/t \leq 2.33/\sqrt{(F_y/E)}$ and $\lambda \leq 0.673$, the nominal concentrated load or reaction strength may be determined by considering web crippling strength only.

In the above formulas,

P = applied concentrated load or reaction in the presence of bending moment.

P_n = nominal strength for concentrated load or reaction in the absence of bending moment

M = applied bending moment at, or immediately adjacent to, the point of application of the concentrated load or reaction P

M_n = nominal flexural strength if bending alone exists excluding lateral buckling strength

w = flat width of the beam flange which contacts the bearing plate

t = thickness of the web or flange

λ = slenderness factor

b. Development of the LRFD Criteria. A total of 551 tests were used in this calibration, which included 445 tests for beams having single unreinforced webs, and 106 tests for I-beam sections. The tested failure loads, P_{test}, were obtained from References 104 through 108. The predicted values of P_{pred} were computed according to the interaction formulas listed above. The tested and predicted failure loads and their ratios, P_{test}/P_{pred}, were listed in Tables 21 through 25 of Reference 34. The mean values and coefficients of variation of the professional factors (P_m and V_P) were also included in the tables mentioned above.

By using the above mentioned values and the values of M_m, V_M, F_m, and V_F listed in Table XI, the values of the safety index for six different cases were determined on the basis of ϕ_w = 0.75 and 0.80 for single

Table X

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 (ϕ_w = 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 (ϕ_w = 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 XI

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 ϕ_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 ϕ_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

unreinforced webs and I-sections, respectively. All the computed values of the safety index are listed in Table XI. From this table, it can be seen that the safety indexes β vary from 2.45 to 3.27 which are satisfactory to the target β of 2.5.

D. CONCENTRICALLY LOADED COMPRESSION MEMBERS

For the design of concentrically loaded compression members of cold-formed steel, consideration should be given to the following types of failure depending on the shape of the cross section, thickness of material, and the stiffness of the compression member[6]:

. Yielding

. Overall column buckling

(a) Flexural buckling: bending about a principal axis

(b) Torsional buckling: twisting about shear center

(c) Torsional-flexural buckling: bending and twisting simultaneously

. Local buckling of individual elements

For short columns, yielding and local buckling are the usual modes of failure. The overall instability caused by elastic flexural buckling or torsional-flexural buckling is normally the failure mode for long columns. Columns having moderate slenderness ratios usually fail by inelastic flexural buckling or torsional-flexural buckling. The formulas for determining nominal axial strength of various failure modes, P_n, used in the LRFD criteria for cold-formed steel members are the same as those included in the 1986 AISI Specification.

1. Design Requirements. For members in which the resultant of all loads acting on the member is an axial load passing through the centroid of the effective section calculated at the stress, F_n, as defined in this section, can be considered as concentrically loaded compression members.

(a) The nominal axial strength, P_n, shall be calculated as follows:

$$P_n = A_e F_n \tag{5.96}$$

where

A_e = effective area at the stress F_n. For sections with circular holes, A_e shall be determined in accordance with Eqs. (5.13) and (5.14) of Section V.C and subject to the limitations of that section. If the number of holes in the effective length region times the hole diameter divided by the effective length does not exceed 0.015, A_e can be determined ignoring the holes.

F_n is determined as follows:

For $F_e > F_y/2$ $\qquad F_n = F_y(1-F_y/4F_e)$ (5.97)

For $F_e \leq F_y/2$ $\qquad F_n = F_e$ (5.98)

F_e is the least of the elastic flexural, torsional and torsional-flexural buckling stress.

(b) For C- and Z-shapes, and single-angle sections with unstiffened flanges, P_n shall be taken as the smaller of P_n calculated above and P_n calculated as follows:

$$P_n = A\pi^2 E/[25.7(w/t)^2] \tag{5.99}$$

where

A = area of the full, unreduced cross section

w = flat width of the unstiffened element

t = thickness of the unstiffened element

(c) Angle sections shall be designed for the applied axial load, P, acting simultaneously with a moment equal to PL/1000 applied about the minor principal axis causing compression in the tips of the angle legs.

(d) The slenderness ratio, KL/r, of all compression members preferably should not exceed 200, except that during construction only, KL/r preferably should not exceed 300.

For doubly-symmetric sections, closed cross sections and any other sections which can be shown not to be subject to torsional or torsional-flexural buckling, the elastic flexural buckling stress, F_e, shall be determined as follows:

$$F_e = \pi^2 E/(KL/r)^2 \qquad (5.100)$$

where

E = modulus of elasticity

K = effective length factor

L = unbraced length of member

r = radius of gyration of the full, unreduced cross section

For sections subject to torsional or torsional-flexural buckling, F_e shall be taken as the smaller of F_e calculated above and F_e calculated as follows:

$$F_e = (1/2\beta)\left[(\sigma_{ex}+\sigma_t)-\sqrt{(\sigma_{ex}+\sigma_t)^2-4\beta\sigma_{ex}\sigma_t}\right] \qquad (5.101)$$

Alternatively, a conservative estimate of F_e can be obtained using the following equation:

$$F_e = \sigma_t\sigma_{ex}/(\sigma_t+\sigma_{ex}) \qquad (5.102)$$

where

σ_t and σ_{ex} are previously defined

$$\beta = 1-(x_o/r_o)^2 \qquad (5.103)$$

For singly-symmetric sections, the x-axis is assumed to be the axis of symmetry.

For shapes whose cross sections do not have any symmetry, either about an axis or about a point, F_e shall be determined by rational analysis.

2. Development of the LRFD Criteria. A total of 264 tests were used in this calibration. The tested failure loads, P_{test}, were obtained from References 94 and 109 through 117. The predicted values of P_{pred} were computed according to the design formulas mentioned above. The tested and predicted failure loads were listed in Tables 26 through 39 of Reference 34. The mean values and the coefficients of variation of the tested-to-predicted load ratios, P_{test}/P_{pred}, were also included in these tables.

On the basis of the statistical data summarized in Section II.E, the values of M_m, V_M, F_m and V_F are listed in Table XII. Based on all these values, the safety indices were computed and presented in the same table. This calibration included 14 different cases according to the types of columns, the types of compression flanges (stiffened or unstiffened), and the types of failure modes (flexural, torsional or torsional-flexural buckling). From these results, it can be seen that the use of ϕ_c = 0.85 will provide values of β ranging from 2.39 to 3.34, which are satisfactory when compared to the target β of 2.5.

Table XII

Computed Safety Index β for Concentrically Loaded Compression Member

(Φ_c = 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	1.14610	0.10452	3.13
2	24	1.10	0.10	1.0	0.05	1.05053	0.07971	2.89
3	15	1.10	0.10	1.0	0.05	1.05523	0.07488	2.93
4	3	1.10	0.10	1.0	0.05	1.10550	0.07601	3.11
5	28	1.10	0.10	1.0	0.05	1.04750	0.11072	2.76
6	25	1.10	0.10	1.0	0.05	1.22391	0.21814	2.72
7	9	1.00	0.06	1.0	0.05	0.96330	0.04424	2.39
8	41	1.10	0.10	1.0	0.05	1.19620	0.09608	3.34
9	18	1.10	0.10	1.0	0.05	1.02900	0.08131	2.81
10	12	1.10	0.11	1.0	0.05	1.06180	0.11062	2.77
11	8	1.00	0.06	1.0	0.05	1.15290	0.10544	2.92
12	30	1.10	0.10	1.0	0.05	1.07960	0.15061	2.68
13	14	1.10	0.10	1.0	0.05	1.07930	0.08042	3.00
14	32	1.10	0.10	1.0	0.05	1.08050	0.10772	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

For cold-formed steel members subjected to combined axial load and bending (beam-column), the structural strength can be evaluated by the interaction between a beam and a column. Two interaction criteria are considered in the LRFD criteria for cold-formed steel members:

(1) Stability interaction criterion

(2) Yielding interaction criterion

In the cases that the beam-column is subjected to small axial load, simplified interaction criterion is considered.

The beam-column interaction formulas used in the LRFD criteria for cold-formed steel members are based on the consideration of both 1986 AISI Specification and AISC LRFD Specification.

1. Design Requirements. The axial force and bending moments shall satisfy the following interaction equations:

$$P/P_n+C_{mx}M_x/M_{nx}\alpha_{nx}+C_{my}M_y/M_{ny}\alpha_{ny} \leq 1.0 \tag{5.104}$$

$$P/P_{no}+M_x/M_{nx}+M_y/M_{ny} \leq 1.0 \tag{5.105}$$

When $P/P_n \leq 0.15$, the following formula may be used in lieu of the above two formulas:

$$P/P_n+M_x/M_{nx}+M_y/M_{ny} \leq 1.0 \tag{5.106}$$

where

P = applied axial load

M_x and M_y = applied moments with respect to the the centroidal axes of the effective section determined for the axial load alone. For angle sections, M_y shall be taken either as the applied moment or the applied moment plus PL/1000, whichever result in a lower value of P_n.

P_n = nominal axial strength

P_{no} = nominal axial strength determined with $F_n = F_y$

M_{nx} and M_{ny} = nominal flexural strengths about the centroidal axes

$1/\alpha_{nx}$, $1/\alpha_{ny}$ = magnification factors

$$= 1/(1-P/P_E) \qquad (5.107)$$

$$P_E = \pi^2 EI_b/(K_b L_b)^2 \qquad (5.108)$$

I_b = moment of inertia of the full, unreduced cross section about the axis of bending

L_b = actual unbraced length in the plane of bending

K_b = effective length factor in the plane of bending

C_{mx}, C_{my} = coefficients whose value shall be taken as follows:

(1) For compression members in frames subject to joint translation (sidesway)

$C_m = 0.85$

(2) For restrained compression members in frames braced against joint translation and not subject to transverse loading between their supports in the plane of bending

$$C_m = 0.6-0.4(M_1/M_2) \qquad (5.109)$$

where

M_1/M_2 is the ratio of the smaller to the larger moment at the ends of that portion of the member under consideration which is unbraced in the plane of bending. M_1/M_2 is positive when the member is bent in reverse curvature and negative when it is bent in single curvature.

(3) For compression members in frames braced against joint translation in the plane of loading and subject to transverse loading between their supports, the value of C_m may be determined by rational analysis. However, in lieu of such analysis, the following values may be used:

(a) for members whose ends are restrained, $C_m = 0.85$,

(b) for members whose ends are unrestrained, $C_m = 1.0$.

2. <u>Development of the LRFD Criteria</u>. A total of 144 tests were used in this calibration. The tested failure loads, P_{test}, were obtained from References 118 through 122. The predicted values of P_{pred} were computed according to the interaction formulas mentioned above. The tested and predicted failure loads and their ratios, P_{test}/P_{pred}, were listed in Tables 40 through 48 of Reference 34.

In view of the fact that the modulus of elasticity is the dominant material parameter for elastic buckling and the yield point of steel is a dominant material parameter for inelastic buckling, it is assumed that $M_m = 1.05$ and $V_M = 0.10$. These values are based on $E_m = E$, $V_E = 0.06$,

$(\sigma_y)_m = 1.10\ F_y$ and $V_{\sigma_y/F_y} = 0.10$, where σ_y and F_y are the actual and specified yield points, respectively.

Based on all these values, the safety indexes were computed and summarized in Table XIII. Nine different cases have been studied according to the types of sections (hat sections and lipped channel sections), the stability conditions (locally stable and locally unstable), and the loading conditions. From these results, it can be seen that based on $\phi_c = 0.85$, the values of safety index vary from 2.7 to 3.34 which are satisfactory when compared to the target β of 2.5.

F. STIFFENERS

For the design of cold-formed steel beams, the maximum depth-to-thickness ratio h/t for unreinforced web is limited to 200. For h/t ratios beyond this limit, stiffeners are required. When transverse stiffeners are provided only at supports and/or under concentrated loads, the maximum depth-to-thickness ratio may be increased to 260. When transverse stiffeners and shear stiffeners are used simultaneously, the maximum h/t ratio can be increased to 300. The following discussions deal with the development of the LRFD criteria for transverse stiffeners and shear stiffeners.

1. Transverse Stiffeners. For beams having large h/t ratios, transverse stiffeners may be used at supports and/or under concentrated loads. Two failure modes are considered in the LRFD criteria for cold-formed steel:

(1) End crushing of the transverse stiffeners

(2) Column-type buckling of the transverse stiffeners

Table XIII

Computed Safety Index β for Combined Axial Load and Bending

(Based on ϕ_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	1.0367	0.06619	2.70
2	13	1.05	0.10	1.0	0.05	1.0509	0.07792	2.72
3	33	1.05	0.10	1.0	0.05	1.1028	0.09182	2.86
4	18	1.05	0.10	1.0	0.05	1.1489	0.10478	2.96
5	6	1.05	0.10	1.0	0.05	1.1600	0.13000	2.87
6	17	1.05	0.10	1.0	0.05	1.1200	0.09000	2.92
7	10	1.05	0.10	1.0	0.05	1.2300	0.08000	3.34
8	17	1.05	0.10	1.0	0.05	1.0910	0.07950	2.86
9	12	1.05	0.10	1.0	0.05	1.1110	0.11450	2.79

Note: Case 1 = Locally stable beam-columns, hat sections of Pekoz and Winter (1967)
Case 2 = Locally unstable beam-columns, lipped channel sections of Thomasson (1978)
Case 3 = Locally unstable beam-columns, lipped channel sections of Loughlan (1979)
Case 4 = Locally unstable beam-columns, lipped channel sections of Mulligan and Pekoz (1983)
Case 5 = Locally stable beam-columns, lipped channel sections of Loh and Pekoz (1985) 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 $e_y\neq0$
Case 9 = Locally unstable beam-columns, lipped channel sections of Loh and Pekoz (1985) with $e_x\neq0$ and $e_y\neq0$

The equations for determining nominal strengths of transverse stiffeners used in the LRFD criteria for cold-formed steel members are the same as those included in the 1986 AISI Specification.

a. Design Requirements. Transverse stiffeners attached to beam webs at points of concentrated loads or reactions, shall be designed as compression members. Concentrated loads or reactions shall be applied directly into the stiffeners, or each stiffener shall be fitted accurately to the flat portion of the flange to provide direct load bearing into the end of the stiffener. The nominal strength, P_n, is the smaller value given by (a) and (b) as follows:

(a) $P_n = F_{wy}A_c$ (5.110)

(b) P_n = nominal axial strength evaluated in accordance with strength for axially loaded compression members with A_e replaced by A_b

where

$A_c = 18t^2+A_s$, for transverse stiffeners at interior support and under concentrated load (5.111)

$A_c = 10t^2+A_s$, for transverse stiffeners at end support (5.112)

F_{wy} = lower value of beam web, F_y or stiffener section, F_{ys}

$A_b = b_1t+A_s$, for transverse stiffeners at interior support and under concentrated load (5.113)

$A_b = b_2t+A_s$, for transverse stiffeners at end support (5.114)

A_s = cross sectional area of transverse stiffeners

$b_1 = 25t[0.0024(L_{st}/t)+0.72] \leq 25t$ (5.115)

$b_2 = 12t[0.0044(L_{st}/t)+0.83] \leq 12t$ (5.116)

L_{st} = length of transverse stiffener

t = base thickness of beam web

The w/t_s ratio for the stiffened and unstiffened elements of cold-formed steel transverse stiffeners shall not exceed $1.28\sqrt{(E/F_{ys})}$ and $0.37\sqrt{(E/F_{ys})}$ respectively, where F_{ys} is the yield stress, and t_s the thickness of the stiffener steel.

b. Development of the LRFD Criteria. A total of 61 tests were used in the calibration. The tested failure loads, P_{test}, were obtained from Reference 123. The predicted values, P_{pred}, were computed according to the design formulas mentioned above. The tested and predicted failure loads were listed in Tables 68 and 69 of Reference 34. The mean values and the coefficients of variation of the tested-to-predicted load ratios, P_{test}/P_{pred}, were also included in these tables.

On the basis of the statistical data summarized in Section II.E, the values of M_m, V_M, F_m and V_F are listed in Table XIV. Based on all these values, the safety indices were computed and presented in the same table. This calibration included 3 different cases : (1) transverse stiffeners at interior support and under concentrated load, (2) transverse stiffeners at end support and (3) sum of cases 1 and 2. From these results, it can be seen that the use of $\phi_c = 0.85$ will provide the values of β ranging from 3.32 to 3.41 which exceed considerably the target β of 2.5.

2. Shear Stiffeners. The design requirements for shear stiffeners in the LRFD criteria for cold-formed steel members are the same as those included in the 1986 AISI Specification.

a. Design Requirements. Where shear stiffeners are required, the spacing shall be such that the shear force shall not exceed the nominal shear strength, V_n, determined in accordance with Section V.C, and the ratio a/h shall not exceed $[260/(h/t)]^2$ nor 3.0.

The actual moment of inertia, I_s, of a pair of attached shear stiffeners, or of a single shear stiffener, with reference to an axis in the plane of the web, shall have a minimum value of

$$I_{smin} = 5ht^3[h/a-0.7(a/h)] \geq (h/50)^4 \quad (5.117)$$

The gross area of shear stiffeners shall be not less than

$$A_{st} = [(1-C_v)/2]\{a/h-(a/h)^2/[(a/h)+\sqrt{1+(a/h)^2}]\}YDht \quad (5.118)$$

where

$$C_v = 45{,}000k_v/[F_y(h/t)^2] \text{ when } C_v \leq 0.8 \quad (5.119)$$

$$C_v = [190/(h/t)](\sqrt{k_v/F_y}) \text{ when } C_v > 0.8 \quad (5.120)$$

$$k_v = 4.00 + 5.34/(a/h)^2 \text{ when } a/h \leq 1.0 \quad (5.121)$$

$$k_v = 5.34 + 4.00/(a/h)^2 \text{ when } a/h > 1.0 \quad (5.122)$$

a = distance between transverse stiffeners

Y = yield point of web steel/yield point of stiffener steel

D = 1.0 for stiffeners furnished in pairs

D = 1.8 for single-angle stiffeners

D = 2.4 for single-plate stiffeners

b. Development of the LRFD Criteria. A total of 32 tests were used in the calibration of shear strength of beams with shear stiffeners. The tested failure shear forces, V_{test}, were obtained from Reference 123. The predicted values of V_{pred} were computed according to the design formulas listed in Section V.C. The tested and predicted failure shear forces were listed in Table 70 of Reference 34. It should be noted that because of large amount of postbuckling strength developed in some tests, only 22 tests were used in the statistical analysis. The mean value and the coefficient of variation of the tested-to-predicted load ratios, V_{test}/V_{pred}, were also included in this table (P_m = 1.5982, V_P = 0.0915).

On the basis of the statistical data summarized in Section II.E, the values of M_m, V_M, F_m and V_F were taken as 1.00, 0.06, 1.00 and 0.05, respectively. Based on all these values and ϕ_v = 0.90, the safety index was found to be 4.10 which exceed considerably the target β of 2.5.

G. WALL STUDS AND WALL STUD ASSEMBLIES

The load-carrying capacity of a stud may be computed on the basis that sheathing furnishes adequate lateral and rotational support to the stud in the plane of the wall. Three types of load-carrying capacity are considered in the LRFD criteria for cold-formed steel members:

(1) Wall studs in compression

(2) Wall studs in bending

(3) Wall studs with combined axial load and bending

The equations for determining nominal load-carrying capacities for wall studs used in the LRFD criteria for cold-formed steel members are the same as those included in the 1986 AISI Specification.

1. Design Requirements. For studs having identical sheathing attached to both flanges, and neglecting any rotational restraint provided by the sheathing, the nominal axial strength, P_n, shall be calculated as follows:

$$P_n = A_e F_n \qquad (5.123)$$

where

A_e = effective area determined at F_n

F_n = the lowest value determined by the following three

Table XIV

Computed Safety Index β for Transverse Stiffeners

(ϕ_c = 0.85)

Case	No. of Tests	M_m	V_M	F_m	V_F	P_m	V_P	β
1	33	1.10	0.10	1.0	0.05	1.1762	0.08658	3.32
2	28	1.10	0.10	1.0	0.05	1.2099	0.09073	3.41
3	61	1.10	0.10	1.0	0.05	1.1916	0.08897	3.36

Note: Case 1 = Transverse stiffeners at interior support and under concentrated load
Case 2 = Transverse stiffeners at end support
Case 3 = Sum of Cases 1 and 2

conditions:

(a) To prevent column buckling between fasteners in the plane of the wall, F_n shall be calculated according to Section V.D with KL equal to two times the distance between fasteners.

(b) To prevent flexural and/or torsional overall column buckling, F_n shall be calculated in accordance with Section V.D with F_e taken as the smaller of the two σ_{CR} values specified for the following section types, where σ_{CR} is the theoretical elastic buckling stress under concentric loading.

(1) Singly-symmetric channels and C-Sections

$$\sigma_{CR} = \sigma_{ey}+\bar{Q}_a \quad (5.124)$$

$$\sigma_{CR} = 1/(2\beta)\left[(\sigma_{ex}+\sigma_{tQ})-\sqrt{(\sigma_{ex}+\sigma_{tQ})^2-(4\beta\sigma_{ex}\sigma_{tQ})}\right] \quad (5.125)$$

(2) Z-Sections

$$\sigma_{CR} = \sigma_t+\bar{Q}_t \quad (5.126)$$

$$\sigma_{CR} = 1/2\left\{(\sigma_{ex}+\sigma_{ey}+\bar{Q}_a)-\left[(\sigma_{ex}+\sigma_{ey}+\bar{Q}_a)^2-4(\sigma_{ex}\sigma_{ey}+\sigma_{ex}\bar{Q}_a-\sigma_{exy}{}^2)\right]^{1/2}\right\} \quad (5.127)$$

(3) I-Sections (doubly-symmetric)

$$\sigma_{CR} = \sigma_{ey}+\bar{Q}_a \quad (5.128)$$

$$\sigma_{CR} = \sigma_{ex} \quad (5.129)$$

In the above formulas

$$\sigma_{ex} = \pi^2E/(K_xL_x/r_x)^2 \quad (5.130)$$

$$\sigma_{exy} = (\pi^2EI_{xy})/(AL^2) \quad (5.131)$$

$$\sigma_{ey} = \pi^2E/(K_yL_y/r_y)^2 \quad (5.132)$$

$$\sigma_t = 1/(Ar_o{}^2)\left[GJ+\pi^2EC_w/(K_tL_t)^2\right] \quad (5.133)$$

$$\sigma_{tQ} = \sigma_t+\bar{Q}_t \quad (5.134)$$

$\bar{Q}$ = $\bar{q}B$ = design shear rigidity for sheathing on both sides of the wall assembly (5.135)

$\bar{q}$ = design shear rigidity for sheathing per inch of stud spacing (see Table XV)

B = stud spacing

$\bar{Q}_a = \bar{Q}/A$ (5.136)

A = area of full unreduced cross section

L = length of stud

$\bar{Q}_t = (\bar{Q}d^2)/(4Ar_o^2)$ (5.137)

d = depth of section

I_{xy} = product of inertia

(c) To prevent shear failure of the sheathing, a value of F_n shall be used in the following equations so that the shear strain of the sheathing, γ, does not exceed the permissible shear strain, $\bar{\gamma}$. The shear strain, γ, shall be determined as follows:

$$\gamma = (\pi/L)[C_1+(E_1 d/2)] \quad (5.138)$$

where

C_1 and E_1 are the absolute values of C_1 and E_1 specified below for each section type:

(1) Singly-Symmetric Channels

$$C_1 = (F_n C_o)/(\sigma_{ey}-F_n+\bar{Q}_a) \quad (5.139)$$

$$E_1 = \frac{F_n[(\sigma_{ex}-F_n)(r_o^2 E_o-x_o D_o)-F_n x_o(D_o-x_o E_o)]}{(\sigma_{ex}-F_n)r_o^2(\sigma_{tQ}-F_n)-(F_n x_o)^2} \quad (5.140)$$

(2) Z-Sections

$$C_1 = \frac{F_n[C_o(\sigma_{ex}-F_n)-D_o\sigma_{exy}]}{(\sigma_{ey}-F_n+\bar{Q}_a)(\sigma_{ex}-F_n)-\sigma_{exy}^2} \quad (5.141)$$

$$E_1 = (F_n E_o)/(\sigma_{tQ}-F_n) \quad (5.142)$$

(3) I-Sections

$C_1 = (F_n C_o)/(\sigma_{ey} - F_n + \bar{Q}_a)$ (5.143)

$E_1 = 0$

where

x_o = distance from shear center to centroid along principal x-axis, in. (absolute value)

C_o, E_o, and D_o are initial column imperfections which shall be assumed to be at least

C_o = L/350 in a direction parallel to the wall (5.144)

D_o = L/700 in a direction perpendicular to the wall (5.145)

E_o = L/(dx10,000), rad., a measure of the initial twist of the stud from the initial, ideal, unbuckled shape. (5.146)

If $F_n > 0.5F_y$, then in the definitions for σ_{ey}, σ_{ex}, σ_{exy} and σ_{tQ}, the parameters E and G shall be replaced by E' and G', respectively, as defined below

$E' = 4EF_n(F_y - F_n)/F_y^2$ (5.147)

$G' = G(E'/E)$ (5.148)

Sheathing parameters q_o and γ may be determined from representative full-scale tests, or from the small-scale-test values given in Table XV.

For studs having identical sheathing attached to both flanges, and neglecting any rotational restraint provided by the sheathing, the nominal flexural strengths are M_{nxo} and M_{nyo}, where

M_{nxo} and M_{nyo} = nominal flexural strengths about the centroidal axes determined in accordance with Section V.C, excluding lateral buckling

Table XV

Sheathing Parameters[1]

Sheathing[2]	$\bar{q}_o$[3] k/in.	$\bar{\gamma}$ in./in.
3/8 to 5/8 in. thick gypsum	2.0	0.008
Lignocellulosic board	1.0	0.009
Fiberboard (regular or impregnated)	0.6	0.007
Fiberboard (heavy impregnated)	1.2	0.010

(1) The values given are subject to the following limitations:
All values are for sheathing on both sides of the wall assembly.
All fasteners are No. 6, type S-12, self-drilling drywall screws with pan or bugle head, or equivalent, at 6-to 12-inch spacing.
(2) All sheathing is 1/2-inch thick except as noted.
(3) $\bar{q} = \bar{q}_o(2-s/12)$
where s = fastener spacing, in.
For other types of sheathing, $\bar{q}_o$ and $\bar{\gamma}$ may be determined conservatively from representative small-specimen tests

For wall studs with combined axial load and bending, the axial load and bending moment shall satisfy the interaction equations of Section V.E with the following redefined terms:

P_n = nominal axial strength determined according to Section V.D

M_{nx} and M_{ny} in Equations (5.104), (5.105), and (5.106) shall be replaced by nominal flexural strengths, M_{nxo} and M_{nyo}, respectively.

2. <u>Development of the LRFD Criteria</u>. (1) Due to the lack of sufficient test data on wall studs in compression, only 7 tests were used in the calibration. The tested failure loads, P_{test}, were obtained from Reference 124. The predicted values, P_{pred}, were computed according to the design formulas mentioned above. The tested and predicted failure loads were listed in Table 71 of Reference 34. The mean value and the coefficient of variation of the tested-to-predicted load ratios, P_{test}/P_{pred}, were also included in this table (P_m = 1.1363, V_P = 0.095). Based on M_m = 1.10, V_M = 0.10, F_m = 1.0, V_F = 0.05, and ϕ = 0.85, the value of β was found to be 3.14 which is larger than the the target β of 2.5.

(2) The test data on wall studs in bending are very limited. Only two tests with stiffened compression flanges were used in the calibration. The tested ultimate moments, M_{test}, were obtained from Reference 125. The predicted values, M_{pred}, were computed according to the design formulas mentioned above. The tested and predicted ultimate moments were listed in Table 72 of Reference 34. The mean value and the coefficient of variation of the tested-to-predicted moment ratios, M_{test}/M_{pred}, were also included in this table (P_m = 1.266, V_P = 0.0073). Based on M_m = 1.10,

$V_M = 0.10$, $F_m = 1.0$, $V_F = 0.05$, and $\phi = 0.95$, the value of β was found to be 3.37 which is larger than the the target β of 2.5.

(3) For wall studs with combined axial load and bending, only 10 tests of wall studs with stiffened compression flanges were used in the calibration. The tested failure loads, P_{test}, were obtained from Reference 125. The predicted values, P_{pred}, were computed according to the design formulas mentioned above. The tested and predicted failure loads were listed in Table 73 of Reference 34. The mean value and the coefficient of variation of the tested-to-predicted load ratios, P_{test}/P_{pred}, were also included in this table ($P_m = 1.1876$, $V_P = 0.1338$). Based on $M_m = 1.05$, $V_M = 0.10$, $F_m = 1.0$, $V_F = 0.05$, and $\phi_c = 0.85$, the value of β was found to be 2.94 which is larger than the the target β of 2.5.

H. WELDED CONNECTIONS

For the design of welded connections, six types of welds are considered in the LRFD criteria for cold-formed steel members:

(1) Groove welds in butt joints

(2) Arc spot welds

(3) Arc seam welds

(4) Fillet welds

(5) Flare groove welds

(6) Resistance welds

The equations for determining nominal strength of welds used in the LRFD criteria for cold-formed steel members are basically the same as those included in the 1986 AISI Specification, except that the design equations for the nominal strength, and the ϕ factors for groove welds in butt joints are adopted from the AISC LRFD criteria. The nominal shear strengths of resistance welds are derived from the allowable values specified in the 1986 AISI Specification by using a safety factor of 2.5.

1. Design Requirements. (1) The nominal strength, P_n, of a groove weld in a butt joint, welded from one or both sides, shall be determined as follows:

(a) Tension or compression normal to the effective area or parallel to the axis of the weld

$$P_n = Lt_eF_y \tag{5.149}$$

(b) Shear on the effective area

$$P_n = Lt_e(0.6F_{xx}); \text{ and} \tag{5.150}$$

$$P_n = Lt_e(F_y/\sqrt{3}) \tag{5.151}$$

where

F_{xx} = strength level designation in AWS electrode classification

F_y = specified minimum yield point of the lower strength base steel

L = length of weld

t_e = effective throat dimension for groove weld

(2) The nominal shear strength, P_n, of each arc spot weld between sheet or sheets and supporting member shall be determined by using the smaller of either

(a) $$P_n = 0.589d_e^2F_{xx}; \text{ or} \tag{5.152}$$

(b) For $(d_a/t) \leq 0.815\sqrt{(E/F_u)}$:

$$P_n = 2.20td_aF_u \tag{5.153}$$

For $0.815\sqrt{(E/F_u)} < (d_a/t) < 1.397\sqrt{(E/F_u)}$:

$$P_n = 0.280[1+5.59\sqrt{E/F_u}/(d_a/t)]td_aF_u \tag{5.154}$$

For $(d_a/t) \geq 1.397\sqrt{(E/F_u)}$:

$$P_n = 1.40td_aF_u \quad (5.155)$$

where

d = visible diameter of outer surface of arc spot weld

d_a = average diameter of the arc spot weld at mid-thickness of t [where d_a = (d-t) for a single sheet, and (d-2t) for multiple sheets (not more than four lapped sheets over a supporting member)]

d_e = effective diameter of fused area

$$d_e = 0.7d-1.5t \text{ but } \leq 0.55d \quad (5.156)$$

t = total combined base steel thickness (exclusive of coatings) of sheets involved in shear transfer

F_{xx} = stress level designation in AWS electrode classification

F_u = tensile strength

The distance measured in the line of force from the centerline of a weld to the nearest edge of an adjacent weld or to the end of the connected part toward which the force is directed shall not be less than the value of e as given below:

$$e = P/(F_ut) \quad (5.157)$$

where

P = force transmitted by weld

t = thickness of thinnest connected sheet

F_{sy} = specified yield point

(3) The nominal tensile strength, P_n, on each arc spot weld between sheet and supporting member, shall be determined as follows:

$$P_n = 0.7td_aF_u \quad (5.158)$$

In using arc spot welds in tension, the following additional limitations shall apply for using Eq. (5.158):

e_{min} = minimum distance measured in the line of force from the centerline of a weld to the nearest edge of an adjacent weld or to the end of the connected part toward which the force is directed

$\geq d$

$F_u \leq 60$ ksi

$F_{xx} \geq 60$ ksi

t = Thickness of connected sheet ≥ 0.031 in.

(4) The nominal shear strength, P_n, of arc seam welds shall be determined by using the smaller of either

(a) $P_n = (\pi d_e^2/4 + Ld_e)(0.75F_{xx})$; or (5.159)

(b) $P_n = 2.5tF_u(0.25L + 0.96d_a)$ (5.160)

where

d = width of arc seam weld

L = length of seam weld not including the circular ends (For computation purposes, L shall not exceed 3d)

d_a = average width of seam weld

where

$d_a = (d-t)$ for a single sheet, and (5.161)

$(d-2t)$ for a double sheet (5.162)

d_e = effective width of arc seam weld at fused surfaces

$d_e = 0.7d - 1.5t$ (5.163)

(5) The nominal shear strength, P_n, of a fillet weld shall be determined as follows:

For $(d_a/t) \geq 1.397\sqrt{(E/F_u)}$:

$$P_n = 1.40td_aF_u \quad (5.155)$$

where

d = visible diameter of outer surface of arc spot weld

d_a = average diameter of the arc spot weld at mid-thickness of t [where d_a = (d-t) for a single sheet, and (d-2t) for multiple sheets (not more than four lapped sheets over a supporting member)]

d_e = effective diameter of fused area

$$d_e = 0.7d-1.5t \text{ but } \leq 0.55d \quad (5.156)$$

t = total combined base steel thickness (exclusive of coatings) of sheets involved in shear transfer

F_{xx} = stress level designation in AWS electrode classification

F_u = tensile strength

The distance measured in the line of force from the centerline of a weld to the nearest edge of an adjacent weld or to the end of the connected part toward which the force is directed shall not be less than the value of e as given below:

$$e = P/(F_ut) \quad (5.157)$$

where

P = force transmitted by weld

t = thickness of thinnest connected sheet

F_{sy} = specified yield point

(3) The nominal tensile strength, P_n, on each arc spot weld between sheet and supporting member, shall be determined as follows:

$$P_n = 0.7td_aF_u \quad (5.158)$$

In using arc spot welds in tension, the following additional limitations shall apply for using Eq. (5.158):

e_{min} = minimum distance measured in the line of force from the centerline of a weld to the nearest edge of an adjacent weld or to the end of the connected part toward which the force is directed

$\geq d$

$F_u \leq 60$ ksi

$F_{xx} \geq 60$ ksi

t = Thickness of connected sheet ≥ 0.031 in.

(4) The nominal shear strength, P_n, of arc seam welds shall be determined by using the smaller of either

(a) $P_n = (\pi d_e^2/4 + Ld_e)(0.75F_{xx})$; or (5.159)

(b) $P_n = 2.5tF_u(0.25L + 0.96d_a)$ (5.160)

where

d = width of arc seam weld

L = length of seam weld not including the circular ends (For computation purposes, L shall not exceed 3d)

d_a = average width of seam weld

where

$d_a = (d-t)$ for a single sheet, and (5.161)

$(d-2t)$ for a double sheet (5.162)

d_e = effective width of arc seam weld at fused surfaces

$d_e = 0.7d - 1.5t$ (5.163)

(5) The nominal shear strength, P_n, of a fillet weld shall be determined as follows:

(a) For longitudinal loading:

For $L/t < 25$:

$$P_n = (1-0.01L/t)tLF_u \quad (5.164)$$

For $L/t \geq 25$:

$$P_n = 0.75tLF_u \quad (5.165)$$

(b) For transverse loading:

$$P_n = tLF_u \quad (5.166)$$

where

t = least value of thickness of two plates

In addition, for $t > 0.150$ inch the nominal strength determined above shall not exceed the following value of P_n:

$$P_n = 0.75t_wLF_{xx} \quad (5.167)$$

where

L = length of fillet weld

t_w = effective throat = $0.707w_1$ or $0.707w_2$, whichever is smaller.

(6) The nominal shear strength, P_n, of a flare groove weld shall be determined as follows:

(a) For flare-bevel groove welds, transverse loading:

$$P_n = 0.833tLF_u \quad (5.168)$$

(b) For flare groove welds, longitudinal loading:

For $t \leq t_w < 2t$ or if the lip height is less than weld length, L:

$$P_n = 0.75tLF_u \quad (5.169)$$

For $t_w \geq 2t$ and the lip height is equal to or greater than L:

$$P_n = 1.50tLF_u \quad (5.170)$$

In addition, if $t > 0.15$ inch, the nominal strength determined above shall not exceed the following value of P_n:

Table XVI

Nominal Shear Strength of Spot Welding

Thickness of Thinnest Outside Sheet, in.	Shear Strength per spot kips	Thickness of Thinnest Outside Sheet, in.	Shear Strength per spot kips
0.010	0.125	0.080	3.325
0.020	0.438	0.094	4.313
0.030	1.000	0.109	5.988
0.040	1.425	0.125	7.200
0.050	1.650	0.188	10.000
0.060	2.275	0.250	15.000

$$P_n = 0.75 t_w L F_{xx} \tag{5.171}$$

(7) The nominal shear strength, P_n, of spot welding shall be determined in accordance with Table XVI.

2. Development of the LRFD Criteria. (1) For shear strength of arc spot welds, 32 tests were used in the calibration. The tested loads, P_{test}, were obtained from Reference 126 and the predicted values, P_{pred}, were computed from the design formulas mentioned above. The tested and predicted loads with the mean value and coefficient of variation of their ratios, P_{test}/P_{pred}, were listed in Table 49 of Reference 34 ($P_m = 1.173$, $V_P = 0.217$). The mean value of the material factors, M_m, was taken as 1.10. The mean value of the fabrication factors, F_m, was assumed to be equal to unity. The coefficient of variation of the material properties, V_M, was taken as 0.10 and the coefficient of variation of the fabrication factors, V_F, was assumed to be 0.10. By using these values and $\phi = 0.60$, the value of β was found to be 3.55 which is larger than the target β of 3.5.

(2) With regard to the type of plate failure considered in the design criteria, the ϕ factors used, and the safety indices computed, are listed in Table XVII. All the statistical data presented in Table XVII were obtained from Reference 28. It can be seen that for all cases the β values are larger than the target β of 3.5.

(3) For tensile strength of arc spot welds, 103 tests were used in the calibration. The tested loads, P_{test}, were obtained from References 127 and 128, and the predicted values, P_{pred}, were computed from the design formulas mentioned above. The tested and predicted loads with the mean value and coefficient of variation of their ratios, P_{test}/P_{pred}, are

listed in Table XVIII. The values of P_m, M_m, F_m, V_P, V_M, and V_F are presented in Table XIX. Two cases were considered in the determination of ϕ factor: 1) $1.2D_n+1.6L_n$ with $\beta_o = 3.5$, and 2) $1.17W_n-0.9D_n$ with $\beta_o = 2.5$ (counteracting loads with a reduction factor of 0.9 applied to the load factor for the nominal wind load). $\phi = 0.65$ was selected for both cases and the values of β corresponding to this value of ϕ are given in Table XIX. It can be seen that for both cases, the β values compare satisfactorily to the target reliability indices.

(4) For plate tearing of arc seam welds, 23 tests were used in the calibration. The tested loads, P_{test}, were obtained from Reference 126. The predicted values, P_{pred}, were computed from the design formulas mentioned above. The tested and predicted loads with the mean value and coefficient of variation of their ratios were listed in Table 50 of Reference 34 ($P_m = 1.004$, $V_P = 0.095$). Based on $M_m = 1.10$, $V_M = 0.10$, $F_m = 1.0$, $V_F = 0.10$, and $\phi = 0.60$, the value of β was found to be 3.81 which is larger than the target β of 3.5.

(5) For fillet welds, the ϕ factors used in the calibration and the safety indices computed for longitudinal and transverse loading are listed in Table XVII. All the statistical data presented in this table were obtained from Reference 28. It can be seen that for all cases, the β values are larger than the target β of 3.5.

(6) For plate tearing failure of transverse flare bevel welds, 42 tests were reported in Reference 126. They were used in the calibration. The tested and predicted loads with the mean value and coefficient of variation of their ratios were listed in Table 51 of Reference 34 ($P_m = 1.04$, $V_P = 0.165$). Based on $M_m = 1.10$, $V_M = 0.10$, $F_m = 1.0$, $V_F = 0.10$,

and $\phi = 0.55$, the value of β was found to be 3.81 which is larger than the target β of 3.5.

(7) For plate tearing failure of longitudinal flare bevel welds, 10 tests were reported in Reference 126. They were used in the calibration. The tested and predicted loads with the mean value and coefficient of variation of their ratios were listed in Table 52 of Reference 34 (P_m = 0.969, V_P = 0.169). Based on F_m = 1.10, V_M = 0.10, F_m = 1.0, V_F = 0.10, and $\phi = 0.55$, the value of β was found to be 3.56 which is larger than the target β of 3.5.

(8) For resistance welds, 13 tests were used in the calibration. The test loads were obtained from References 129 and 130. The predicted loads were based on Table XVI. The tested and predicted loads with the mean value and coefficient of variation of their ratios were listed in Table 53 of Reference 34 (P_m = 0.999, V_P = 0.0266). Based on M_m = 1.10, V_M = 0.10, F_m = 1.00, V_F = 0.10, and $\phi = 0.65$, the value of β was found to be 3.71 which is larger than the target β of 3.5.

I. BOLTED CONNECTIONS

For the design of bolted connections, four design provisions are included in the LRFD criteria for cold-formed steel members to consider various failure modes:

(1) Spacing and edge distance

(2) Tension in connected part

(3) Bearing

(4) Shear and tension in bolts

Table XVII

Computed Safety Index β for Plate Failure in Welded Connections

Case	M_m	V_M	F_m	V_F	P_m	V_P	Φ	β
Arc Spot Welds								
1	1.10	0.08	1.00	0.15	1.10	0.17	0.60	3.52
2	1.10	0.08	1.00	0.15	0.98	0.18	0.50	3.64
Fillet Welds								
3	1.10	0.08	1.00	0.15	1.01	0.08	0.60	3.65
4	1.10	0.08	1.00	0.15	0.89	0.09	0.55	3.59
5	1.10	0.08	1.00	0.15	1.05	0.11	0.60	3.72

Note: Case 1 = For $d_a/t \leq 0.815\sqrt{(E/F_u)}$

Case 2 = For $d_a/t > 1.397\sqrt{(E/F_u)}$

Case 3 = Longitudinal Loading, $L/t < 25$

Case 4 = Longitudinal Loading, $L/t \geq 25$

Case 5 = Transverse Loading

Table XVIII

Comparison of Tested and Predicted Tensile Strengths of Arc Spot Welds

Specimen	t (in.)	d (in.)	F_u (ksi)	P_{pred} (lbs.)	P_{test} (lbs.)	$\frac{P_{test}}{P_{pred}}$
2AT-107	0.059	0.69	52.0	1355	1620	1.1955
2AT-108	0.059	0.88	52.0	1763	2420	1.3725
2AT-109	0.059	0.69	52.0	1355	2560	1.8891
2AT-207	0.059	0.63	52.0	1226	2360	1.9245
2AT-208	0.059	0.63	52.0	1226	2080	1.6962
2AT-209	0.059	0.66	52.0	1291	2560	1.9834
2AT-314	0.059	0.72	52.0	1420	2920	2.0570
2AT-315	0.059	0.75	52.0	1484	1630	1.0984
2AT-316	0.059	0.75	52.0	1484	2140	1.4421
2AT-317	0.059	0.56	52.0	1076	1893	1.7594
2AT-318	0.059	0.56	52.0	1076	2586	2.4035
2AT-319	0.059	0.59	52.0	1140	1864	1.6346
2AT-320	0.059	0.59	52.0	1140	2745	2.4071
2AT-101	0.031	0.88	60.1	1105	740	0.6694
2AT-102	0.031	0.88	60.1	1105	630	0.5699
2AT-103	0.031	0.88	60.1	1105	870	0.7871
2AT-201	0.031	0.72	60.1	897	1260	1.4046
2AT-202	0.031	0.69	60.1	858	1160	1.3520
2AT-203	0.031	0.59	60.1	728	760	1.0442
2AT-301	0.031	0.75	60.1	936	1070	1.1430
2AT-302	0.031	0.72	60.1	897	1350	1.5049
2AT-303	0.031	0.69	60.1	858	1110	1.2937
2AT-304	0.035	0.72	54.6	916	1380	1.5060
2AT-305	0.035	0.69	54.6	876	1310	1.4951
2AT-306	0.035	0.69	54.6	876	1210	1.3810
2AT-307	0.035	0.69	54.6	876	1425	1.6264
2AT-308	0.035	0.72	54.6	916	1164	1.2703
2AT-309	0.035	0.66	54.6	836	1429	1.7092
2AT-310	0.035	0.66	54.6	836	1014	1.2128
2AT-104	0.049	0.81	51.0	1331	1400	1.0517
2AT-105	0.049	0.81	51.0	1331	1680	1.2620
2AT-106	0.049	0.75	51.0	1226	1620	1.3211
2AT-204	0.049	0.69	51.0	1121	1680	1.4983
2AT-205	0.049	0.66	51.0	1069	2140	2.0022
2AT-206	0.049	0.63	51.0	1016	1620	1.5940

Table XVIII (Continued)

Specimen	t (in.)	d (in.)	F_u (ksi)	P_{pred} (lbs.)	P_{test} (lbs.)	$\frac{P_{test}}{P_{pred}}$
2AT-311	0.049	0.75	51.0	1226	1860	1.5168
2AT-312	0.049	0.81	51.0	1331	1950	1.4648
2AT-313	0.049	0.75	51.0	1226	1930	1.5739
3BT-101	0.032	0.81	64.4	1046	1040	0.9946
3BT-102	0.032	0.75	64.4	965	910	0.9430
3BT-103	0.032	0.75	64.4	965	1170	1.2125
3BT-201	0.032	0.72	64.4	925	1340	1.4492
3BT-202	0.032	0.63	64.4	804	1060	1.3189
3BT-203	0.032	0.66	64.4	844	840	0.9952
3BT-104	0.047	0.81	50.0	1255	2120	1.6891
3BT-105	0.047	0.81	50.0	1255	1855	1.4779
3BT-106	0.047	0.94	50.0	1469	2130	1.4500
3BT-204	0.047	0.66	50.0	1008	1680	1.6660
3BT-205	0.047	0.63	50.0	959	2140	2.2314
3BT-206	0.047	0.63	50.0	959	1620	1.6892
3BT-107	0.072	0.88	68.5	2443	2780	1.1378
3BT-108	0.072	0.81	68.5	2232	2680	1.2009
3BT-109	0.072	0.88	68.5	2443	2680	1.0968
3BT-207	0.072	0.66	68.5	1778	3380	1.9009
3BT-208	0.072	0.66	68.5	1778	2460	1.3835
3BT-209	0.072	0.72	68.5	1960	3140	1.6024
3CT-401	0.035	0.69	54.6	876	800	0.9130
3CT-402	0.035	0.56	54.6	702	980	1.3954
3CT-403	0.035	0.59	54.6	742	1570	2.1147
3CT-404	0.035	0.65	54.6	823	1000	1.2155
3CT-405	0.035	0.69	54.6	876	890	1.0158
3CT-406	0.035	0.63	54.6	796	970	1.2187
3CT-411	0.035	0.88	54.6	1130	1100	0.9731
3CT-412	0.035	0.81	54.6	1037	910	0.8778
3CT-413	0.035	0.75	54.6	956	1600	1.6728
3CT-414	0.035	0.94	54.6	1211	1470	1.2143
3CT-415	0.035	0.81	54.6	1037	1470	1.4179
3CT-416	0.035	0.88	54.6	1130	1270	1.1235
3CT-407	0.059	0.47	52.0	883	1778	2.0144
3CT-408	0.059	0.75	52.0	1484	1879	1.2662
3CT-409	0.059	0.59	52.0	1140	2866	2.5132
3CT-410	0.059	0.53	52.0	1012	1944	1.9219
3DT-301	0.035	0.50	54.6	622	1014	1.6302
3DT-302	0.035	0.66	54.6	836	981	1.1734
3DT-303	0.035	0.63	54.6	796	967	1.2149

Table XVIII (Continued)

Specimen	t (in.)	d (in.)	F_u (ksi)	P_{pred} (lbs.)	P_{test} (lbs.)	$\frac{P_{test}}{P_{pred}}$
3DT-401	0.035	0.69	54.6	876	1025	1.1698
3DT-402	0.035	0.69	54.6	876	1291	1.4734
3DT-403	0.035	0.59	54.6	742	1118	1.5059
3DT-411	0.035	0.66	54.6	836	885	1.0585
3DT-412	0.035	0.63	54.6	796	1398	1.7564
3DT-413	0.035	0.59	54.6	742	1610	2.1686
3DT-414	0.035	0.59	54.6	742	2031	2.7356
3DT-415	0.035	0.63	54.6	796	937	1.1772
3DT-404	0.059	0.50	52.0	947	2014	2.1265
3DT-405	0.059	0.56	52.0	1076	2577	2.3951
3DT-406	0.059	0.65	52.0	1269	2435	1.9185
3DT-407	0.059	0.63	52.0	1226	2778	2.2654
3DT-408	0.059	0.53	52.0	1012	2153	2.1285
3DT-409	0.059	0.69	52.0	1355	2033	1.5002
3DT-416	0.059	0.63	52.0	1226	2853	2.3266
3DT-417	0.059	0.59	52.0	1140	2839	2.4895
3DT-418	0.059	0.59	52.0	1140	2137	1.8739
3DT-419	0.059	0.66	52.0	1291	2336	1.8099
3DT-420	0.059	0.59	52.0	1140	1775	1.5565
3DT-421	0.059	0.56	52.0	1076	3037	2.8226
3ET-401	0.035	0.66	54.6	836	1141	1.3647
3ET-402	0.035	0.63	54.6	796	1068	1.3418
3ET-403	0.035	0.66	54.6	836	864	1.0334
3ET-404	0.035	0.66	54.6	836	1129	1.3504
3ET-417	0.059	0.63	52.0	1226	1918	1.5641
3ET-418	0.059	0.53	52.0	1012	2122	2.0978
3ET-419	0.059	0.66	52.0	1291	2241	1.7363
3ET-420	0.059	0.63	52.0	1226	1816	1.4809

Number of specimens $N = 103$
Mean $P_m = 1.5405$
Coefficient of Variation $V_P = 0.2949$

Table XIX

Computed Safety Index β for Tensile Strength of Arc Spot Weld

(ϕ = 0.65)

Case	No. of Tests	M_m	V_M	F_m	V_F	P_m	V_P	β
1	103	1.10	0.08	1.0	0.15	1.5405	0.2949	3.45
2	103	1.10	0.08	1.0	0.15	1.5405	0.2949	2.62

Note: Case 1 is for $1.2D_n+1.6L_n$ (β_o = 3.5)

Case 2 is for $1.17W_n-0.9D_n$ (β_o = 2.5)

The equations for determining nominal strengths of bolted connections used in the LRFD criteria for cold-formed steel members are basically the same as those included in the 1986 AISI Specification except that the nominal shear and tensile strengths, and the ϕ factors, for the high strength bolts are adopted from the AISC LRFD criteria.

1. Design Requirements. (1) For the design of spacing and edge distance of bolted connections, the nominal shear strength, P_n, of the connected part along two parallel lines in the direction of applied force shall be determined as follows:

$$P_n = teF_u \tag{5.172}$$

where

- e = the distance measured in the line of force from the center of a standard hole to the nearest edge of an adjacent hole or to the end of the connected part
- t = thickness of thinnest connected part
- F_u = tensile strength of the connected part
- F_{sy} = yield point of the connected part

(2) The nominal tensile strength, P_n, on the net section of the connected part shall be determined as follows:

(a) Washers are provided under both the bolt head and the nut

$$P_n = (1.0-0.9r+3rd/s)F_uA_n \le F_uA_n \tag{5.173}$$

(b) Either washers are not provided under the bolt head and nut, or only one washer is provided under either the bolt head or nut

$$P_n = (1.0-r+2.5rd/s)F_uA_n \le F_uA_n \tag{5.174}$$

where

A_n = net area of the connected part

r = force transmitted by the bolt or bolts at the section considered, divided by the tension force in the member at that section. If r is less than 0.2, it may be taken equal to zero.

s = spacing of bolts perpendicular to line of stress. In the case of a single bolt, s = Width of sheet

(3) The nominal bearing strength, P_n, shall be determined by the values given in Tables XX and XXI for the applicable thickness and F_u/F_{sy} ratio of the connected part and the type of joint used in the connection.

(4) The nominal shear or tensile strength, P_n, of bolts shall be determined as follows:

$$P_n = A_b F_n \tag{5.175}$$

where

A_b = gross cross-sectional area of bolt

F_n is given by F_{nv} or F_{nt} in Table XXII

When bolts are subject to a combination of shear and tension, the tension force shall not exceed the nominal tensile strength $P_n = A_b F'_{nt}$, where F'_{nt} is given in Table XXIII, in which f_v is the shear stress produced by the same loads. The shear force shall not exceed the nominal shear strength, $A_b F_{nv}$, determined in accordance with Table XXII.

2. Development of the LRFD Criteria. (1) For the calibration of minimum spacing and edge distance in the line of stress, the mean value M_m computed by $(F_u)_{test}/(F_u)_{specified}$, was 1.10. F_m was assumed to be 1.00 and P_m was determined according to P_t/P_p, in which P_t is the tested failure load, and P_p is the predicted failure load. The tested values were obtained from References 131 through 137. The tested and predicted

Table XX

Nominal Bearing Strength for Bolted Connections

With Washers Under Both Bolt Head and Nut

Thickness of Connected Part in.	Type of Joint	F_u/F_{sy} ratio of Connected Part	Nominal Resistance P_n
≥ 0.024 but < 3/16	Inside sheet of double shear connection	≥ 1.15	$3.33F_udt$
		< 1.15	$3.00F_udt$
	Single shear and outside sheets of double shear connection	No limit	$3.00F_udt$
≥ 3/16	See AISC LRFD Specification		

Table XXI

Nominal Bearing Strength for Bolted Connections

Without Washers Under Both Bolt Head and Nut,

or With Only One Washer

Thickness of Connected Part in.	Type of Joint	F_u/F_{sy} ratio of Connected Part	Nominal Resistance P_n
≥ 0.036 but < 3/16	Inside sheet of double shear connection	≥ 1.15	$3.00F_u dt$
	Single shear and outside sheets of double shear connection	≥ 1.15	$2.22F_u dt$
≥ 3/16	See AISC LRFD Specification		

Table XXII

Nominal Tensile and Shear Strengths for Bolts

Description of Bolts	Tensile Strength F_{nt}	Shear Strength F_{nv}
A307 Bolts, Grade A (1/4 in. ≤ d < 1/2 in.)	40.5	24.0
A307 Bolts, Grade A (d ≥ 1/2 in.)	45.0	27.0
A325 bolts, when threads are not excluded from shear planes	90.0	54.0
A325 bolts, when threads are excluded from shear planes	90.0	72.0
A354 Grade B Bolts (1/4 in. ≤ d < 1/2 in.), when threads are not excluded from shear planes	101.0	59.0
A354 Grade B Bolts (1/4 in. ≤ d < 1/2 in.), when threads are excluded from shear planes	101.0	90.0
A449 Bolts (1/4 in. ≤ d < 1/2 in.), when threads are not excluded from shear planes	81.0	47.0
A449 Bolts (1/4 in. ≤ d < 1/2 in.), when threads are excluded from shear planes	81.0	72.0
A490 Bolts, when threads are not excluded from shear planes	112.5	67.5
A490 Bolts, when threads are excluded from shear planes	112.5	90.0

Table XXIII

Nominal Tension Stress, F'_{nt}, for Bolts

Subject to the Combination of Shear and Tension

Description of Bolts	Threads Not Excluded from Shear Planes	Threads Excluded from Shear Planes
A325 Bolts	$113-2.4f_v \leq 90$	$113-1.9f_v \leq 90$
A354 Grade BD Bolts	$127-2.4f_v \leq 101$	$127-1.9f_v \leq 101$
A449 Bolts	$101-2.4f_v \leq 81$	$101-1.9f_v \leq 81$
A490 Bolts	$141-2.4f_v \leq 112.5$	$141-1.9f_v \leq 112.5$
A307 Bolts, Grade A when 1/4 in. ≤ d < 1/2 in.	$47-2.4f_v \leq 40.5$	
when d ≥ 1/2 in.	$52-2.4f_v \leq 45$	

Note: The general form for formulas listed in this table can be written as $C_1 - D_1 f_v \leq Z_1$

failure loads were listed in Tables 54 through 59 of Reference 34 for six different cases. The mean values and coefficients of variation of professional factors, P, are summarized as follows:

Case 1. Single shear, with washers, $F_u/F_y \geq 1.15$ (49 tests)

$P_m = 1.13$, $V_P = 0.12$ (see Table 54 of Reference 34)

Case 2. Double shear, with washers, $F_u/F_y \geq 1.15$ (39 tests)

$P_m = 1.18$, $V_P = 0.14$ (see Table 55 of Reference 34)

Case 3. Single shear, with washers, $F_u/F_y < 1.15$ (7 tests)

$P_m = 0.84$, $V_P = 0.05$ (see Table 56 of Reference 34)

Case 4. Double shear, with washers, $F_u/F_y < 1.15$ (10 tests)

$P_m = 0.94$, $V_P = 0.09$ (see Table 57 of Reference 34)

Case 5. Single shear, without washers, $F_u/F_y \geq 1.15$ (8 tests)

$P_m = 1.06$, $V_P = 0.11$ (see Table 58 of Reference 34)

Case 6. Single shear, without washers, $F_u/F_y < 1.15$ (8 tests)

$P_m = 1.14$, $V_P = 0.19$ (see Table 59 of Reference 34)

Based on all these values, the safety indices were computed and summarized in Table XXIV. By using different ϕ factors for different cases, the values of β vary from 3.61 to 3.90 which are larger than the target β of 3.5.

(2) For the calibration of tension stress on net section, $M_m = 1.10$ and $F_m = 1.00$. The mean value P_m was determined from the ratios of $(\sigma_{net})_t/(\sigma_{net})_p$, in which $(\sigma_{net})_t$ is the tested value and $(\sigma_{net})_p$ is the predicted value. The tested values were obtained from the experimental data given in References 131, 132 and 136. The tested and predicted values were listed in Tables 60 and 61 of Reference 34. The following is a summary of P_m and V_P for three different cases:

Case 7. $t < 3/16$ in., double shear, with washers (51 tests)

$P_m = 1.14$, $V_P = 0.20$ (see Table 60 of Reference 34)

Case 8. $t < 3/16$ in., single shear, with washers (58 tests)

$P_m = 0.95$, $V_P = 0.21$ (see Table 61 of Reference 34)

Case 9. $t < 3/16$ in., single shear, without washers (37 test data presented in Figure 10 of Reference 137)

$P_m = 1.04$, $V_P = 0.14$

Based on all these values, the safety indices were computed and summarized in Table XXIV. By using different ϕ factors for different cases, the values of β vary from 3.41 to 3.63, which are satisfactory when compared to the target β of 3.5.

(3) For the calibration of bearing stress in bolted connections, $M_m = 1.10$ and $F_m = 1.00$. The mean value P_m was determined from the ratios of P_t/P_p, in which P_t is the tested failure load, and P_p is the predicted failure load. The tested values were obtained from References 131 through 137. The tested and predicted failure loads were listed in Tables 62 through 67 of Reference 34 for six different cases. The mean values and coefficients of variation of professional factor, P, are summarized as follows:

Case 10. $0.024 \le t < 3/16$ in., double shear, with washers,

$F_u/F_y \ge 1.15$ (18 tests)

$P_m = 1.08$, $V_P = 0.23$ (see Table 62 of Reference 34)

Case 11. $0.024 \le t < 3/16$ in., double shear, with washers,

$F_u/F_y < 1.15$ (5 tests)

$P_m = 0.97$, $V_P = 0.07$ (see Table 63 of Reference 34)

Case 12. $0.024 \le t < 3/16$ in., single shear, with washers,

$F_u/F_y \geq 1.15$ (24 tests)

$P_m = 1.02$, $V_P = 0.20$ (see Table 64 of Reference 34)

Case 13. $0.024 \leq t < 3/16$ in., single shear, with washers,

$F_u/F_y < 1.15$ (16 tests)

$P_m = 1.05$, $V_P = 0.13$ (see Table 65 of Reference 34)

Case 14. $0.036 \leq t < 3/16$ in., single shear, without washers,

$F_u/F_y \geq 1.15$ (13 tests)

$P_m = 1.01$, $V_P = 0.04$ (see Table 66 of Reference 34)

Case 15. $0.036 \leq t < 3/16$ in., double shear, without washers,

$F_u/F_y \geq 1.15$ (8 tests)

$P_m = 0.93$, $V_P = 0.05$ (see Table 67 of Reference 34)

Based on all these values, the safety indices were computed and summarized in Table XXIV. By using different ϕ factors for different cases, the values of β vary from 3.43 to 4.06, which are satisfactory when compared to the target β of 3.5.

(4) For the calibration of shear stress on A307 bolts, the mean shear resistance of a bolt can be written in the following form:

$$R_m = (\tau_f/\sigma_f)_m(\sigma_f/F_u)_m(A_{SA}F_u) \quad (5.176)$$

in which τ_f is the actual ultimate shear stress, σ_f the actual ultimate tensile stress and F_u the nominal ultimate tensile stress of the bolt material. The term A_{SA} represents the stress area equal to the shank area if the shear plane passes through the shank, and it is the root area if the shear plane passes through the threads.

The coefficient of variation of the resistance, V_R, contains three parameters, V_M, V_F and V_P as shown below:

$$V_R = \sqrt{V_M^2+V_F^2+V_P^2} \quad (5.177)$$

In view of the fact that a combination of the coefficient of variation of the bolt material properties, V_M, and the design assumptions, V_P, can be considered to be

$$\sqrt{V_M{}^2+V_P{}^2} = \sqrt{V^2_{\tau_f/\sigma_f}+V^2_{\sigma_f/F_u}} \tag{5.178}$$

the value of V_R can be computed as follows:

$$V_R = \sqrt{V^2_{\tau_f/\sigma_f}+V^2_{\sigma_f/F_u}+0.05^2} \tag{5.179}$$

In the above equation, the value of V_F is assumed to be 0.05 to reflect the tolerance of the cross-sectional area of the bolt.

The following statistical data were computed on the basis of the test data provided in References 131, 132, 138 and 139 for bolted connection tests.

Case 16. Double shear, with washers, 3/8 in. diameter (11 tests)

$(\tau_f/\sigma_f)_m = 0.68$, $V = 0.11$

$(\sigma_f/F_u)_m = 1.28$, $V = 0.08$

Case 17. Double shear, with washers, 3/4 in. diameter (8 tests)

$(\tau_f/\sigma_f)_m = 0.60$, $V = 0.10$

$(\sigma_f/F_u)_m = 1.13$, $V = 0.08$

Case 18. Single shear, with washers, 3/8 in. diameter (19 tests)

$(\tau_f/\sigma_f)_m = 0.75$, $V = 0.10$

$(\sigma_f/F_u)_m = 1.28$, $V = 0.08$

Case 19. Single shear, with washers, 1/2 in. diameter (11 tests)

$(\tau_f/\sigma_f)_m = 0.63$, $V = 0.06$

$(\sigma_f/F_u)_m = 1.36$, $V = 0.08$

Case 20. Single shear, with washers, 3/4 in. diameter (14 tests)

$(\tau_f/\sigma_f)_m = 0.76$, $V = 0.06$

$(\sigma_f/F_u)_m = 1.13$, $V = 0.08$

Based on all these values, the safety indices were computed and summarized in Table XXIV. By using $\phi = 0.65$ for different cases, the values of β vary from 3.85 to 5.23 which are larger than the target β of 3.5.

Table XXIV

Computed Safety Index β for Bolted Connections

Case	M_m	V_M	F_m	V_F	P_m	V_P	Φ	β
Minimum Spacing and Edge Distance								
1	1.10	0.08	1.00	0.05	1.13	0.12	0.70	3.75
2	1.10	0.08	1.00	0.05	1.18	0.14	0.70	3.84
3	1.10	0.08	1.00	0.05	0.84	0.05	0.60	3.61
4	1.10	0.08	1.00	0.05	0.94	0.09	0.60	3.90
5	1.10	0.08	1.00	0.05	1.06	0.11	0.70	3.62
6	1.10	0.08	1.00	0.05	1.14	0.19	0.60	3.87
Tension Stress on Net Section								
7	1.10	0.08	1.00	0.05	1.14	0.20	0.65	3.53
8	1.10	0.08	1.00	0.05	0.95	0.21	0.55	3.41
9	1.10	0.08	1.00	0.05	1.04	0.14	0.65	3.63
Bearing Stress on Bolted Connections								
10	1.10	0.08	1.00	0.05	1.08	0.23	0.55	3.65
11	1.10	0.08	1.00	0.05	0.97	0.07	0.65	3.80
12	1.10	0.08	1.00	0.05	1.02	0.20	0.60	3.43
13	1.10	0.08	1.00	0.05	1.05	0.13	0.60	4.06
14	1.10	0.08	1.00	0.05	1.01	0.04	0.70	3.71

Table XXIV (Continued)

Case	M_m	V_M	F_m	V_F	P_m	V_P	Φ	β
15	1.10	0.08	1.00	0.05	0.93	0.05	0.65	3.70
Shear Strength on A307 Bolts								
16	1.28	0.08	1.00	0.05	0.68	0.11	0.65	4.73
17	1.13	0.08	1.00	0.05	0.60	0.10	0.65	3.85
18	1.28	0.08	1.00	0.05	0.75	0.10	0.65	5.23
19	1.36	0.08	1.00	0.05	0.63	0.06	0.65	4.49
20	1.13	0.08	1.00	0.05	0.76	0.06	0.65	5.09

Note: Case 1 = Single shear, with washers, $F_u/F_y \geq 1.15$

Case 2 = Double shear, with washers, $F_u/F_y \geq 1.15$

Case 3 = Single shear, with washers, $F_u/F_y < 1.15$

Case 4 = Double shear, with washers, $F_u/F_y < 1.15$

Case 5 = Single shear, without washers, $F_u/F_y \geq 1.15$

Case 6 = Single shear, without washers, $F_u/F_y < 1.15$

Case 7 = $t < 3/16$ in., double shear, with washers

Case 8 = $t < 3/16$ in., single shear, with washers

Case 9 = $t < 3/16$ in., single shear, without washers

Case 10 = $0.024 \leq t < 3/16$ in., double shear, with washers, $F_u/F_y \geq 1.15$

Case 11 = $0.024 \leq t < 3/16$ in., double shear, with washers, $F_u/F_y < 1.15$

Case 12 = $0.024 \leq t < 3/16$ in., single shear, with washers,

$F_u/F_y \geq 1.15$

Case 13 = $0.024 \leq t < 3/16$ in., single shear, with washers,

$F_u/F_y < 1.15$

Case 14 = $0.036 \leq t < 3/16$ in., single shear, without washers,

$F_u/F_y \geq 1.15$

Case 15 = $0.036 \leq t < 3/16$ in., double shear, without washers,

$F_u/F_y \geq 1.15$

Case 16 = Double shear, with washers, 3/8 in. diameter

Case 17 = Double shear, with washers, 3/4 in. diameter

Case 18 = Single shear, with washers, 3/8 in. diameter

Case 19 = Single shear, with washers, 1/2 in. diameter

Case 20 = Single shear, with washers, 3/4 in. diameter

VI. EVALUATION PROCEDURE OF TESTS FOR DETERMINING STRUCTURAL PERFORMANCE

A. GENERAL

The determination of design strengths for various structural components has been discussed in great detail in Section V. However, it is limited to those components that calculation of their load-carrying capacity can be made in accordance with the design provisions. For those components that calculation of their load-carrying capacity can not be made in accordance with the design provisions, their structural performance could be established from tests. In order to determine the design strengths for those components from the test results, an evaluation procedure of tests compatible with the LRFD design criteria is needed. This section presents the evaluation procedure of tests which is based on the same approach as used to developed the LRFD design criteria, and is consistent with the test procedure included in the 1986 AISI Specification.

B. EUROPEAN CONVENTION FOR CONSTRUCTIONAL STEELWORK (ECCS) APPROACH FOR TEST EVALUATION

The E.C.C.S. approach for test evaluation[140] is probability based, it can be summarized as follows:

(1) For a series of tests involving different thicknesses, an empirical curve is fitted through a plot of test results versus thickness. This curve is determined by minimizing the errors between the tested and the predicted load-carrying capacities. This curve gives the predicted capacity, P_p, as a function of the

thickness, t.

(2) The mean, P_m, and the coefficient of variation, V_P, of the tested-to-predicted load ratios are then calculated for all the tests.

(3) The characteristic load, P_k, for a given thickness is determined as follows:

$$P_k = P_p(1-cV_p) \tag{6.1}$$

where c is a statistical number that depends on the number of tests conducted, the probability distribution of the test results, and the confidence level desired. A characteristic value obtained in this manner is then compared with the factored design load.

The E.C.C.S. approach is based on characteristic values instead of nominal values used in the LRFD approach, also, the tests involve different thicknesses instead of one thickness as considered in the test procedure of the 1986 AISI Specification. Therefore, the E.C.C.S. approach can not be used in the AISI LRFD criteria for cold-formed steel members.

C. LRFD APPROACH FOR THE EVALUATION OF TEST RESULTS

In the 1986 AISI Specification, the provisions on special tests for determining structural performance are mainly used for test specimens with identical nominal dimensions. The evaluation procedure of test results derived herein is for the same purpose, except that the evaluation procedure is reliability-based, instead of engineering judgement and long time experience.

In order to develop an evaluation procedure on special tests for determining structural performance that is compatible with the LRFD design criteria, Eqs. (4.14) and (4.24) should be used. Due consideration should also be given to the following factors:

(1) Target reliability indices, β_o

(2) Coefficient of variation of the load effect, V_Q

(3) Statistical data for materials and sectional properties, M_m, V_M, F_m, and V_F

(4) Modification of P_m and V_P to account for the influence due to the small number of tests

(5) Determination of the predicted capacity R_P

Regarding to the target reliability indices β_o used in the evaluation procedure for tests, it is believed that the same target reliability indices β_o used in the development of the LRFD design criteria of cold-formed steel should be used to achieve a consistent reliability for both components designed by calculation and tests.

For the coefficient of variation of the load effect, the same V_Q values should be used for the development of the LRFD design criteria, and the development of evaluation procedure for special tests, because the load factors and load combinations are the same for both cases.

Regarding to the statistical data for materials and sectional properties used for the evaluation procedure for test results, values of M_m, V_M, F_m, and V_F used in the development of the LRFD design criteria for cold-formed steel are appropriate for the evaluation procedure for special tests because those statistical data are based on a large number of test specimens and therefore are considered to be representative values.

For the purpose of modifying P_m and V_P to account for the influence due to the small number of tests, the following formulas are assumed:

$$P'_m = C_1 P_m \tag{6.2}$$

$$V_{P'} = C_2 V_P \tag{6.3}$$

where

P'_m = modified mean value of the tested-to-predicted load ratios

$V_{P'}$ = modified coefficient of variation of the tested-to-predicted load ratios

C_1 = correction factor for P_m

C_2 = correction factor for V_P

In order to determine C_1 and C_2, it was recommended in Reference 141 that the probability distribution for ln(P') could be assumed to be student t rather than normal. The student t distribution is shown in Figure 12. It can be seen that for degree of freedom $\nu = \infty$, the student t distribution is identical to the normal distribution. Also, the student t distribution has the same mean value as the normal distribution, and its variance is $\nu/(\nu-2)$ times the variance of normal distribution[41]. Therefore,

$$P'_m = P_m \tag{6.4}$$

$$V_{P'} \approx \sqrt{\nu/(\nu-2)}\ V_P \tag{6.5}$$

where

ν = degree of freedom

= n-1, and n is number of tests (6.6)

From Eqs. (6.4) and (6.5) it can be seen that the only modification needed to account for the influence due to the small number of tests is that V_P^2 in Eqs. (4.14) and (4.24) be replaced by $(n-1)V_P^2/(n-3)$. To study

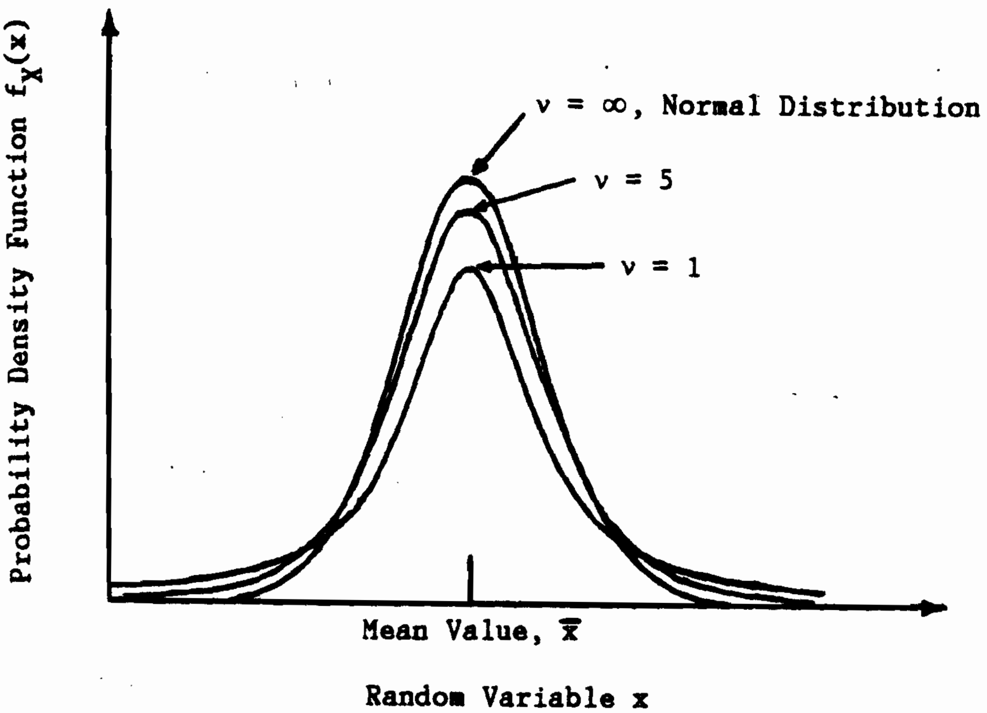

Figure 12 The Student t Distribution

the effect of the modification on the determination of ϕ factors, the following values are considered:

$M_m = 1.10, \quad V_M = 0.10$

$F_m = 1.00, \quad V_F = 0.05$

$P_m = 1.02, \quad V_P = 0.23$

$n = 4, \quad \beta_o = 2.5$

By using Eq. (4.14), it was found that (1) without a correction factor ($n = \infty$), $\phi = 0.75$; (2) with a correction factor ($n = 4$), $\phi = 0.55$. It can be seen that the larger the n value the larger the ϕ factor. This result is reasonable and appropriate because when more test data are used in the calibration the more reliable result can be obtained as expected. It should be noted that the number of tests, n, can not be less than four, otherwise, Eq. (6.5) is not applicable.

In the determination of tested-to-predicted load ratio, both tested and predicted loads are needed. In view of the fact that for performance tests, the average value of test data plays the same role as the nominal resistance equation given in the design criteria, such an average value of test data can be used as the predicted load in the determination of P_m and V_P.

Based on all these considerations, the following evaluation procedure of test results for determining structural performance is recommended in the LRFD criteria for cold-formed steel:

(a) Where practicable, evaluation of the test results shall be made on the basis of the average value of test data resulting from tests of not fewer than four identical specimens, provided the deviation of any individual test result from the average value obtained from all tests does not exceed ±10 percent. If such deviation from the average value exceeds 10 percent, at least three more tests of the same kind shall be made. The average value of all tests made shall then be regarded as the predicted capacity, R_P, for the series of the tests. The mean value, and the coefficient of variation, of the tested-to-predicted load ratios for all tests, P_m and V_P, shall be determined for statistical analysis.

(b) The load-carrying capacity of the tested elements, assemblies, connections, or members shall satisfy Eq. (6.7)

$$\phi R_P \geq \Sigma\gamma_i Q_i \tag{6.7}$$

where

$\Sigma\gamma_i Q_i$ = required resistance based on the most critical load combination. γ_i and Q_i are load factors and load effects, respectively.

R_P = average value of all test results

ϕ = resistance factor

$$= 1.5(M_m F_m P_m)\exp(-\beta_o\sqrt{V_M^2+V_F^2+C_P V_P^2+V_Q^2}) \tag{6.8}$$

M_m = mean value of the material factor listed in Table XXV for the type of component involved

F_m = mean value of the fabrication factor listed in Table XXV for the type of component involved

P_m = mean value of the tested-to-predicted load ratios

β_o = target reliability index

= 2.5 for structural members and 3.5 for connections

V_M = coefficient of variation of the material factor listed in Table XXV for the type of component involved

Table XXV

Statistical Data for the Determination of Resistance Factor

Type of Component	M_m	V_M	F_m	V_F
Transverse Stiffeners	1.10	0.10	1.00	0.05
Shear Stiffeners	1.00	0.06	1.00	0.05
Tension Members	1.10	0.10	1.00	0.05
Flexural Members				
Bending Strength	1.10	0.10	1.00	0.05
Lateral Buckling Strength	1.00	0.06	1.00	0.05
One Flange Through-Fastened to Deck or Sheathing	1.10	0.10	1.00	0.05
Shear Strength	1.10	0.10	1.00	0.05
Combined Bending and Shear	1.10	0.10	1.00	0.05
Web Crippling Strength	1.10	0.10	1.00	0.05
Combined Bending and Web Crippling	1.10	0.10	1.00	0.05
Concentrically Loaded Compression Members	1.10	0.10	1.00	0.05
Combined Axial Load and Bending	1.05	0.10	1.00	0.05
Cylindrical Tubular Members				
Bending Strength	1.10	0.10	1.00	0.05
Axial Compression	1.10	0.10	1.00	0.05
Wall Studs and Wall Stud Assemblies				
Wall Studs in Compression	1.10	0.10	1.00	0.05
Wall Studs in Bending	1.10	0.10	1.00	0.05
Wall Studs with Combined Axial Load and Bending	1.05	0.10	1.00	0.05

Table XXV (Continued)

Type of Component	M_m	V_M	F_m	V_F
Welded Connections				
Arc Spot Welds				
Shear Strength of Welds	1.10	0.10	1.00	0.10
Plate Failure	1.10	0.08	1.00	0.15
Tensile Strength	1.10	0.08	1.00	0.15
Arc Seam Welds				
Shear Strength of Welds	1.10	0.10	1.00	0.10
Plate Tearing	1.10	0.10	1.00	0.10
Fillet Welds				
Shear Strength of Welds	1.10	0.10	1.00	0.10
Plate Failure	1.10	0.08	1.00	0.15
Flare Groove Welds				
Shear Strength of Welds	1.10	0.10	1.00	0.10
Plate Failure	1.10	0.10	1.00	0.10
Resistance Welds	1.10	0.10	1.00	0.10
Bolted Connections				
Minimum Spacing and Edge Distance	1.10	0.08	1.00	0.05
Tension Strength on Net Section	1.10	0.08	1.00	0.05
Bearing Strength	1.10	0.08	1.00	0.05

V_F = coefficient of variation of the fabrication factor listed in Table XXV for the type of component involved

C_P = correction factor

= (n-1)/(n-3) (6.9)

V_P = coefficient of variation of the tested-to-predicted load ratios

n = number of tests

V_Q = coefficient of variation of the load effect

= 0.21

Exception: For beams having tension flange through-fastened to deck or sheathing and with compression flange laterally unbraced, ϕ shall be determined with a coefficient of 1.6 in lieu of 1.5 in Eq. (6.8), β_o = 1.5, and V_Q = 0.43.

The listing in Table XXV does not exclude the use of other documented statistical data if they are established from sufficient results on material properties and fabrication.

When distortions interfere with the proper functioning of the specimen, the load effects based on the critical load combination at the occurence of the acceptable distortion shall also satisfy Eq. (6.7), except that the resistance factor ϕ is taken as unity and that the load factor for dead load may be taken as 1.0.

D. DETERMINATION OF SAFETY FACTOR FOR ASD CRITERIA

The evaluation procedure mentioned above can also be used for the determination of the safety factor needed in the Allowable Stress Design criteria. For a general case of dead and live load combination, Eqs.

(6.10) and (6.11) give the values of R_n according to LRFD and ASD criteria, respectively.

LRFD criteria:

$$R_n = c(1.2D_n+1.6L_n)/\phi = 1.84cL_n/\phi \tag{6.10}$$

ASD criteria:

$$R_n = c(D_n+L_n)(FS) = 1.2cL_n(FS) \tag{6.11}$$

From Eqs. (6.10) and (6.11), the safety factor needed for the ASD criteria to achieve the same reliability index as the LRFD criteria is

$$FS = 1.84cL_n/[\phi(1.2cL_n)] = 1.84/(1.2\phi) = 1.53/\phi \tag{6.12}$$

For counteracting loads, the procedure used for determining the safety factor is similar as the dead and live load combination as follows:

LRFD criteria:

$$R_n = c(1.17W_n-0.9D_n)/\phi = 1.08cW_n/\phi \tag{6.13}$$

ASD criteria:

$$R_n = c(W_n-D_n)(0.75)(FS) = 0.675cW_n(FS) \tag{6.14}$$

Therefore,

$$FS = 1.08cW_n/[\phi(0.675cW_n)] = 1.6/\phi \tag{6.15}$$

Because the safety factors determined in Eqs. (6.12), and (6.15) are reliability-based, these safety factors are more reliable than those based on the engineering judgement and long time experience.

VII. COMPARATIVE STUDY OF DESIGN METHODS FOR COLD-FORMED STEEL

A. GENERAL

The primary purpose of this section is to study, and compare, the Load and Resistance Factor Design (LRFD) criteria for cold-formed steel members with the existing Allowable Stress Design (ASD) criteria included in the 1986 Specification for the design of cold-formed steel structural members. This comparison involves studies of different variables used for the design of various types of structural members and discussions of different load-carrying capacities determined by these two methods.

This study compares the existing Allowable Stress Design method, with the Load and Resistance Factor Design method, for cold-formed steel structural members generally used in building construction. These shapes include channels with stiffened or unstiffened flanges, I-sections made from channels, and hat sections with unreinforced webs. The yield points of steel range from 33 to 50 ksi.

The AISI Specification and the LRFD criteria for cold-formed steel can be used for the design of tension members, flexural members, compression members, members subjected to a combination of bending and axial loads, bolted connections, welded connections, stiffeners, and wall studs. Even though the allowable stress design provisions and the LRFD criteria were prepared for any combinations of different loads, only dead and live loads are used in this comparison, for each type of structural members, with the exception that wind and dead loads are used for members subjected to uplift loading condition. Ratios of load-carrying capacities are computed and evaluated for different shapes of structural members which are used in typical design situations.

B. ALLOWABLE LOAD DETERMINED FROM THE LRFD CRITERIA

In the LRFD criteria, the design strength of any structural component is ϕR_n. 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:

$$\phi R_n \geq c(1.2D_n+1.6L_n) \tag{7.1}$$

$$\phi R_n \geq c(1.2D_n/L_n+1.6)L_n \tag{7.2}$$

$$\phi R_n \geq c(1.2D_n/L_n+1.6)[(D_n+L_n)/(D_n/L_n+1)] \tag{7.3}$$

Therefore,

$$c(D_n+L_n) \leq \frac{R_n}{(1.2D_n/L_n+1.6)/[\phi(D_n/L_n+1)]} \tag{7.4}$$

where c is the deterministic influence coefficient to transform the load to load effect.

From Eq. (7.4), the factor of safety against the nominal resistance used in the LRFD is:

$$(F.S.)_{LRFD} = (1.2D_n/L_n+1.6)/[\phi(D_n/L_n+1)] \tag{7.5}$$

For the counteracting loads applied to individual purlins, girts, wall panels and roof decks, the unfactored load combination (W_n-D_n) or allowable load can be computed from the nominal resistance R_n, the resistance factor ϕ, and a given D_n/W_n ratio as follows:

$$\phi R_n \geq c(1.17W_n-0.9D_n) \tag{7.6}$$

$$\phi R_n \geq c(1.17-0.9D_n/W_n)W_n \tag{7.7}$$

$$\phi R_n \geq c(1.17-0.9D_n/W_n)[(W_n-D_n)/(1-D_n/W_n)] \tag{7.8}$$

Therefore,

$$c(W_n-D_n) \le \frac{R_n}{(1.17-0.9D_n/W_n)/[\phi(1-D_n/W_n)]} \quad (7.9)$$

From Eq. (7.9) with the consideration of 1/3 strength increase for members subjected to wind load, the factor of safety against the nominal resistance used in the LRFD is:

$$(F.S.)_{LRFD} = 0.75(1.17-0.9D_n/W_n)/[\phi(1-D_n/W_n)] \quad (7.10)$$

Eqs. (7.4) and (7.9) are used in this study to compare the AISI Specification for allowable stress design and the Load and Resistance Factor Design criteria.

C. COMPARATIVE STUDY OF TENSION MEMBERS

For a comparison between the allowable stress design and the LRFD approach, the unfactored load can be calculated by using the following equation for both design methods:

$$P_T = P_{DL}+P_{LL} \quad (7.11)$$

where

P_T = total unfactored load applied to the member

P_{DL}= axial tension due to the nominal dead load

P_{LL}= axial tension due to the nominal live load

This total unfactored load should be less than or equal to the allowable load. For allowable stress design, the allowable load is

$$(P_a)_{ASD} = A_nF_y/\Omega_t = A_nF_y/1.67 \quad (7.12)$$

For LRFD, the allowable load can be calculated by using Eq. (7.4), i.e.,

$$(P_a)_{LRFD} = \phi_tT_n(D/L+1)/(1.2D/L+1.6) \quad (7.13)$$

Because $T_n = A_nF_y$, Eq. (7.13) can be rewritten as

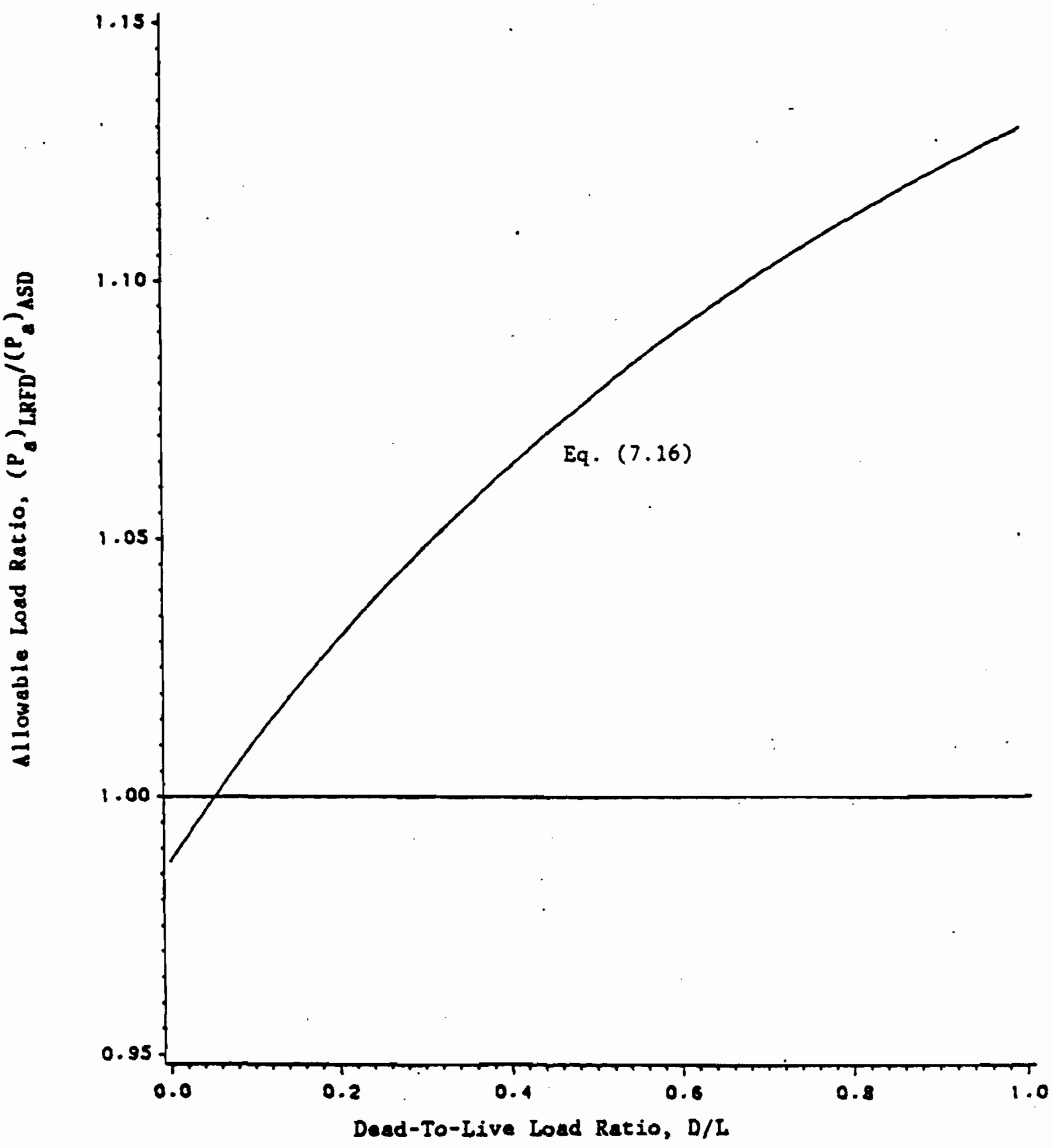

Figure 13 Allowable Load Ratio vs. D/L Ratio for Tension Members

$$(P_a)_{LRFD} = \phi_t A_n F_y (D/L+1)/(1.2D/L+1.6) \qquad (7.14)$$

where D/L is the ratio of the nominal dead load to the nominal live load. From Eq. (7.14) it is clear that the allowable load based on LRFD is a function of not only cross-sectional area and yield strength of the steel but also the dead-to-live load ratio. This will be true for all structural members designed by LRFD method.

Therefore, based on Eqs. (7.12) and (7.14), the allowable load ratio for tension members is

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.67\phi_t \frac{D/L+1}{1.2D/L+1.6} \qquad (7.15)$$

For the value of $\phi_t = 0.95$

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.58 \frac{D/L+1}{1.2D/L+1.6} \qquad (7.16)$$

Figure 13 shows the allowable load ratio versus dead-to-live load ratio. When $D/L < 1/25$, the allowable load determined by the LRFD method is slightly less than that determined by the allowable stress design. For D/L = 1/5, ASD is about 3.2% conservative compared to LRFD.

D. COMPARATIVE STUDY OF FLEXURAL MEMBERS

1. Strength for Bending Only. The unfactored moment can be calculated by using Eq. (7.17) for both methods (ASD and LRFD).

$$M_{TL} = M_{DL} + M_{LL} \qquad (7.17)$$

where

M_{TL} = total unfactored moment

M_{DL} = moment due to the nominal dead load

M_{LL} = moment due to the nominal live load

For allowable stress design, the allowable moment is determined from either nominal section strength, or lateral buckling strength, with a factor of safety of 1.67. Therefore, the allowable moment for beams is

$$(M_a)_{ASD} = M_n/\Omega_f = M_n/1.67 \tag{7.18}$$

For LRFD, the allowable moment can be computed by using the following equation developed from Eq. (7.4).

$$(M_a)_{LRFD} = \phi_b M_n(D/L+1)/(1.2D/L+1.6) \tag{7.19}$$

The ratio of the allowable moments for both nominal section strength and lateral buckling strength is

$$\frac{(M_a)_{LRFD}}{(M_a)_{ASD}} = 1.67\phi_b \frac{D/L+1}{1.2D/L+1.6} \tag{7.20}$$

For nominal section strength of sections with stiffened or partially stiffened compression flanges, $\phi_b = 0.95$

$$\frac{(M_a)_{LRFD}}{(M_a)_{ASD}} = 1.58 \frac{D/L+1}{1.2D/L+1.6} \tag{7.21}$$

The solid curve in Figure 14 shows the allowable moment ratio versus dead-to-live load ratio for beams with stiffened or partially stiffened compression flanges based on the nominal section strength. For D/L = 1/25 both design methods will give the same value of allowable moment. However, LRFD will be conservative for D/L < 1/25 and unconservative for D/L > 1/25 as compared with the allowable stress design method.

For nominal section strength of sections with unstiffened compression flanges and lateral buckling strength, $\phi_b = 0.90$

$$\frac{(M_a)_{LRFD}}{(M_a)_{ASD}} = 1.50 \frac{D/L+1}{1.2D/L+1.6} \tag{7.22}$$

The dotted curve in Figure 14 shows the allowable moment ratio versus dead-to-live load ratio for this case. Both design methods will give the same value for D/L = 1/3. For D/L = 0.5, the allowable moment based on LRFD is about 2.3% larger than the value obtained from allowable stress design. When the dead-to-live load ratio is less than 1/3, the LRFD criteria are found to be conservative for both nominal section strength of sections with unstiffened compression flanges and lateral buckling, as compared with the allowable stress design method.

For C- or Z-section with the tension flange attached to deck or sheathing and with compression flange laterally unbraced, i.e., member subjected to counteracting loads, a different approach for comparison is used. The required nominal moment for ASD criteria, $(M_n)_{ASD}$, is determined from the applied loads with a factor of safety of 1.67 and a strength increase of 1/3. Therefore,

$$(M_n)_{ASD} = 0.75\Omega_f(W-D)c = 1.25cW(1-D/W) \tag{7.23}$$

For LRFD, the required nominal moment, $(M_n)_{LRFD}$, can be computed by using the following equation:

$$(M_n)_{LRFD} = (c/\phi_b)(1.17W-0.9D) = (c/\phi_b)(1.17-0.9D/W)W \tag{7.24}$$

With $\phi_b = 0.90$, the ratio of the required nominal moments is

$$\frac{(M_n)_{ASD}}{(M_n)_{LRFD}} = 1.25\phi_b \frac{1-D/W}{1.17-0.9D/W} = 1.125 \frac{1-D/W}{1.17-0.9D/W} \tag{7.25}$$

If the allowable moment ratio is needed, Eq. (7.26) can be used:

$$\frac{(M_a)_{LRFD}}{(M_a)_{ASD}} = \frac{(M_n)_{ASD}}{(M_n)_{LRFD}} \tag{7.26}$$

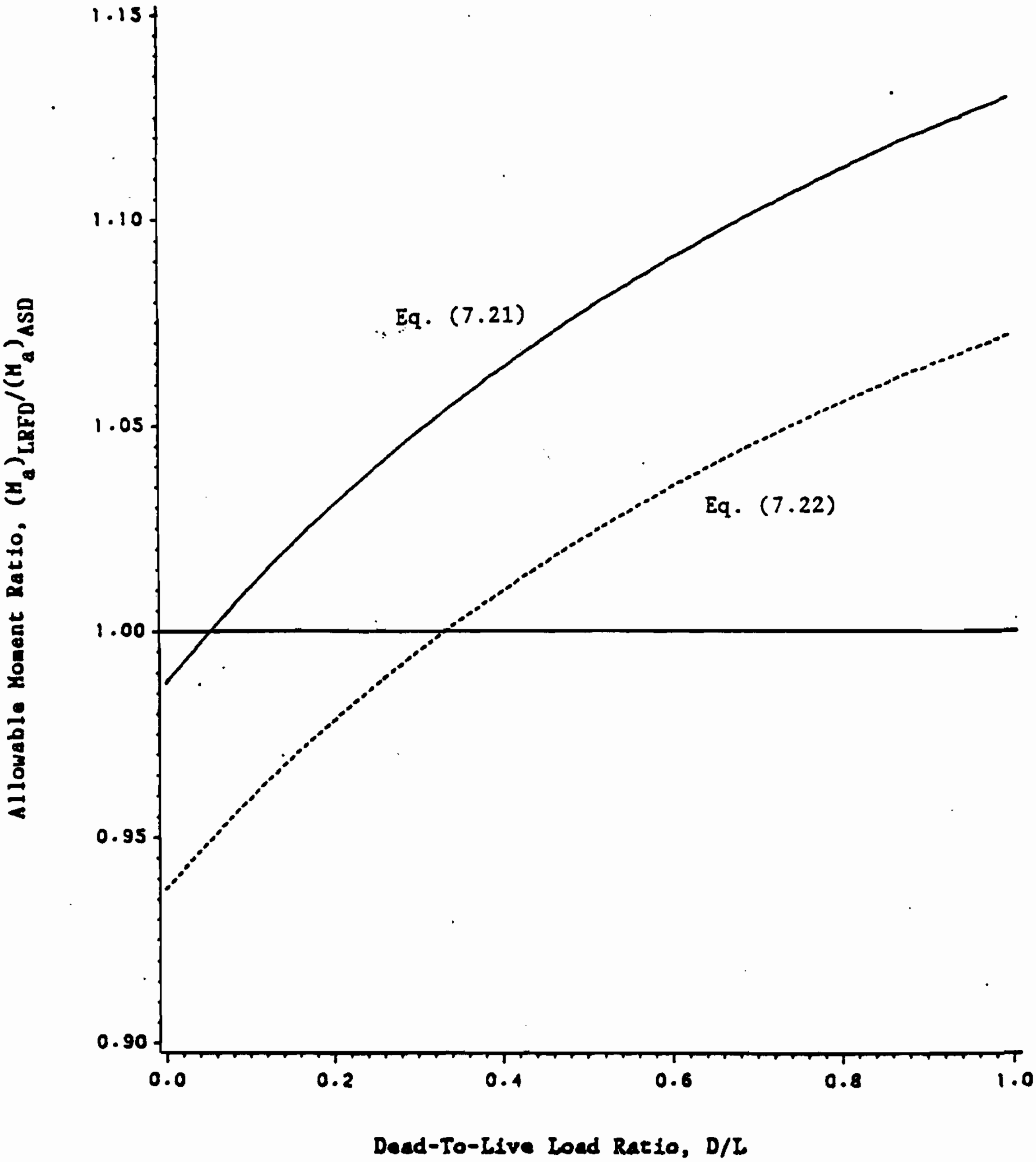

Figure 14 Allowable Moment Ratio vs. D/L Ratio for Bending Strength of Beams

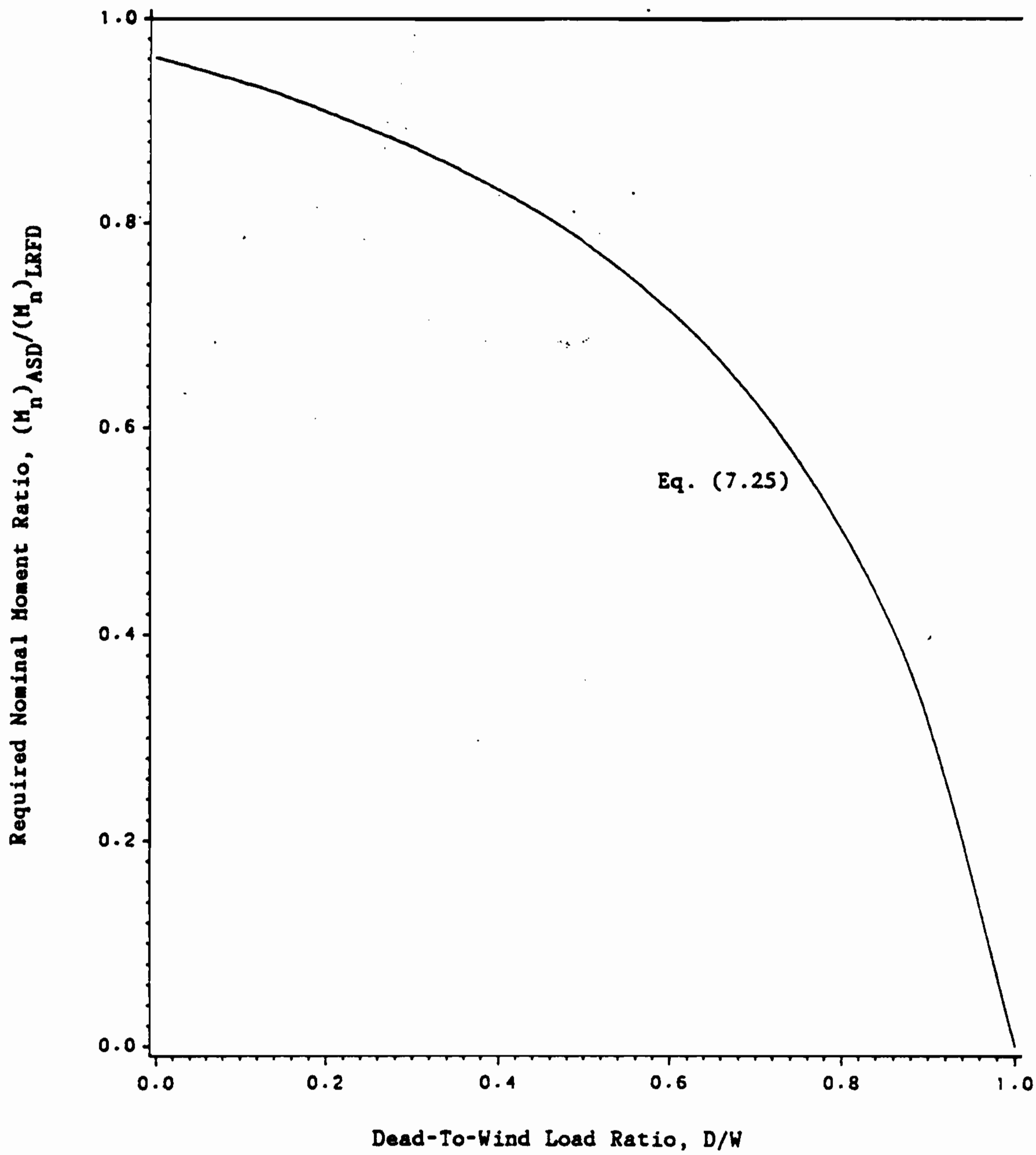

Figure 15 **Required Nominal Moment Ratio vs. D/W Ratio for C- or Z-Section With the Tension Flange Attached to Deck or Sheathing and With Compression Flange Laterally Unbraced**

Figure 15 shows the required nominal moment ratio versus dead-to-wind load ratio for this case. As shown in the figure, the LRFD criteria for C- or Z-section with the tension flange attached to deck or sheathing and with compression flange laterally unbraced are slightly conservative for the values of D/W ratios generally used in cold-formed steel construction. For D/W = 0.1, the required nominal moment based on ASD is about 6.2% lower than the value obtained from LRFD criteria.

2. Strength for Shear Only. The unfactored shear force can be calculated for both ASD and LRFD methods by using the following equation.

$$V_T = V_{DL} + V_{LL} \tag{7.27}$$

where

V_T = total unfactored shear force

V_{DL} = shear force due to the nominal dead load

V_{LL} = shear force due to the nominal live load

This total unfactored shear force should be less than or equal to the allowable shear capacity. For allowable stress design, the allowable shear load for beam webs is

$$(V_a)_{ASD} = V_n/\Omega \tag{7.28}$$

For LRFD, the allowable shear load equation was developed from Eq. (7.4) and is

$$(V_a)_{LRFD} = \phi_v V_n (D/L+1)/(1.2D/L+1.6) \tag{7.29}$$

The allowable shear force, V_a, for allowable stress design is determined from shear yielding with a factor of safety of 1.44, from the critical stress for elastic shear buckling with a factor of safety of 1.71, and from the critical stress for inelastic shear buckling with a factor of safety of 1.67. The limits of the h/t ratio were obtained by equating the formulas for the three shear failure modes for both allowable stress and LRFD criteria. Because each failure mode has a different factor of safety, the h/t limits are slightly different for both design criteria. The allowable shear ratios are:

For $h/t \le \sqrt{Ek_v/F_y}$ and $\phi_v = 1.0$,

$$\frac{(V_a)_{LRFD}}{(V_a)_{ASD}} = 1.443\phi_v \frac{D/L+1}{1.2D/L+1.6} = 1.443 \frac{D/L+1}{1.2D/L+1.6} \qquad (7.30)$$

For $\sqrt{Ek_v/F_y} < h/t \le 1.38\sqrt{Ek_v/F_y}$ and $\phi_v = 0.90$

$$\frac{(V_a)_{LRFD}}{(V_a)_{ASD}} = 1.674\phi_v \frac{D/L+1}{1.2D/L+1.6} = 1.507 \frac{D/L+1}{1.2D/L+1.6} \qquad (7.31)$$

For $h/t > 1.415\sqrt{Ek_v/F_y}$ and $\phi_v = 0.90$

$$\frac{(V_a)_{LRFD}}{(V_a)_{ASD}} = 1.712\phi_v \frac{D/L+1}{1.2D/L+1.6} = 1.541 \frac{D/L+1}{1.2D/L+1.6} \qquad (7.32)$$

It should be noted that for h/t greater than $1.38\sqrt{Ek_v/F_y}$ and less than $1.415\sqrt{Ek_v/F_y}$, inelastic shear buckling will govern for LRFD.

Figure 16 shows the allowable shear ratio versus dead-to-live load ratio for the three failure modes. For D/L = 0.5, the allowable shear determined according to LRFD may be up to 5% higher than the value obtained from allowable stress design. For D/L < 0.17, LRFD is generally conservative. When D/L > 0.65, LRFD gives larger values of the allowable shear capacity.

In Figure 17, the relationships of allowable shear ratio and h/t ratio are shown graphically for dead-to-live load ratios equal to 1/5, 1/3, and 1/2. The transition zones between h/t limits can be seen clearly in this figure.

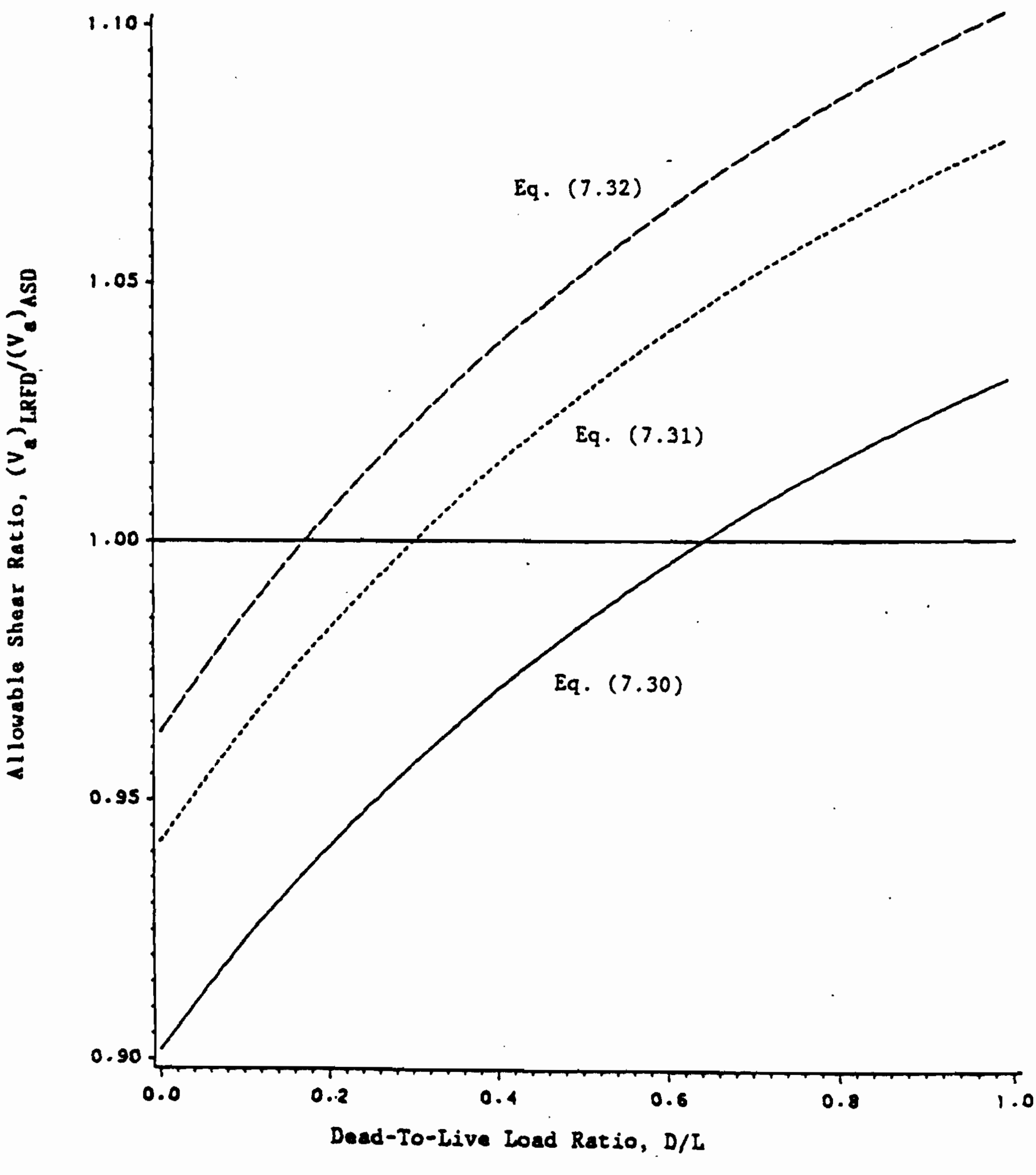

Figure 16 Allowable Shear Ratio vs. D/L Ratio for Shear Strength of Beams

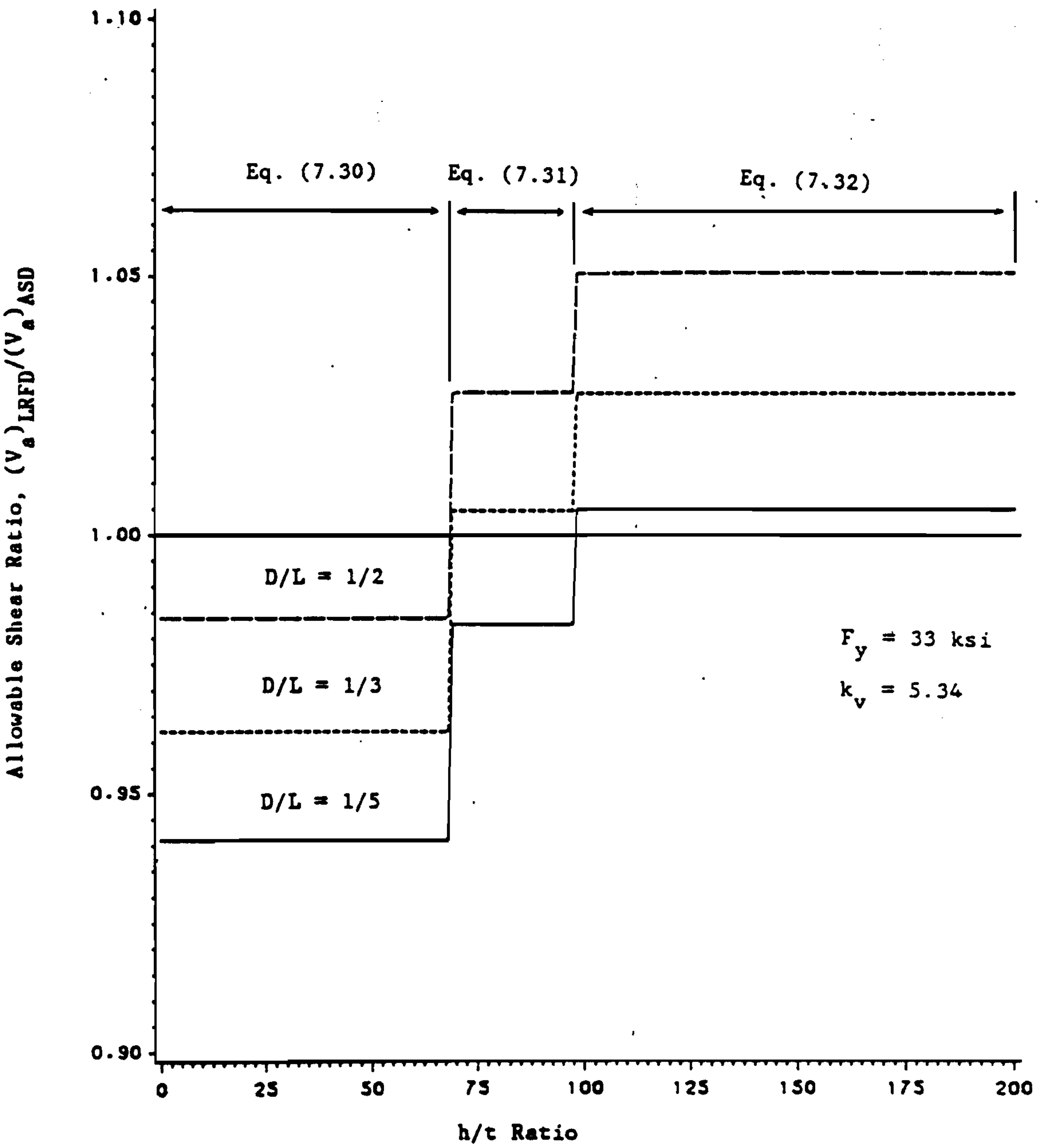

Figure 17 Allowable Shear Ratio vs. h/t Ratio for Shear Strength of Beams

3. Strength for Combined Bending and Shear. A typical design example was selected for comparison purposes. The example deals with a three-equal-span continuous beam subjected to a uniformly distributed dead and live load. The combination of the following maximum moment and shear would occur at the interior supports.

$$M_{TL} = M_{DL}+M_{LL} = c_m w_T L^2 \tag{7.33}$$

$$V_T = V_{DL}+V_{LL} = c_v w_T L \tag{7.34}$$

where c_m and c_v are the deterministic influence coefficients for applied moment and shear based on support conditions and number of spans and w_T is the unfactored applied uniform load.

The allowable load based on allowable stress design was calculated as follows:

$$\frac{M}{M_a} = \frac{M_{TL}}{0.6M_n} = \frac{1.667c_m w_T L^2}{M_n} \tag{7.35}$$

For $h/t \leq 1.38\sqrt{Ek_v/F_y}$,

$$\frac{V}{V_a} = \frac{V_T}{V_n/1.674} = \frac{1.674c_v w_T L}{V_n} \tag{7.36}$$

By using Eqs. (7.35) and (7.36), the following interaction formula can be obtained[32]:

$$\left(\frac{M}{M_a}\right)^2 + \left(\frac{V}{V_a}\right)^2 = w_T^2\left[\left(\frac{1.667c_m L^2}{M_n}\right)^2 + \left(\frac{1.674c_v L}{V_n}\right)^2\right] = 1 \tag{7.37}$$

Therefore,

$$(w_T)_{ASD} = \frac{1}{L\sqrt{\left(\frac{1.667c_m L}{M_n}\right)^2+\left(\frac{1.674c_v}{V_n}\right)^2}} \tag{7.38}$$

For $h/t > 1.415\sqrt{Ek_v/F_y}$,

$$\frac{V}{V_a} = \frac{V_T}{V_n/1.712} = \frac{1.712c_v w_T L}{V_n} \tag{7.39}$$

By using Eqs. (7.35) and (7.39), the following interaction formula can be obtained:

$$\left(\frac{M}{M_a}\right)^2 + \left(\frac{V}{V_a}\right)^2 = w_T^2\left[\left(\frac{1.667c_m L^2}{M_n}\right)^2 + \left(\frac{1.712c_v L}{V_n}\right)^2\right] = 1 \tag{7.40}$$

Therefore,

$$(w_T)_{ASD} = \frac{1}{L\sqrt{\left(\frac{1.667c_m L}{M_n}\right)^2 + \left(\frac{1.712c_v}{V_n}\right)^2}} \tag{7.41}$$

The allowable uniform load based on LRFD was calculated as follows:

$$\frac{M_u}{\phi_b M_n} = \frac{1.2D/L+1.6}{D/L+1}\left[\frac{M_{TL}}{\phi_b M_n}\right] = \frac{1.2D/L+1.6}{D/L+1}\left[\frac{c_m w_T L^2}{\phi_b M_n}\right] \tag{7.42}$$

$$\frac{V_u}{\phi_v V_n} = \frac{1.2D/L+1.6}{D/L+1}\left[\frac{V_T}{\phi_v V_n}\right] = \frac{1.2D/L+1.6}{D/L+1}\left[\frac{c_v w_T L}{\phi_v V_n}\right] \tag{7.43}$$

By using Eqs. (7.42) and (7.43), the following interaction formula can be obtained:

$$\left(\frac{M_u}{\phi_b M_n}\right)^2 + \left(\frac{V_u}{\phi_v V_n}\right)^2 = w_T^2\left(\frac{1.2D/L+1.6}{D/L+1}\right)^2\left[\left(\frac{c_m L^2}{\phi_b M_n}\right)^2 + \left(\frac{c_v L}{\phi_v V_n}\right)^2\right] = 1 \tag{7.44}$$

Therefore,

$$(w_T)_{LRFD} = \frac{D/L+1}{1.2D/L+1.6}\,\frac{1}{L\sqrt{\left(\frac{c_m L}{\phi_b M_n}\right)^2 + \left(\frac{c_v}{\phi_v V_n}\right)^2}} \tag{7.45}$$

For the design example used in this comparison, the coefficients, c_m and c_v, are equal to 0.10 and 0.60, respectively. Therefore, by using ϕ_b = 0.95 and 0.90 for sections with stiffened or partially stiffened compression flanges and unstiffened compression flanges, respectively, for nominal section strength and $\phi_v = 0.90$, the allowable uniform load ratios are as follows:

For $h/t \leq 1.38\sqrt{Ek_v/F_y}$,

$$\frac{(w_T)_{LRFD}}{(w_T)_{ASD}} = \frac{D/L+1}{1.2D/L+1.6}\sqrt{\frac{2.803+0.07716(V_nL/M_n)^2}{1.235+0.02778(V_nL/\phi_bM_n)^2}} \tag{7.46}$$

For $h/t > 1.415\sqrt{Ek_v/F_y}$,

$$\frac{(w_T)_{LRFD}}{(w_T)_{ASD}} = \frac{D/L+1}{1.2D/L+1.6}\sqrt{\frac{2.929+0.07716(V_nL/M_n)^2}{1.235+0.02778(V_nL/\phi_bM_n)^2}} \tag{7.47}$$

Eqs. (7.46) and (7.47) can be expressed in the following form:

$$\frac{(w_T)_{LRFD}}{(w_T)_{ASD}} = \frac{D/L+1}{1.2D/L+1.6}(K_w) \tag{7.48}$$

where K_w is a variable determined from section properties, material strength, and span length for a particular design example.

For combined bending and shear, the allowable load ratio can be determined by using Eq. (7.48) as given above. It is not only a function of dead-to-live load ratio but is also a function of h/t, cross sectional geometry, and material strength. Because of the complexity involved in the comparison, several individual beam sections of different depths and thicknesses were studied.

Figure 18 shows the allowable load ratio versus dead-to-live load ratio for 5 in. x 2 in. standard channel sections with stiffened flanges which are listed in Table 1 of Part V of the AISI Design Manual[142]. Different curves represent the relationships for different thicknesses by using the same span length and material. Table XXVI shows the sectional properties and calculated values used to obtain the curves which indicate that thinner members result in slightly lower values for the allowable load ratio except t = 0.048 in. which is governed by Eq. (7.47) because of the higher h/t ratio.

In Figure 19, the span length was varied for a 5 in. x 2 in. x 0.105 in. channel with stiffened flanges for D/L = 1/5 and F_y = 33 to 50 ksi. Span lengths and calculated values used to obtain the curves are included in Table XXVII. It can be seen that the material strength has little effect on the allowable uniform load ratio. This figure also shows that for the channel section used in this comparison, the allowable load permitted by LRFD is larger than that determined by ASD for span length larger than 20 in.

Figure 20 shows the allowable uniform load ratio versus h/t ratio for the 5 in. - deep channels used in Figure 18 and Table XXVI for a dead-to-live load ratio of 1/5 and a span length of 5 ft. Table XXVIII shows the calculated values for F_y = 50 ksi. For F_y = 33 and 50 ksi, this figure shows that the smallest allowable load ratio occurs at h/t = 75.

Figure 21 shows the relationship of allowable load ratio and dead-to-live load ratio for channels with stiffened flanges. Cross sectional properties and other related data are included in Table XXIX. Deeper sections with larger h/t ratios give smaller values of the allowable load ratio as indicated in Figure 21.

Channels with unstiffened flanges were also studied. The curves obtained for channels with unstiffened flanges are similar to those curves obtained for channels with stiffened flanges. However, the allowable load ratios computed for channels with unstiffened flanges are smaller, as compared with the allowable load ratios computed for channels with

Table XXVI

Channels With Stiffened Flanges, 5 in. Depths - Case A.

Section	h/t	V_n (Kips)	M_n (K-in.)	$\Phi_b M_n$ (K-in.)	K_w
5x2x0.135	32.26	26.594	61.803	58.712	1.5790
0.105	42.05	16.088	49.625	47.144	1.5761
0.075	62.17	8.208	36.917	35.071	1.5694
0.060	78.21	5.253	28.555	27.127	1.5646
0.048	98.26	3.343	21.795	20.705	1.5695

* F_y = 33 ksi, L = 60 in.

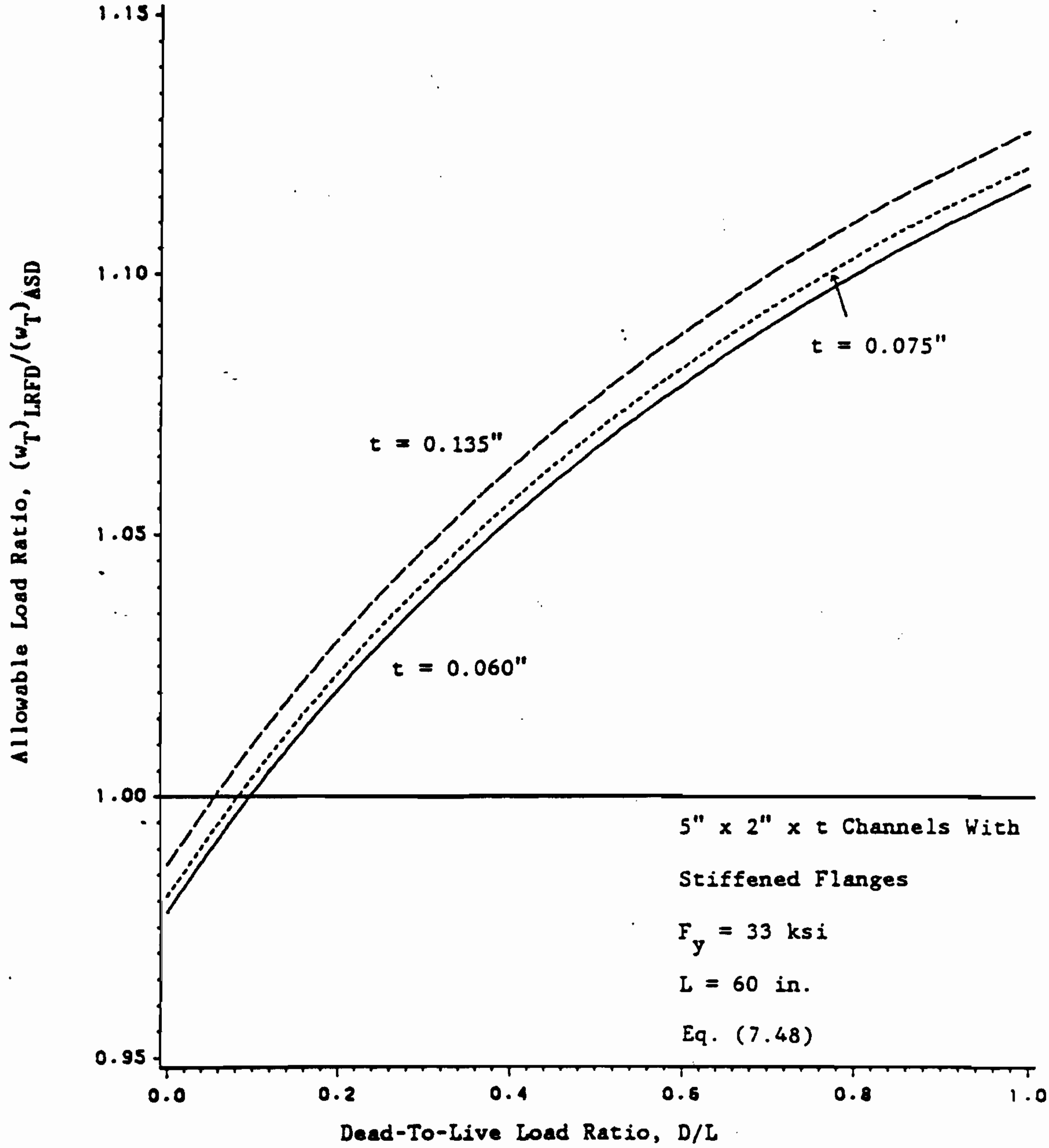

Figure 18 Allowable Load Ratio vs. D/L Ratio for Combined Bending and Shear in Beams - Case A

Table XXVII

5 in. x 2 in. x 0.105 in. Channels With Stiffened Flanges

for Various Lengths and Yield Points

F_y (ksi)	L (in.)	V_n (Kips)	M_n (K-in.)	$\Phi_b M_n$ (K-in.)	K_w
33	0	16.088	49.625	47.144	1.5065
	25	16.088	49.625	47.144	1.5546
	50	16.088	49.625	47.144	1.5733
	75	16.088	49.625	47.144	1.5785
	100	16.088	49.625	47.144	1.5805
50	0	19.803	75.190	71.430	1.5065
	25	19.803	75.190	71.430	1.5469
	50	19.803	75.190	71.430	1.5691
	75	19.803	75.190	71.430	1.5763
	100	19.803	75.190	71.430	1.5792

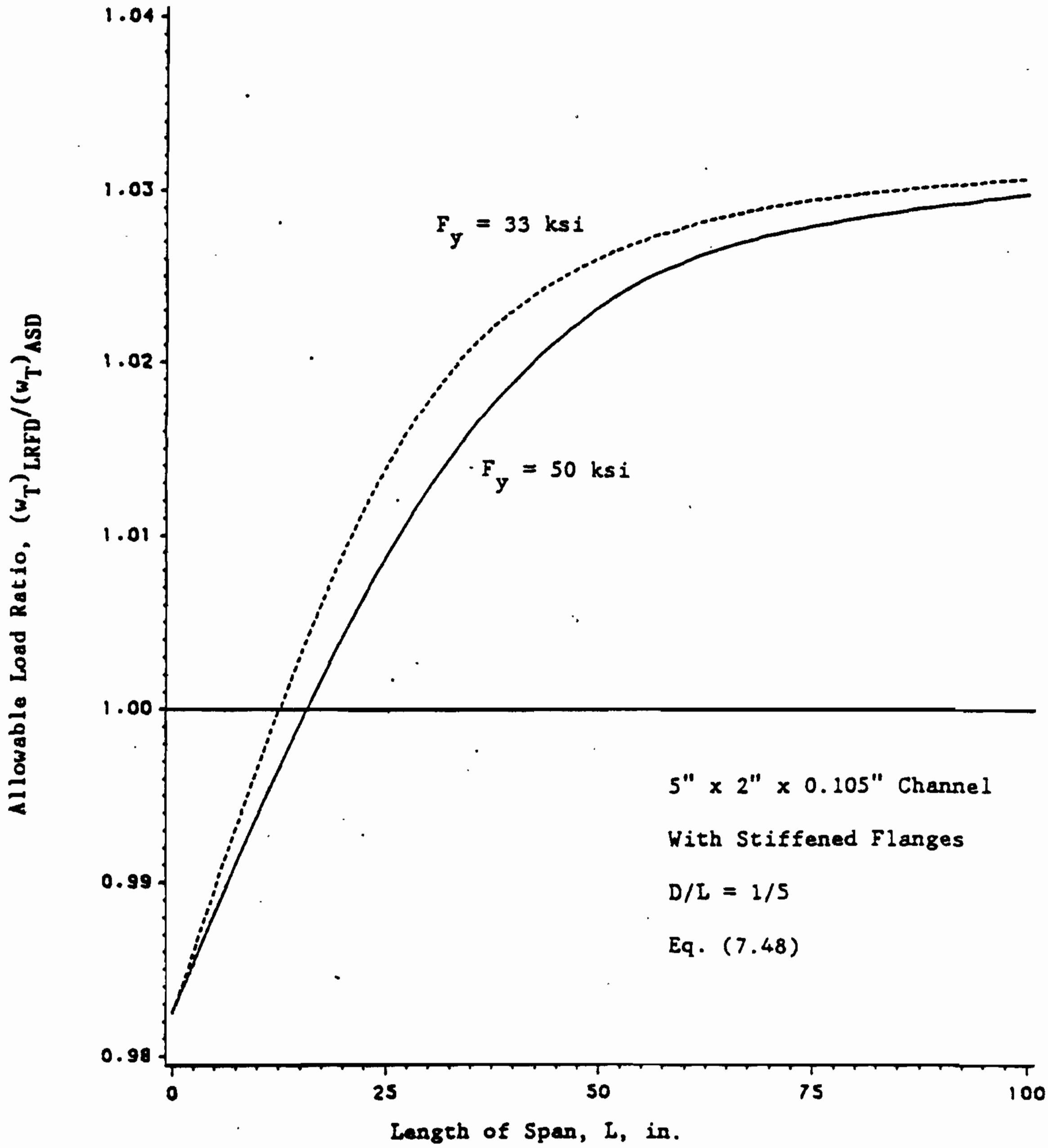

Figure 19 Allowable Load Ratio vs. Span Length - Case A

Table XXVIII

5 in. x 2 in. Channels With Stiffened Flanges for F_y = 50 ksi

Section	h/t	V_n (Kips)	M_n (K-in.)	$\phi_b M_n$ (K-in.)	K_w
5x2x0.135	32.26	32.735	93.640	88.958	1.5770
0.105	42.05	19.803	75.190	71.430	1.5729
0.075	62.17	10.103	54.626	51.895	1.5648
0.060	78.21	6.466	39.016	37.066	1.5615
0.048	98.26	3.343	30.687	29.153	1.5625

* L = 60 in.

Table XXIX

Channels With Stiffened Flanges - Case B.

Section	h/t	V_n (Kips)	M_n (K-in.)	$\phi_b M_n$ (K-in.)	K_w
9x3.25x0.105	80.14	16.088	152.534	144.907	1.5453
7x2.75x0.105	61.10	16.088	99.487	94.512	1.5607
5x2x0.105	42.05	16.088	49.625	47.144	1.5761
3.5x2x0.105	27.76	16.088	30.531	29.005	1.5804

* F_y = 33 ksi, L = 60 in.

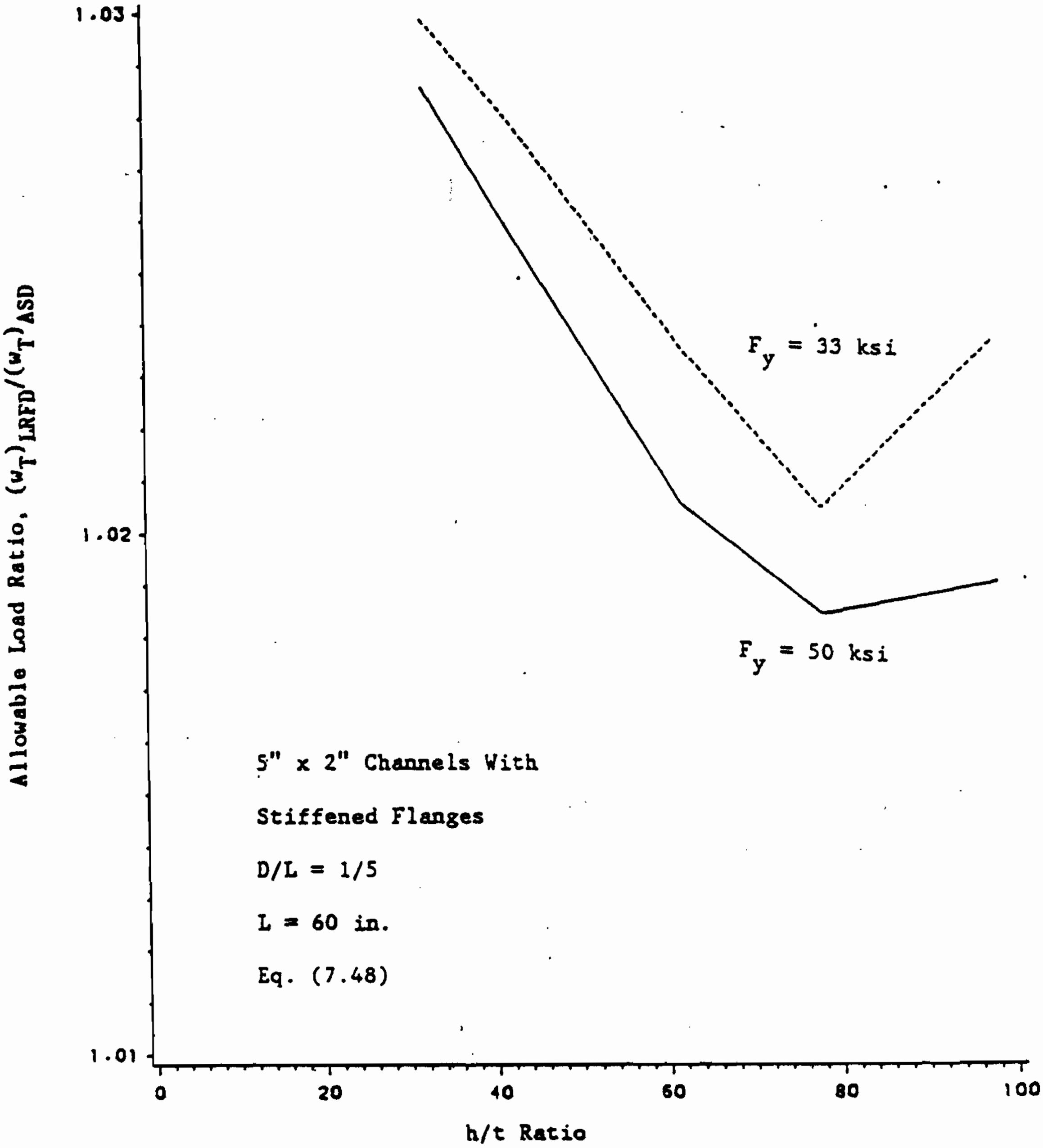

Figure 20 Allowable Load Ratio vs. h/t Ratio for Combined Bending and Shear in Beams - Case A

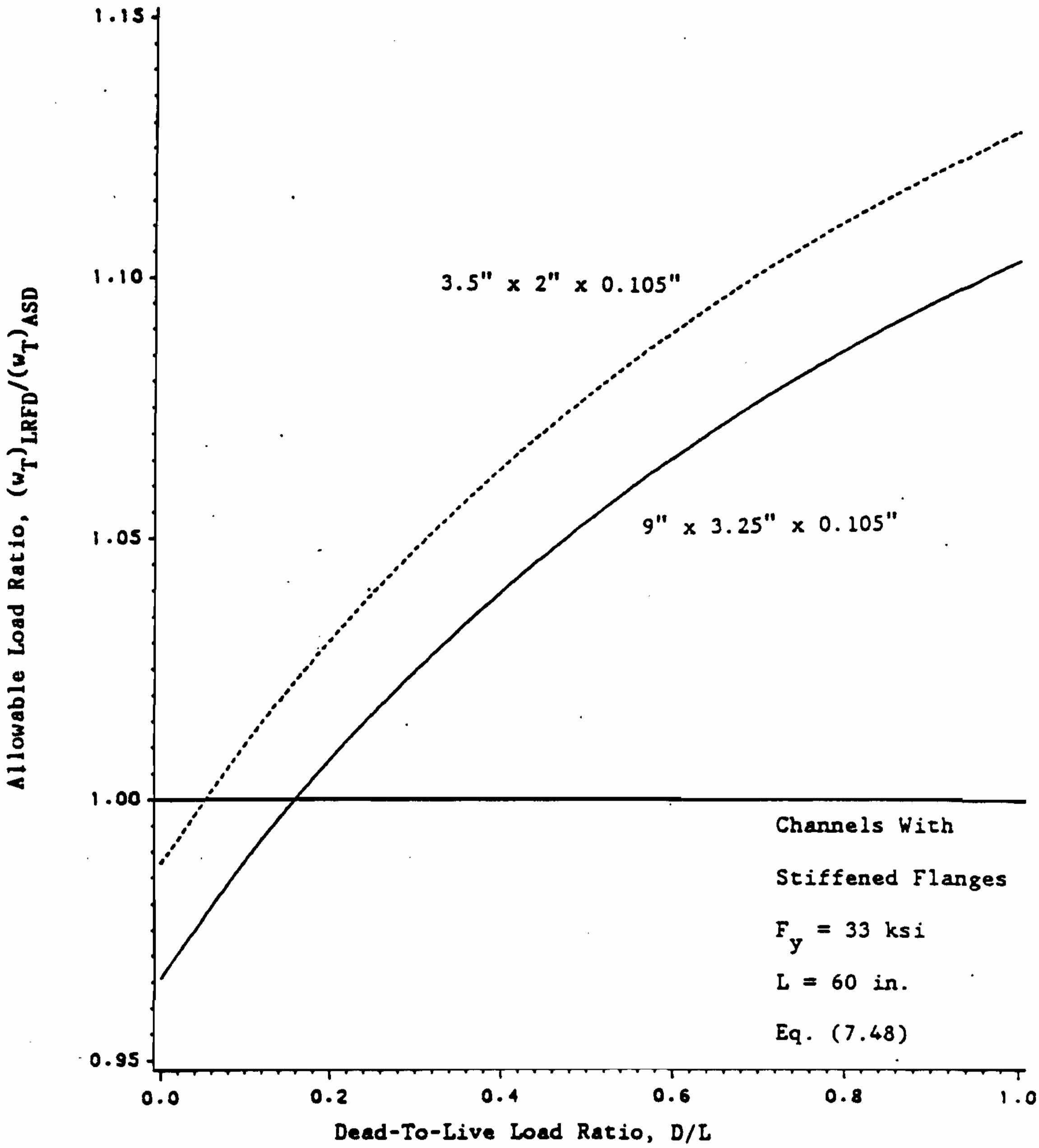

Figure 21 Allowable Load Ratio vs. D/L Ratio for Combined Bending and Shear in Beams - Case B

stiffened flanges. This is because $\phi_b = 0.90$ for sections with unstiffened compression flanges, while $\phi_b = 0.95$ for sections with stiffened compression flanges. For hat sections (positive bending), the results of the comparative study are similar to those obtained from the study of channels with stiffened flanges. Detailed information can be found in Reference 36.

I-sections made of two channels back-to-back would result in the same comparison and conclusions as the single channel sections.

From Figures 18 through 21, it can be seen that for dead-to-live load ratios less than about 1/10, the LRFD criteria for combined bending and shear are usually conservative when compared with the allowable stress design method. For D/L = 0.5, the differences range from 2.7% to 7.8%. For large D/L ratios, ASD method is always more conservative than LRFD. Yield point of steel has little effect on the allowable load ratio. However, the lower the yield point, the larger the difference. Span length has little effect on the allowable uniform load ratio as shown in Figure 19. For channels and I-sections, smaller h/t ratios result in a slightly larger difference between allowable uniform loads obtained from these two design methods.

4. Web Crippling Strength. The unfactored concentrated load or reaction can be calculated for both methods by using Eq. (7.49):

$$P_T = P_{DL} + P_{LL} \tag{7.49}$$

where

P_T = total unfactored load

P_{DL} = nominal dead load

P_{LL} = nominal live load

The total unfactored load should be less than, or equal to, the allowable load based on web crippling. For allowable stress design, the allowable load is P_a. For LRFD, the allowable load is computed from Eq. (7.4) and is as follows:

$$(P_a)_{LRFD} = \phi_w P_n (D/L+1)/(1.2D/L+1.6) \qquad (7.50)$$

For shapes with single webs, the allowable load is derived from the ultimate value with a factor of safety of 1.85. For I-sections or similar shapes, the allowable load is derived from the ultimate web crippling load using a factor of safety of 2.0. Therefore, the allowable load ratio are as follows:

For shapes with single webs and $\phi_w = 0.75$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.85\phi_w \frac{D/L+1}{1.2D/L+1.6} = 1.39 \frac{D/L+1}{1.2D/L+1.6} \qquad (7.51)$$

For I-sections or similar shapes and $\phi_w = 0.80$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.00\phi_w \frac{D/L+1}{1.2D/L+1.6} = 1.60 \frac{D/L+1}{1.2D/L+1.6} \qquad (7.52)$$

Figure 22 shows the allowable load ratio versus dead-to-live load ratio for both types of beams based on the comparison of web crippling loads.

For single web beams, LRFD is always conservative as compared with ASD approach for $D/L < 1.11$. For I-sections, the ASD approach is always more conservative than LRFD. For $D/L = 0.5$, the allowable load permitted by the allowable stress design method for I-sections is about 9% lower than that permitted by the LRFD criteria.

5. <u>Combined Bending and Web Crippling Strength</u>. A simply supported beam with a concentrated load at midspan was selected as a typical design example. This example has a maximum moment of PL/4 at midspan, under the

concentrated load. The allowable load, P_T, was calculated for each design method. Since each design procedure utilizes separate design variables, the allowable loads were determined using nominal resistances.

The allowable load based on allowable stress design was calculated as follows:

$$\frac{M}{M_a} = \frac{M_{TL}}{0.6M_n} = \frac{P_T L/4}{0.6M_n} = \frac{0.4167P_T L}{M_n} \tag{7.53}$$

For beams with single webs,

$$\frac{P}{P_a} = \frac{P_T}{P_n/1.85} = \frac{1.85P_T}{P_n} \tag{7.54}$$

By using Eqs. (7.53) and (7.54), the interaction formula for beams with single webs can be obtained as follow:

$$1.2\frac{P}{P_a} + \frac{M}{M_a} = \frac{2.22P_T}{P_n} + \frac{0.4167P_T L}{M_n} = 1.5 \tag{7.55}$$

Therefore,

$$(P_T)_{ASD} = \frac{3.6P_n}{5.328 + (P_n L/M_n)} \tag{7.56}$$

For I-sections,

$$\frac{P}{P_a} = \frac{P_T}{P_n/2.00} = \frac{2.00P_T}{P_n} \tag{7.57}$$

By using Eqs. (7.53) and (7.57), the interaction formula for I-sections can be obtained as follow:

$$1.1\frac{P}{P_a} + \frac{M}{M_a} = \frac{2.20P_T}{P_n} + \frac{0.4167P_T L}{M_n} = 1.5 \tag{7.58}$$

Therefore,

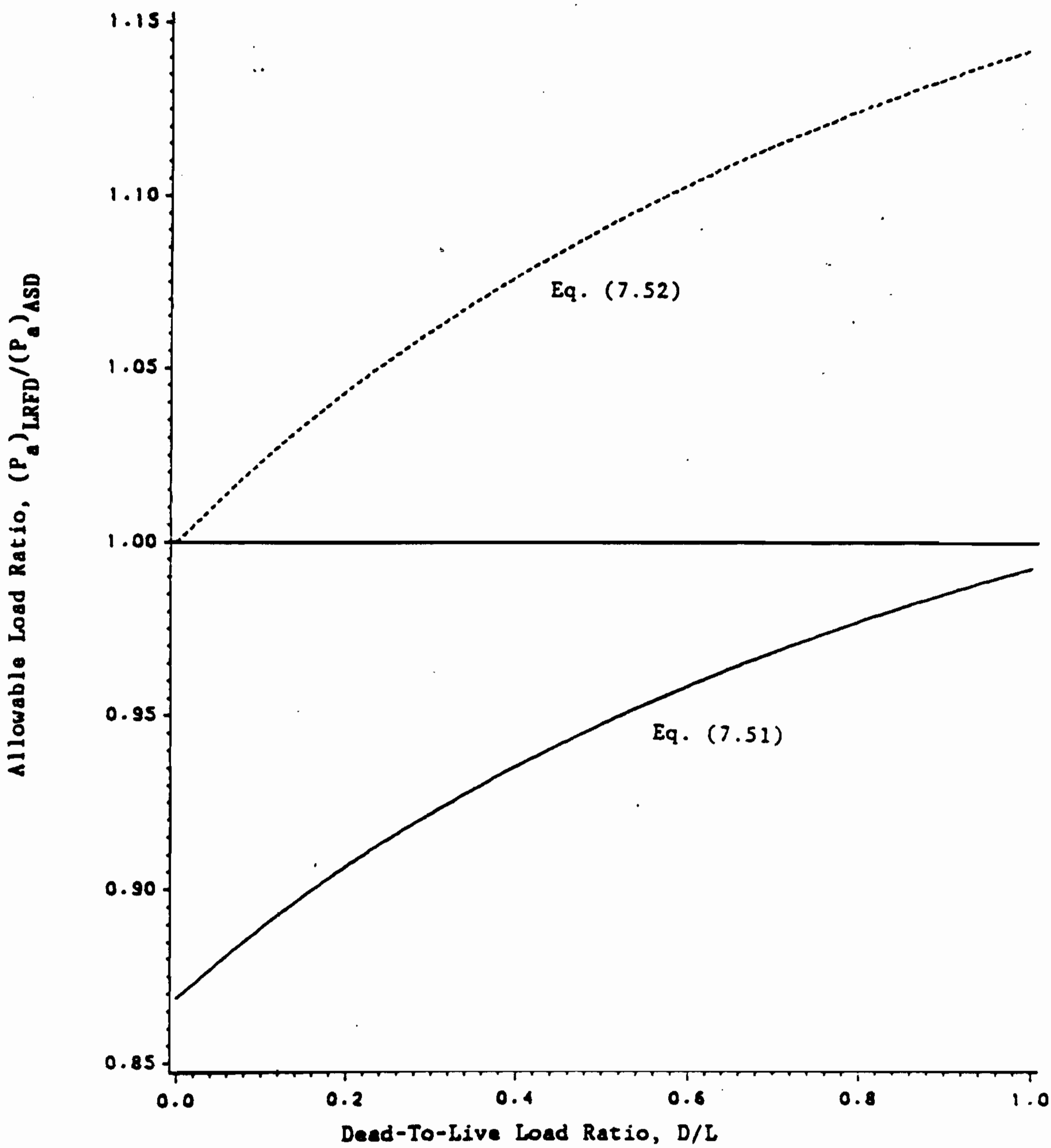

Figure 22 Allowable Load Ratio vs. D/L Ratio for Web Crippling

$$(P_T)_{ASD} = \frac{3.6P_n}{5.280 + (P_nL/M_n)} \tag{7.59}$$

The allowable load based on LRFD criteria was calculated as follows:

$$\frac{M_u}{\phi_bM_n} = \frac{1.2D/L+1.6}{D/L+1}\left[\frac{M_{TL}}{\phi_bM_n}\right] = \frac{1.2D/L+1.6}{D/L+1}\left[\frac{P_TL/4}{\phi_bM_n}\right] \tag{7.60}$$

$$\frac{P_u}{\phi_wP_n} = \frac{1.2D/L+1.6}{D/L+1}\left[\frac{P_T}{\phi_wP_n}\right] \tag{7.61}$$

For beams with single webs, Eqs. (7.60) and (7.61) were used to obtain the following interaction formula:

$$1.07\frac{P_u}{\phi_wP_n} + \frac{M_u}{\phi_bM_n} = \frac{1.2D/L+1.6}{D/L+1}(P_T)\left[\frac{1.07}{\phi_wP_n} + \frac{0.25L}{\phi_bM_n}\right] = 1.42 \tag{7.62}$$

Therefore,

$$(P_T)_{LRFD} = \frac{D/L+1}{1.2D/L+1.6}\left[\frac{5.680\phi_wP_n}{4.280+(\phi_wP_nL/\phi_bM_n)}\right] \tag{7.63}$$

For I-sections, Eqs. (7.60) and (7.61) were used to obtain the following interaction formula:

$$0.82\frac{P_u}{\phi_wP_n} + \frac{M_u}{\phi_bM_n} = \frac{1.2D/L+1.6}{D/L+1}(P_T)\left[\frac{0.82}{\phi_wP_n} + \frac{0.25L}{\phi_bM_n}\right] = 1.32 \tag{7.64}$$

Therefore,

$$(P_T)_{LRFD} = \frac{D/L+1}{1.2D/L+1.6}\left[\frac{5.280\phi_wP_n}{3.280+(\phi_wP_nL/\phi_bM_n)}\right] \tag{7.65}$$

The allowable load ratios based on the design examples for combined bending and web crippling are given in Eqs. (7.66) and (7.67) for ϕ_b = 0.95 and 0.90 for nominal section strength of sections with stiffened or partially stiffened compression flanges and unstiffened compression flanges, respectively.

For beams with single webs ($\phi_w = 0.75$),

$$\frac{(P_T)_{LRFD}}{(P_T)_{ASD}} = \frac{D/L+1}{1.2D/L+1.6}\left[\frac{6.305+1.183(P_nL/M_n)}{4.280+(0.75/\phi_b)(P_nL/M_n)}\right] \quad (7.66)$$

For I-sections ($\phi_w = 0.80$),

$$\frac{(P_T)_{LRFD}}{(P_T)_{ASD}} = \frac{D/L+1}{1.2D/L+1.6}\left[\frac{6.195+1.173(P_nL/M_n)}{3.280+(0.80/\phi_b)(P_nL/M_n)}\right] \quad (7.67)$$

Eqs. (7.66) and (7.67) can be expressed in the following form:

$$\frac{(P_T)_{LRFD}}{(P_T)_{ASD}} = \frac{D/L+1}{1.2D/L+1.6}\,(K_w) \quad (7.68)$$

where K_w is a variable determined from section properties, material strength, and span length for a particular design example.

Because the interaction combines moment and web crippling, the allowable load ratio is rather complex. It is not only a function of dead-to-live load ratio but is also a function of span length, cross sectional geometry, and material strength. Several individual beam sections with different conditions were studied due to the complexity involved in the comparison.

Figures 23 and 24 show the relationships between allowable load ratio and dead-to-live load ratio for various channel sections with stiffened flanges using L = 5 ft and F_y = 33 ksi. Tables XXX and XXXI present section properties and calculated member strengths for several channel sections with stiffened flanges selected from Table 1 of Part V of the AISI Design Manual. In these two figures for D/L = 0.5, the allowable web crippling loads determined by LRFD are from 1.1% to 1.5% larger than that permitted by allowable stress design. The channel sections with the smaller h/t ratios resulted in larger values of allowable load ratio. Therefore, with increasing h/t ratio, the difference between the allowable loads obtained from these two design methods decreases.

Figure 25 shows how the span length and yield point of steel affect the allowable load ratio for channels with stiffened flanges. Table XXXII presents calculated member strengths for different span lengths and yield points. As shown in this figure, larger span lengths will result in slightly higher values of the allowable load ratio. Also from Figure 25, it can be seen that yield point of steel has a negligible effect on the allowable load ratio.

Similar types of comparison were also studied for channels with unstiffened flanges and I-sections with stiffened flanges. In general, the allowable load ratios computed for channels with stiffened flanges ($\phi_b = 0.95$, $\phi_w = 0.75$) are larger than those computed for channels with unstiffened flanges ($\phi_b = 0.90$, $\phi_w = 0.75$) but smaller than those computed for I-sections with stiffened flanges ($\phi_b = 0.95$, $\phi_w = 0.80$). Detailed information can be obtained from Reference 36.

E. COMPARATIVE STUDY OF CONCENTRICALLY LOADED COMPRESSION MEMBERS

The unfactored load applied to the member can be computed for both design methods by using the following formula:

$$P_T = P_{DL} + P_{LL} \tag{7.69}$$

where

P_T = unfactored compressive load

P_{DL} = compressive load due to the nominal axial dead load

P_{LL} = compressive load due to the nominal axial live load

Table XXX

Channels With Stiffened Flanges

Section	h/t	P_n (Kips)	M_n (K-in.)	$\phi_b M_n$ (K-in.)	K_w
8x3x0.105	70.62	7.144	124.769	118.531	1.4830
5x2x0.105	42.05	7.455	49.625	47.144	1.4890

* F_y = 33 ksi, L = 60 in., N = 6 in.

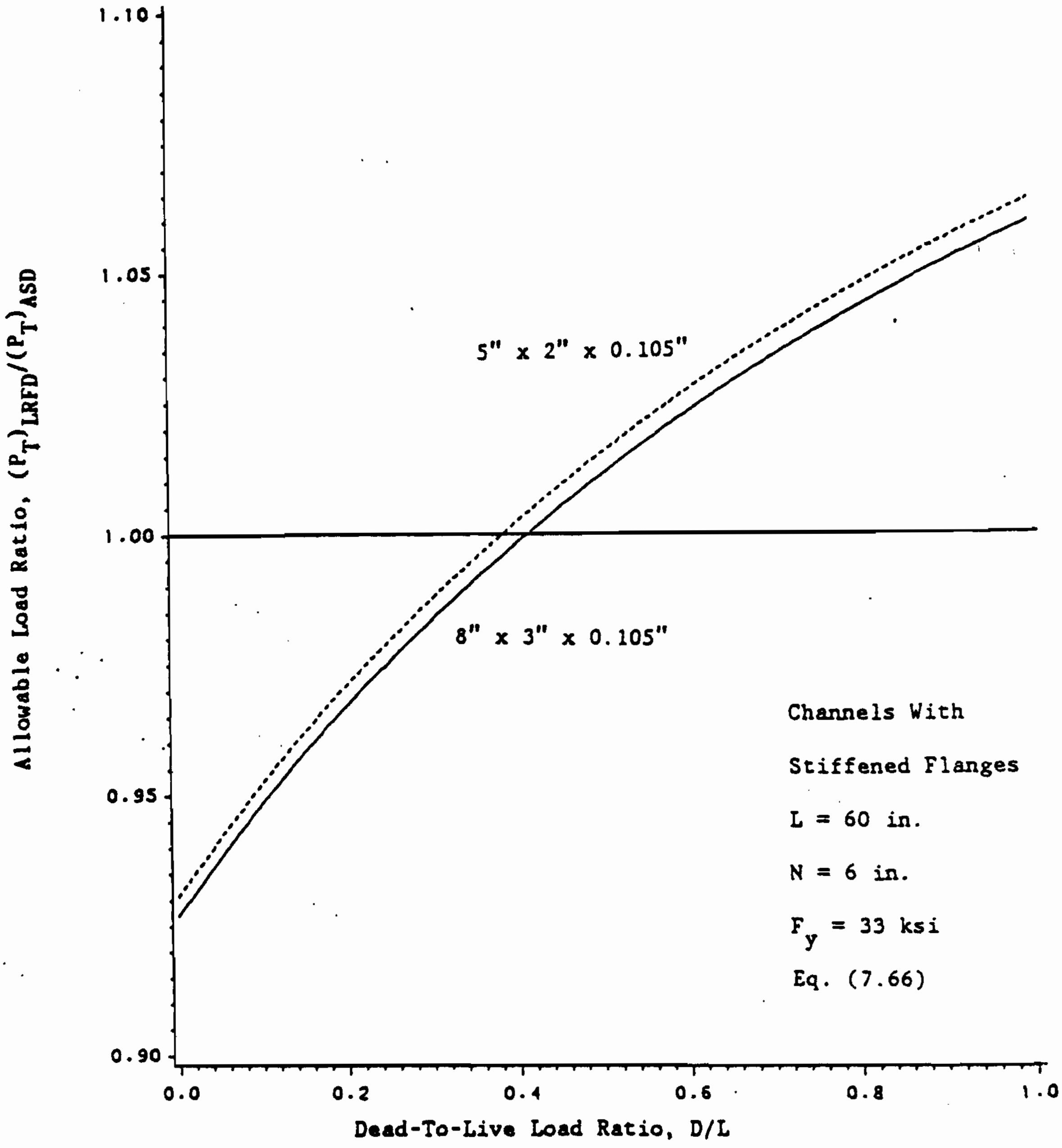

Figure 23 Allowable Load Ratio vs. D/L Ratio for Combined Bending and Web Crippling - Case 1

Table XXXI

Channels With Stiffened Flanges, 5 in. Depths

Section	h/t	P_n (Kips)	M_n (K-in.)	$\Phi_b M_n$ (K-in.)	K_w
5x2x0.075	62.17	4.443	36.917	35.071	1.4876
0.048	98.26	2.148	21.795	20.705	1.4863

* F_y = 33 ksi, L = 60 in., N = 6 in.

Table XXXII

5 in. x 2 in. x 0.105 in. Channels With Stiffened Flanges

for Various Lengths and Yield Points

F_y (ksi)	L (in.)	P_n (Kips)	M_n (K-in.)	$\phi_b M_n$ (K-in.)	K_w
33	0	7.455	49.625	47.144	1.4731
	25	7.455	49.625	47.144	1.4835
	50	7.455	49.625	47.144	1.4878
	75	7.455	49.625	47.144	1.4902
	100	7.455	49.625	47.144	1.4917
50	0	10.015	75.190	71.430	1.4731
	25	10.015	75.190	71.430	1.4828
	50	10.015	75.190	71.430	1.4871
	75	10.015	75.190	71.430	1.4896
	100	10.015	75.190	71.430	1.4911

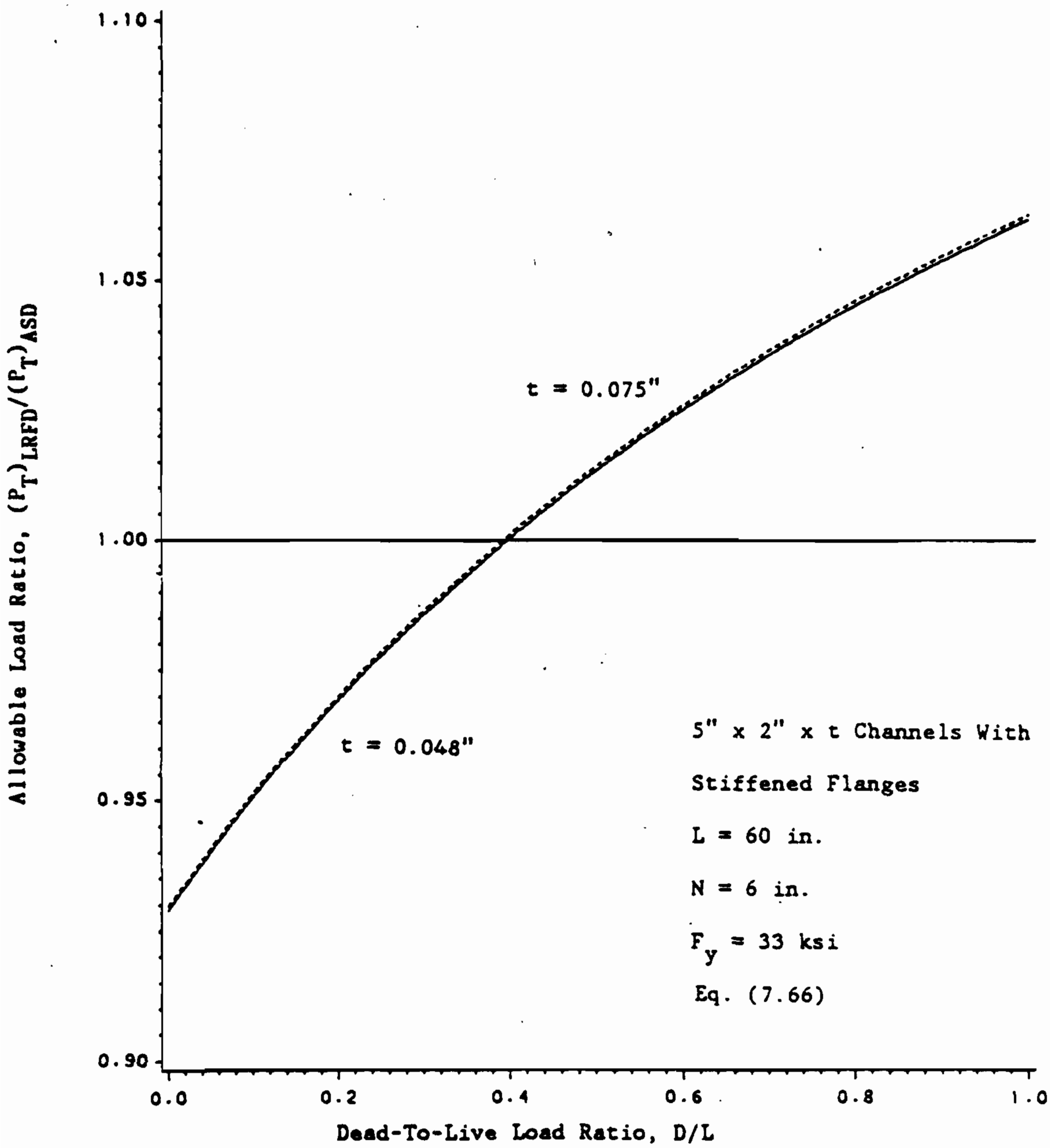

Figure 24 Allowable Load Ratio vs. D/L Ratio for Combined Bending and Web Crippling - Case 2

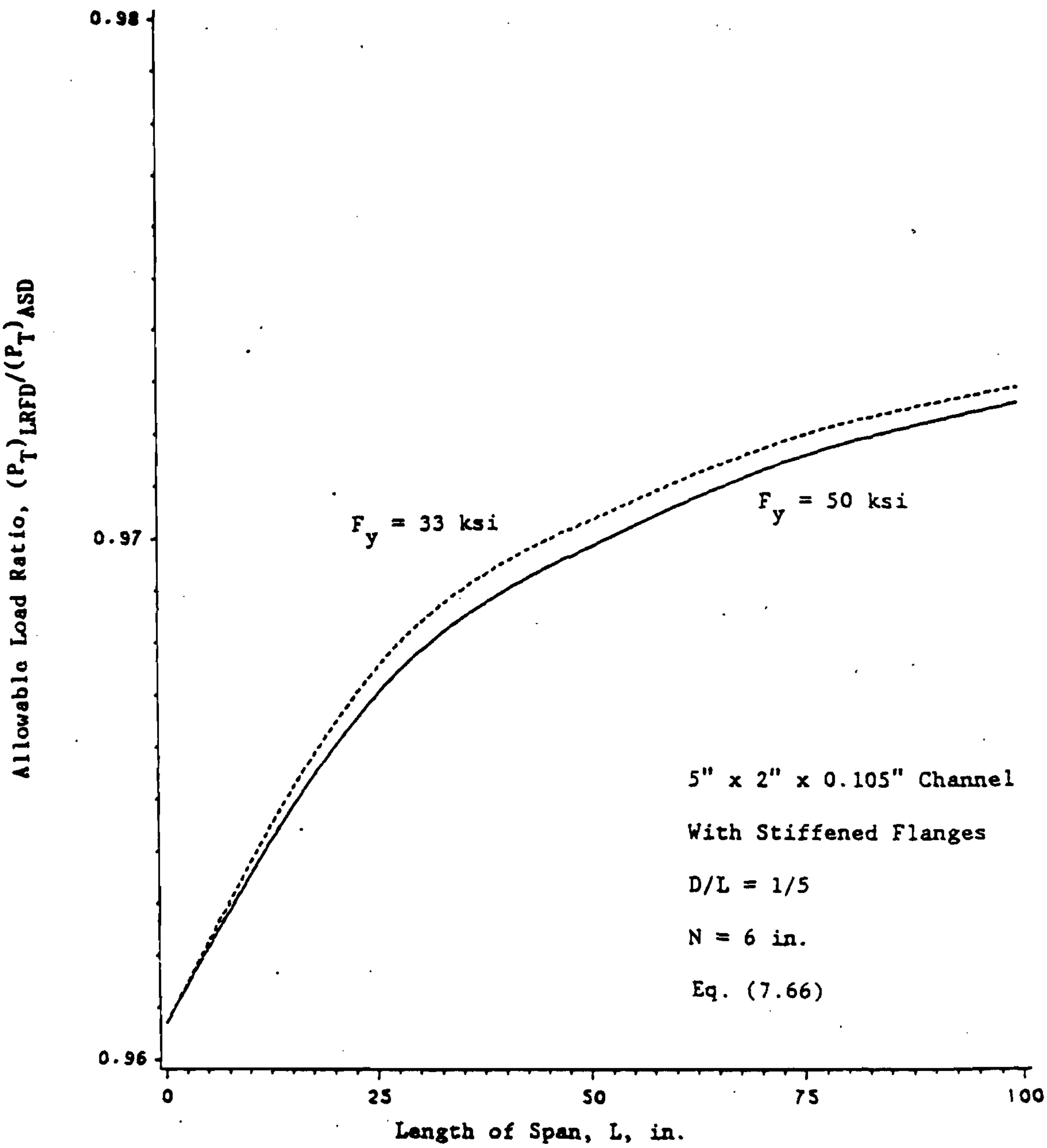

Figure 25 Allowable Load Ratio vs. Span Length for Combined Bending and Web Crippling - Case 2

The total unfactored load should be less than or equal to the allowable loads computed from allowable stress design and LRFD. For allowable stress design, the allowable load is

$$(P_a)_{ASD} = P_n/\Omega_c \tag{7.70}$$

For LRFD, the allowable axial load can be computed by using the following equation developed from Eq. (7.4):

$$(P_a)_{LRFD} = \Phi_c P_n (D/L+1)/(1.2D/L+1.6) \tag{7.71}$$

Then, the allowable load ratio can be determined as follow:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = \frac{\Phi_c P_n}{P_n/\Omega_c}\left[\frac{D/L+1}{1.2D/L+1.6}\right] = 0.85\Omega_c\left[\frac{D/L+1}{1.2D/L+1.6}\right] \tag{7.72}$$

For fully effective sections having wall thickness greater than 0.09 in. and $F_e > F_y/2$,

$$\Omega_c = 5/3+(3/8)R-(1/8)R^3 \tag{7.73}$$

where

$$R = \sqrt{(F_y/2F_e)} \tag{7.74}$$

Therefore, the allowable load ratio is

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 0.85\left(\frac{5}{3} + \frac{3}{8}R - \frac{1}{8}R^3\right)\frac{D/L+1}{1.2D/L+1.6} \tag{7.75}$$

For all other cases, Ω_c = 1.92 = 23/12, therefore the allowable load ratio is

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 0.85(23/12)\frac{D/L+1}{1.2D/L+1.6} = 1.629\frac{D/L+1}{1.2D/L+1.6} \tag{7.76}$$

Figure 26 shows the allowable load ratio versus dead-to-live load ratio for the columns used to develop Eq. (7.76). For this case, the LRFD criteria always permit larger allowable loads than the allowable stress design. For D/L = 0.5, the LRFD criteria gives an allowable load about 11% greater than the load obtained by using allowable stress design.

The allowable load ratio versus slenderness ratio, KL/r, for columns having fully effective sections, $t \geq 0.09$ in., and $F_e > F_y/2$ is shown in Figure 27. For this case, the LRFD criteria were found to be conservative for short columns as compared with allowable stress design. As shown in Figure 27, higher yield point materials give slightly higher values of the allowable load ratio.

F. COMPARATIVE STUDY OF COMBINED AXIAL LOAD AND BENDING

Because of the complexity of the interaction formulas, this comparison was studied by using two different kinds of sections, namely, doubly-symmetric sections and singly-symmetric sections.

1. Doubly-Symmetric Sections. I-sections bending about the x-axis were considered. A typical design example was selected, and the allowable axial loads were calculated by using the three interaction equations for each design method. The example used a beam-column with equal moments applied to each end so that the member is bent in single curvature. 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.

For allowable stress design the allowable axial loads were computed as follows:

$$\frac{P}{P_a} = \frac{P_T}{P_n/\Omega_c} = \frac{\Omega_c P_T}{P_n} \qquad (7.77)$$

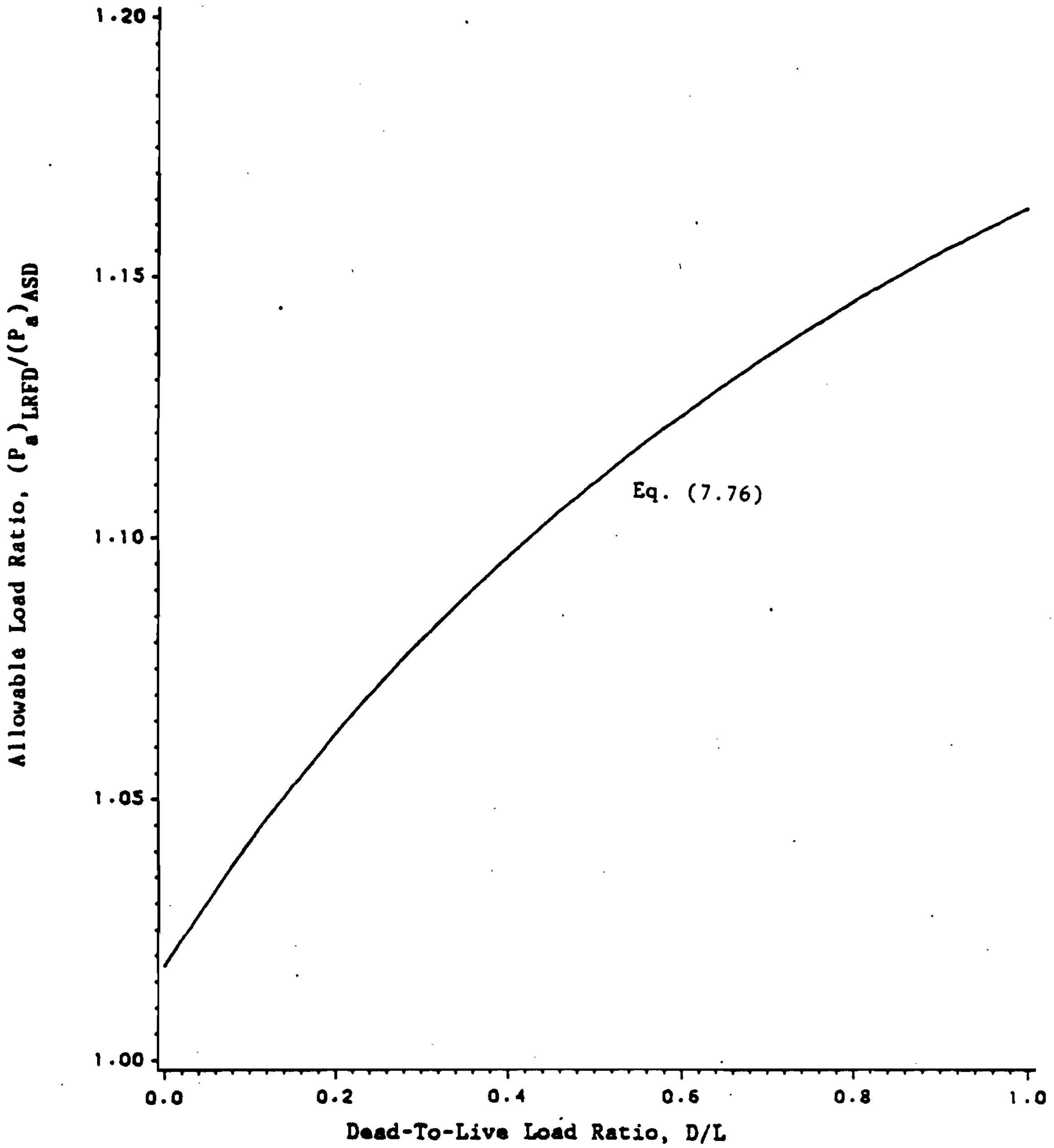

Figure 26 Allowable Load Ratio vs. D/L Ratio for Column Buckling

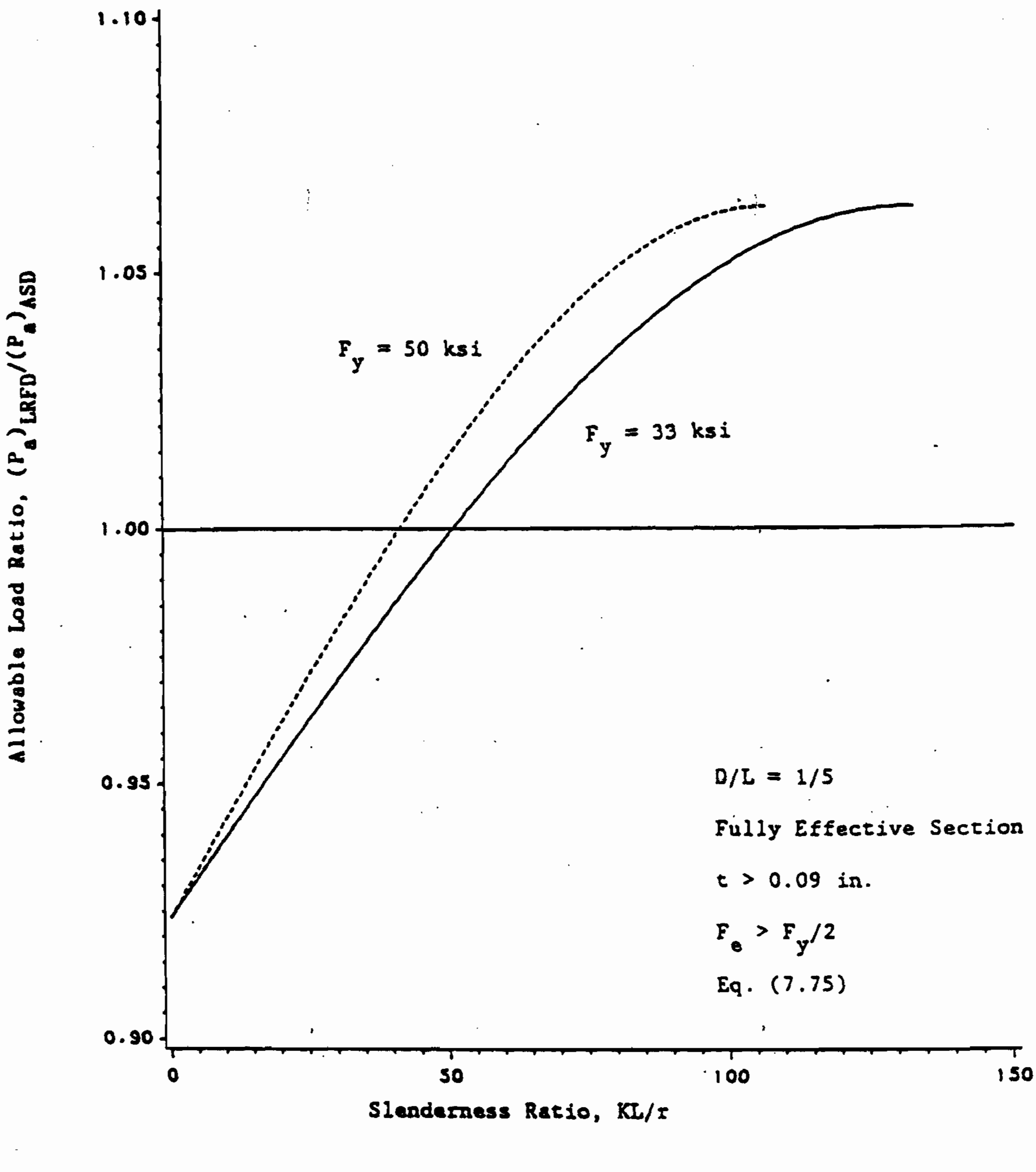

Figure 27 Allowable Load Ratio vs. Slenderness Ratio for Flexural Buckling of Columns

$$\frac{M}{M_a} = \frac{M_T}{0.6M_n} = \frac{(M_T/M_{no})(M_{no}/M_n)}{0.6} \tag{7.78}$$

where

P_T = applied unfactored axial load

M_T = applied unfactored bending moment at each end of the member

Ω_c = factor of safety of axially loaded compression members

The use of Eqs. (7.77) and (7.78) results in the following interaction formula:

$$\frac{\Omega_c P_T}{P_n} + \frac{C_m(M_T/M_{no})(M_{no}/M_n)}{0.6(1-\Omega_c P_T/P_{cr})} = 1.0 \tag{7.79}$$

where

P_{cr} = Euler buckling load

By solving for P_T in the first term of Eq. (7.79), the following equation for allowable load is obtained :

$$(P_T)_{ASD1} = \left[1- \frac{C_m(M_T/M_{no})(M_{no}/M_n)}{0.6(1-\Omega_c P_T/P_{cr})}\right]\frac{P_n}{\Omega_c} \tag{7.80}$$

Equation (7.80) is based on the failure at the midlength of the beam-column and requires a solution by iterations.

The following expression was used to solve for the allowable load based on the failure at the braced points:

$$\frac{P}{P_{ao}} = \frac{P_T}{P_{no}/\Omega_c} = \frac{\Omega_c P_T}{P_{no}} \tag{7.81}$$

where

P_{ao} = allowable axial load determined with $F_n = F_y$

The use of Eqs. (7.78) and (7.81) results in the following interaction formula:

$$\frac{\Omega_c P_T}{P_{no}} + \frac{(M_T/M_{no})(M_{no}/M_n)}{0.6} = 1.0 \qquad (7.82)$$

By solving for P_T in Eq. (7.82), the following equation for allowable load is obtained :

$$(P_T)_{ASD2} = \left[1 - \frac{(M_T/M_{no})(M_{no}/M_n)}{0.6}\right] \frac{P_{no}}{\Omega_c} \qquad (7.83)$$

Equation (7.83) is based on the failure at the braced points.

When $P/P_a \leq 0.15$, the following interaction formula can be written by using Eqs. (7.77) and (7.78) :

$$\frac{\Omega_c P_T}{P_n} + \frac{(M_T/M_{no})(M_{no}/M_n)}{0.6} = 1.0 \qquad (7.84)$$

By solving for P_T in Eq. (7.84), the following equation for allowable load is obtained :

$$(P_T)_{ASD3} = \left[1 - \frac{(M_T/M_{no})(M_{no}/M_n)}{0.6}\right] \frac{P_n}{\Omega_c} \qquad (7.85)$$

Equation (7.85) is based on the flexural failure when the effect of the secondary moment is neglected.

For LRFD, the allowable axial loads were computed in accordance with Eq. (7.4) as follows :

$$\frac{P_u}{\phi_c P_n} = \frac{1.2D/L+1.6}{D/L+1} \left[\frac{P_T}{\phi_c P_n}\right] \qquad (7.86)$$

$$\frac{M_u}{\phi_b M_n} = \frac{1.2D/L+1.6}{D/L+1} \left[\frac{(M_T/M_{no})(M_{no}/M_n)}{\phi_b}\right] \qquad (7.87)$$

$$\frac{P_u}{\phi_c P_E} = \frac{1.2D/L+1.6}{D/L+1} \left[\frac{P_T}{\phi_c P_E}\right] \qquad (7.88)$$

The use of Eqs. (7.86), (7.87), and (7.88) results in the following interaction formula:

$$\frac{1.2D/L+1.6}{D/L+1}\left\{\frac{P_T}{\phi_c P_n}+\frac{C_m(M_T/M_{no})(M_{no}/M_n)}{\phi_b[1-(1.2D/L+1.6)P_T/(D/L+1)\phi_c P_E]}\right\}=1.0 \quad (7.89)$$

By solving for P_T in the first term of Eq. (7.89), the following equation for allowable load is obtained :

$$(P_T)_{LRFD1}=\left\{\frac{D/L+1}{1.2D/L+1.6}-\frac{C_m(M_T/M_{no})(M_{no}/M_n)}{\phi_b[1-(1.2D/L+1.6)P_T/(D/L+1)\phi_c P_E]}\right\}\phi_c P_n \quad (7.90)$$

Equation (7.90) is based on the flexural failure at the midlength of the beam-column and requires a solution by iterations.

The following expression was used to solve for the allowable load based on the failure at the braced points:

$$\frac{P_u}{\phi_c P_{no}}=\frac{1.2D/L+1.6}{D/L+1}\left[\frac{P_T}{\phi_c P_{no}}\right] \quad (7.91)$$

The use of Eqs. (7.87) and (7.91) results in the following interaction formula:

$$\frac{1.2D/L+1.6}{D/L+1}\left[\frac{P_T}{\phi_c P_{no}}+\frac{(M_T/M_{no})(M_{no}/M_n)}{\phi_b}\right]=1.0 \quad (7.92)$$

By solving for P_T in Eq. (7.92), the following equation for allowable load is obtained :

$$(P_T)_{LRFD2}=\left[\frac{D/L+1}{1.2D/L+1.6}-\frac{(M_T/M_{no})(M_{no}/M_n)}{\phi_b}\right]\phi_c P_{no} \quad (7.93)$$

Equation (7.93) is based on the failure at the braced points.

When $P_u/(\phi_c P_n) \leq 0.15$, the following interaction formula can be written by using Eqs. (7.86) and (7.87) :

$$\frac{1.2D/L+1.6}{D/L+1}\left[\frac{P_T}{\phi_c P_n}+\frac{(M_T/M_{no})(M_{no}/M_n)}{\phi_b}\right]=1.0 \qquad (7.94)$$

By solving for P_T in Eq. (7.94), the following equation for allowable load is obtained :

$$(P_T)_{LRFD3}=\left[\frac{D/L+1}{1.2D/L+1.6}-\frac{(M_T/M_{no})(M_{no}/M_n)}{\phi_b}\right]\phi_c P_n \qquad (7.95)$$

Equation (7.95) is based on the flexural failure when the effect of the secondary moment is neglected.

Equations (7.80), (7.83), and (7.85) for determining the allowable axial load based on allowable stress design and Eqs. (7.90), (7.93), and (7.95) for determining the allowable axial load based on LRFD are very complex and utilize iterations with multiple variables. The allowable load ratios, $(P_T)_{LRFD}/(P_T)_{ASD}$, for various lengths combined with different applied end moment ratios, M_T/M_{no}, with respect to the beam strength of the member were studied. Typical I-sections and their section properties used in this study were obtained from Tables 5 and 6 of Part V of the AISI Cold-Formed Steel Design Manual.

An I-section (3.5 in. x 4 in. x 0.105 in.) with stiffened flanges was studied with a yield point of 33 ksi. Figure 28 shows the allowable load ratio versus dead-to-live load ratio for a 4 ft length with various end moment ratios, M_T/M_{no}. This figure is based on Eqs. (7.80) and (7.90) for flexural failure at the midlength of the beam-column. For a D/L ratio around 0.35, the LRFD criteria gives an allowable load about 9% more than the value computed from allowable stress design for all end moment ratios indicated in the figure. For other values of the D/L ratio, the difference between the allowable loads computed by using these two methods depends on the end moment ratio as shown in Figure 28. For D/L > 0.35, the larger the end moment ratio, the higher the allowable load ratio. For example, for D/L = 0.5, the $(P_T)_{LRFD}/(P_T)_{ASD}$ ratios are 1.137 and 1.117 for M_T/M_{no} = 0.3 and 0.1, respectively.

Figure 29 shows the allowable load ratio based on Eqs. (7.83) and (7.93) versus dead-to-live load ratio for the same I-section used in Figure 28. Figure 29 is based on failure at the braced points which corresponds to Eqs. (7.83) and (7.93). For D/L = 0.5, the allowable loads obtained from LRFD are from 11.6% to 13.6% greater than allowable loads determined from allowable stress design for end moment ratios from 0.1 to 0.3.

Figures 30 and 31 show the relationships between allowable load ratio and dead-to-live load ratio for end moment ratios of 0.2 and 0.3, respectively. The different curves in each figure represent different lengths of the 3.5 in. x 4 in. x 0.105 in. I-section. With end moment ratio of 0.2 and D/L = 0.5, ASD would provide conservative values up to 12.9% for column lengths equal to 4 ft, 7 ft, and 9 ft as compared with the LRFD method. For the same column lengths and an end moment ratio of 0.3, ASD would be conservative (13.7% to 14.8%) as compared with the LRFD method for D/L = 0.5.

The relationships between the allowable load ratio and column length are shown in Figures 30 and 31 for various D/L ratios. Figures 32 and 33 show the allowable load ratio versus slenderness ratio, KL/r_y, for end moment ratios of 0.2 and 0.3, respectively. Each curve in the figure represents a different D/L ratio for the same I-section used in Figures 28 through 31. As shown in these two figures, the allowable load ratio increases with increasing slenderness ratios for large D/L ratios. For small D/L ratios, the slenderness ratio has small effect on the allowable load ratio. These two figures also show that for all three D/L ratios, the LRFD method would permit a larger load than the ASD method.

A deeper I-section (6 in. x 5 in. x 0.105 in.) with stiffened flanges was also studied for a length of 5 ft. Figure 34 shows the allowable load ratio, based on Eqs. (7.80) and (7.90), versus dead-to-live load ratio for various end moment ratios. This figure is also based on flexural failure at the midlength of the beam-column which governs the design for this case. The curves without star symbols are for $C_m = 1.0$. They are the same as those shown in Figure 28 for the 4 in. deep I-section. For this case, the yield point of steel would not affect the allowable load ratio. For D/L = 0.5 and $M_T/M_{no} = 0.1$, the allowable load computed from LRFD is 11.6% greater than the value determined from allowable stress design. However, for D/L = 0.5 and $M_T/M_{no} = 0.3$, the allowable load computed from LRFD is 13.6% higher than the value computed from allowable stress design.

The curves with star symbols in Figure 34 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 Figure 34. 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 $C_m = 1.0$.

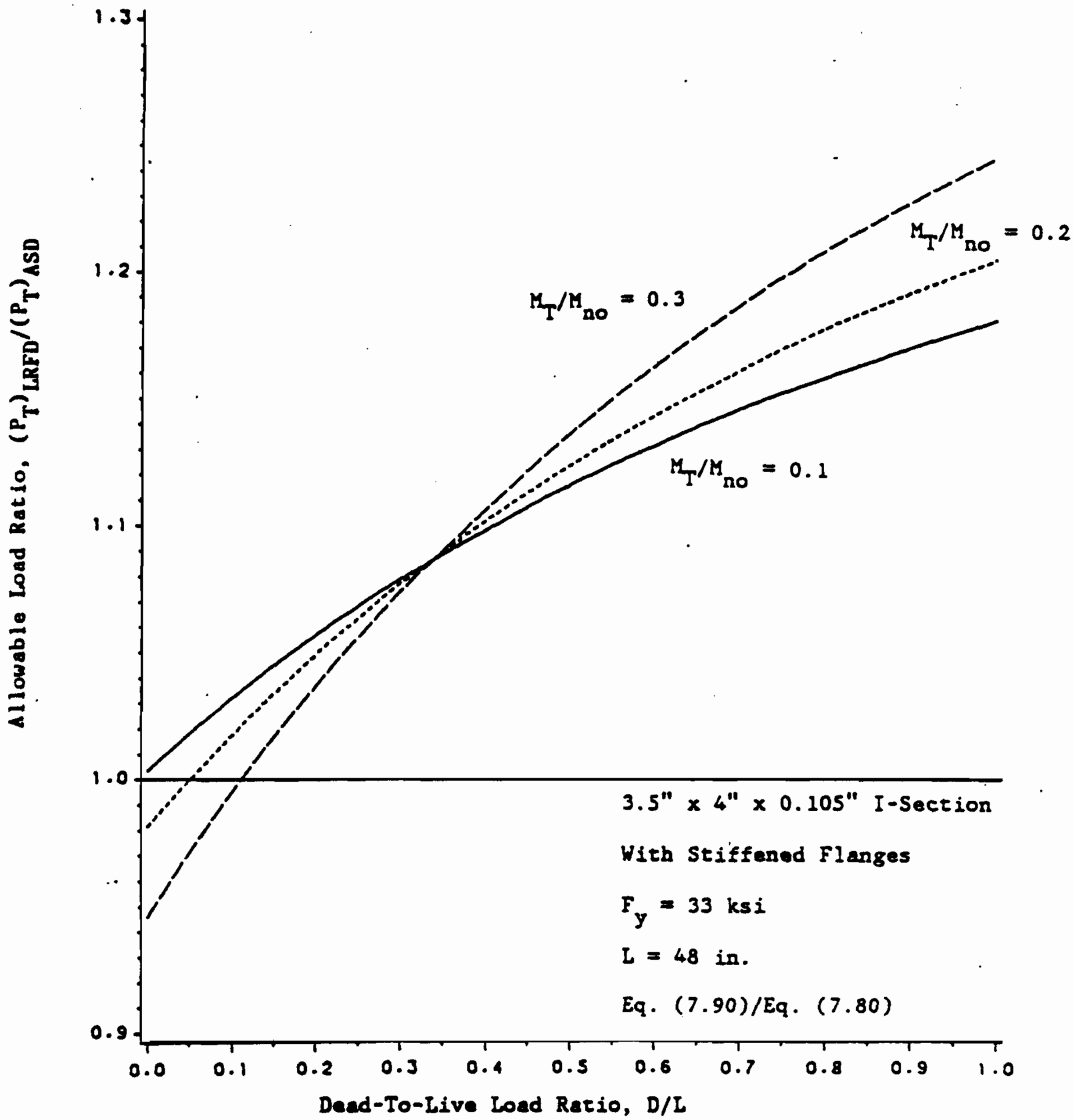

Figure 28 Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case A

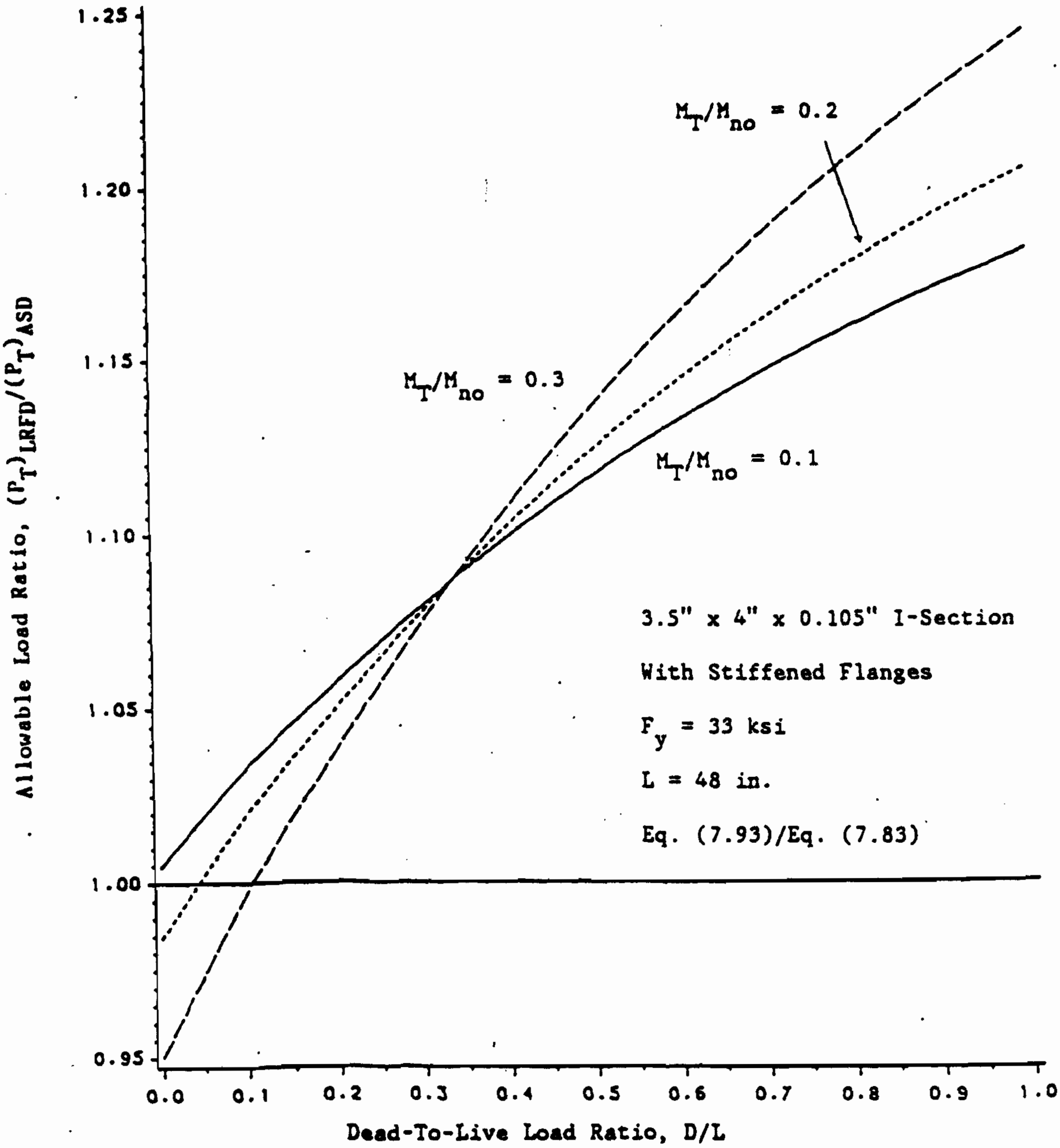

Figure 29 Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case B

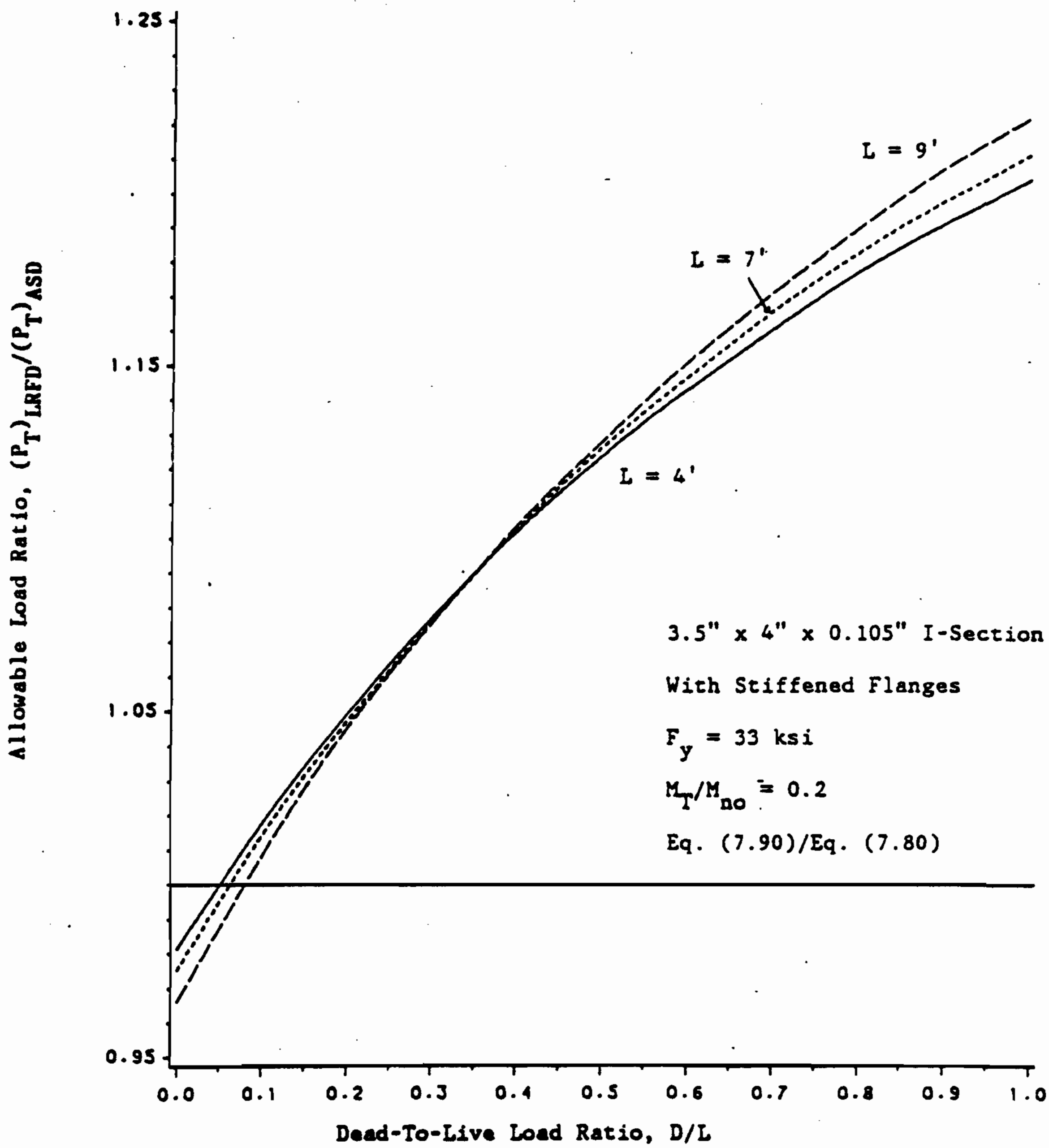

Figure 30 **Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case C**

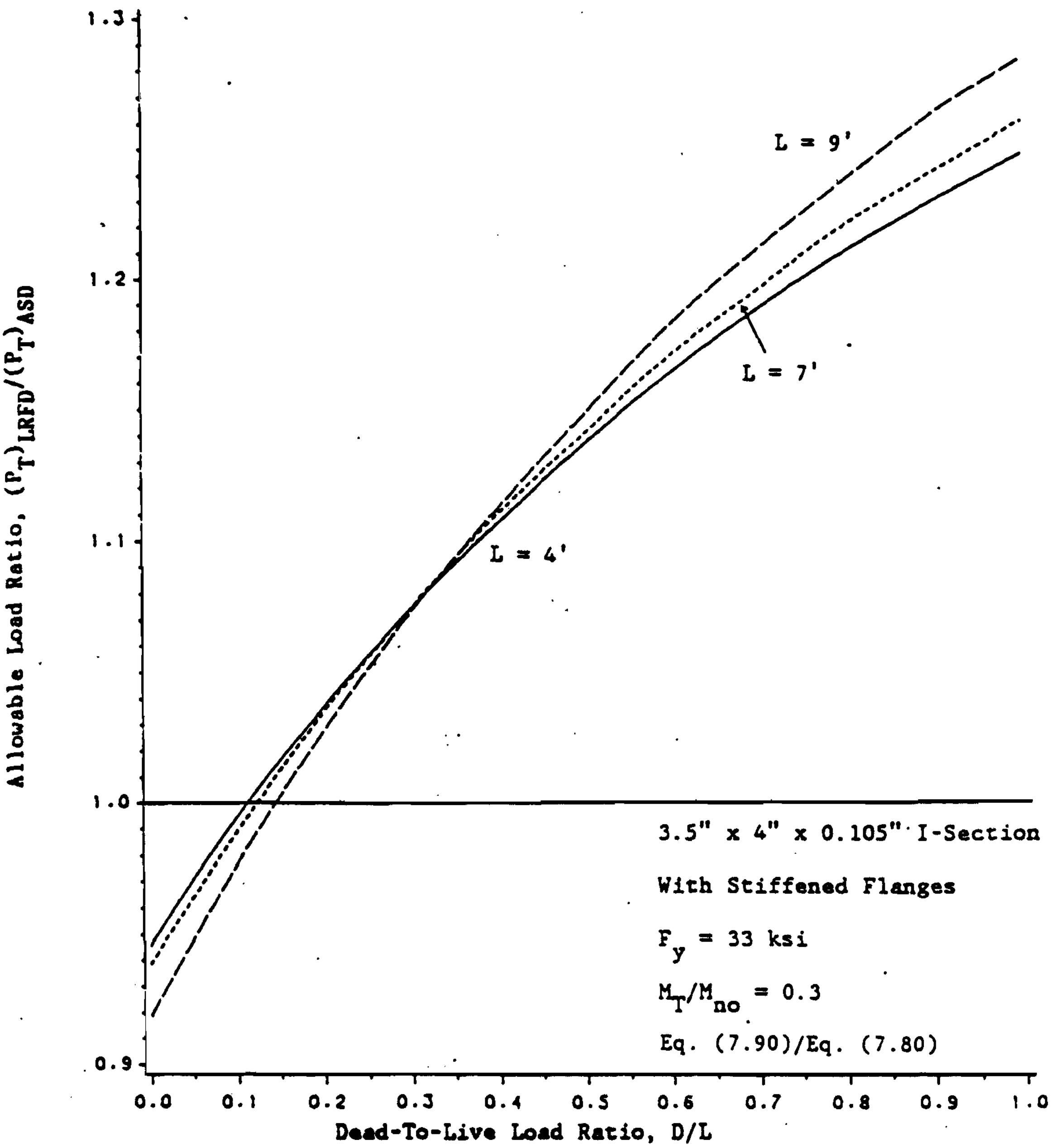

Figure 31 Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case D

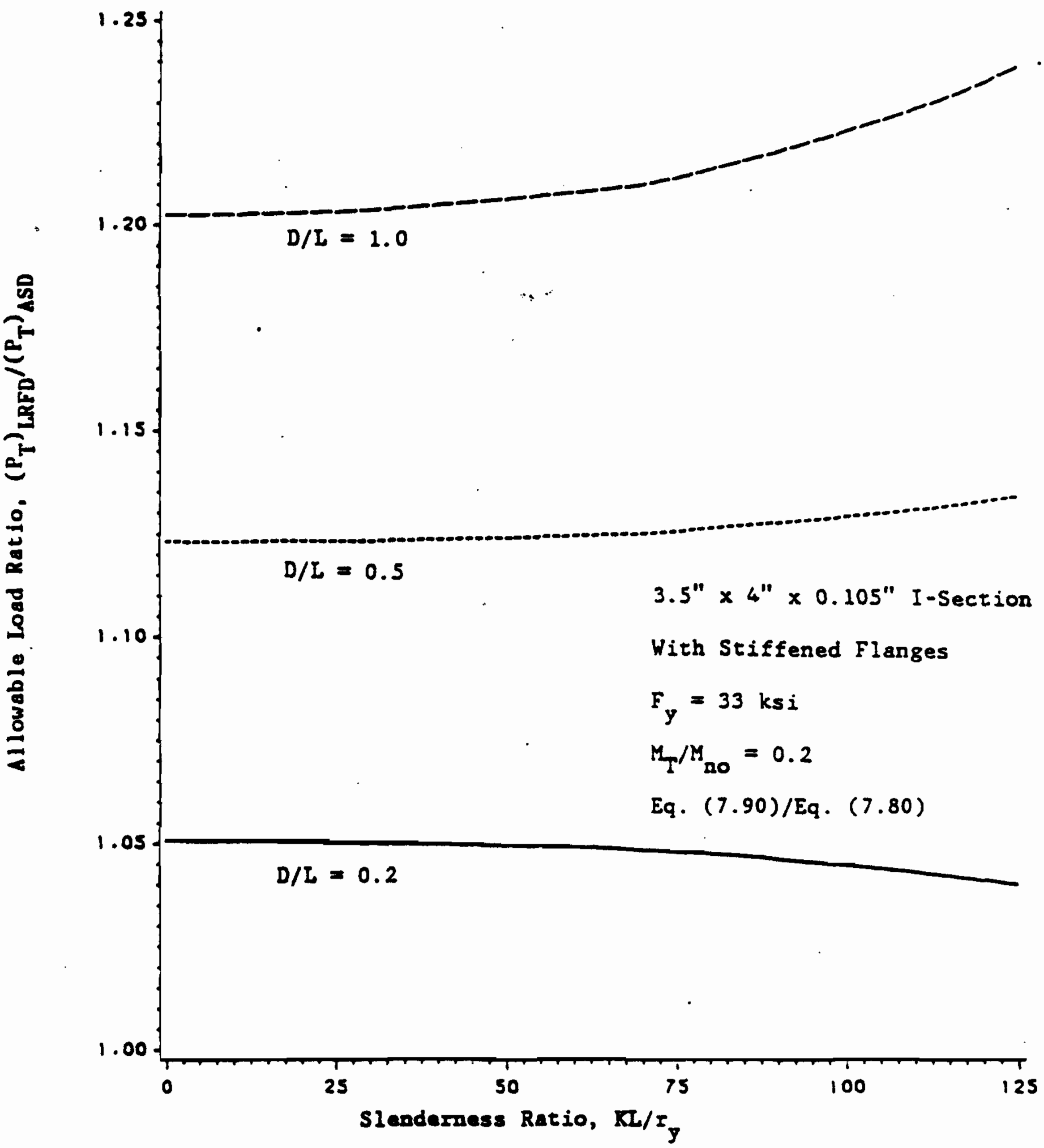

Figure 32 Allowable Load Ratio vs. Slenderness Ratio for Beam-Columns - Case C

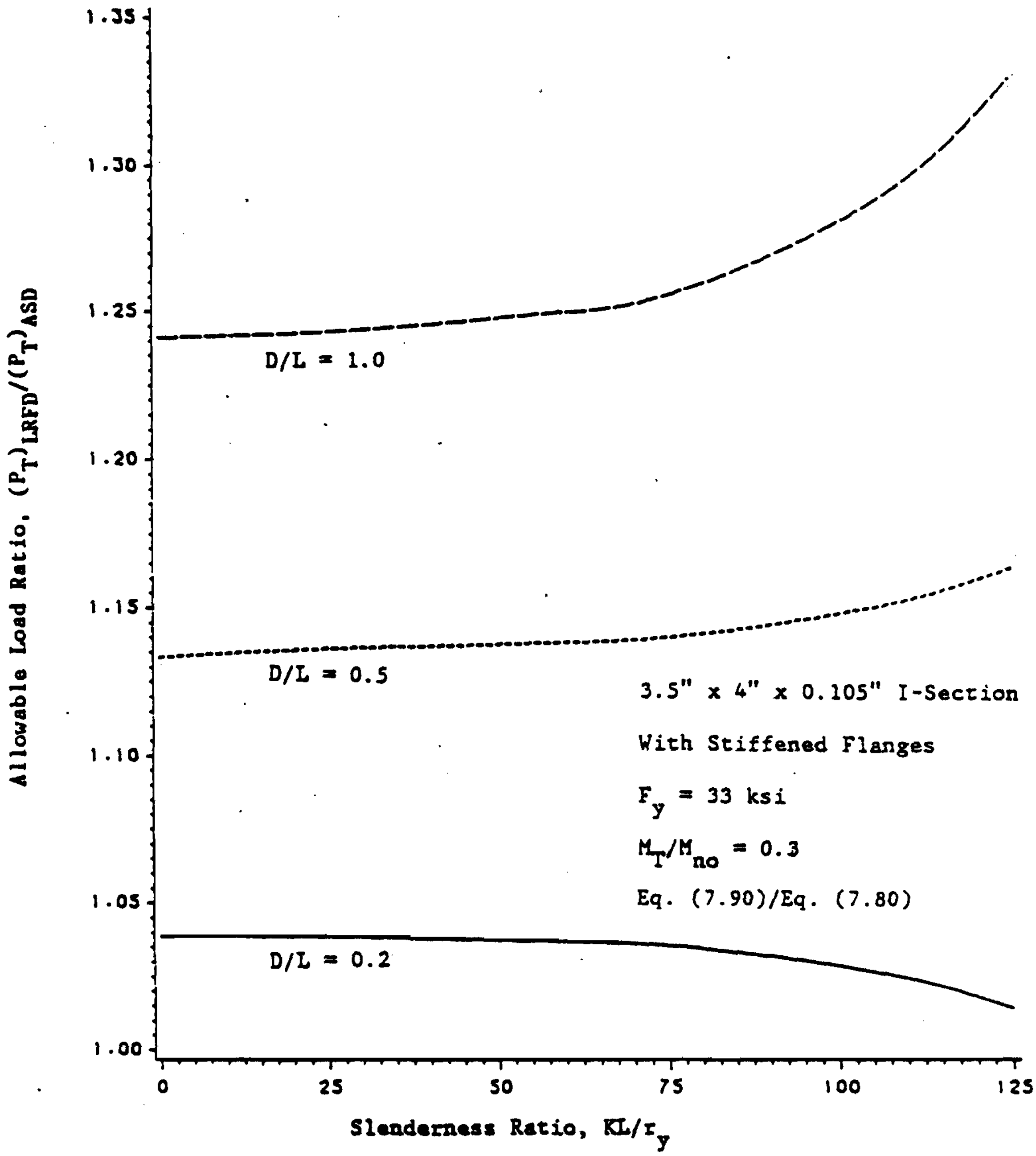

Figure 33 Allowable Load Ratio vs. Slenderness Ratio for Beam-Columns - Case D

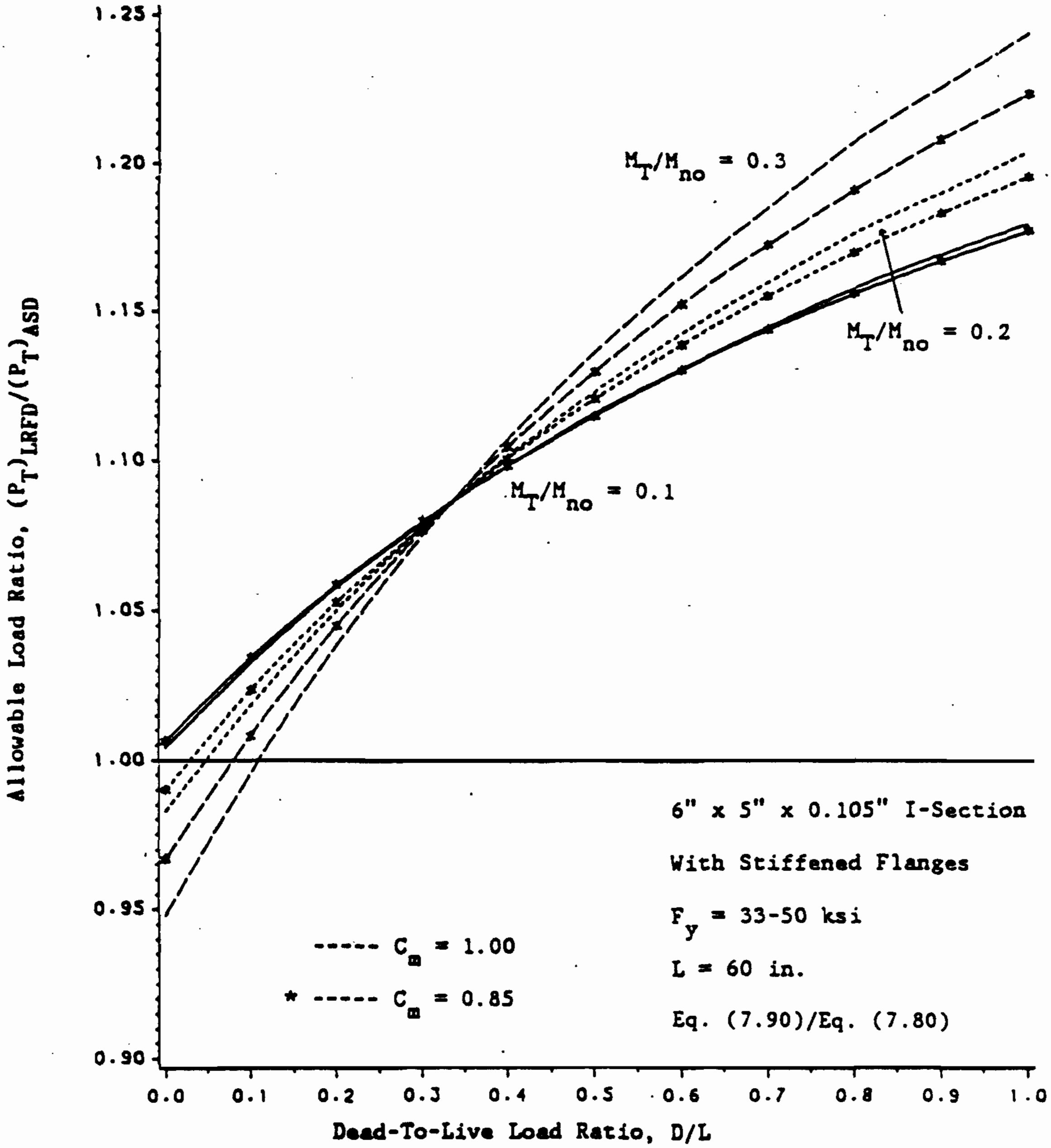

Figure 34 Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case E

I-sections with unstiffened flanges were studied in a similar manner. The results of the comparative study are similar to those obtained from the study of I-sections with stiffened flanges. Detailed information can be found in Reference 36.

2. Singly-Symmetric Sections. The allowable eccentric axial loads were calculated for allowable stress design and LRFD. The applied end moments are a result of the eccentric axial loads, and can be calculated using the following equation:

$$M_T = e_T P_T \qquad (7.96)$$

where

$e_T = e - e_x$

e = eccentricity of the axial load with respect to the centroidal axis of the full section, negative when on the shear center side of the centroid

e_x = distance between the centroid of the full section and the centroid of the effective section, negative when on the shear center side of the centroid of the full section

Procedures similar to the ones made to solve for the allowable loads of beam-columns with doubly-symmetric shapes were used to solve for the allowable loads for members with singly-symmetric shapes.

For allowable stress design, the interaction formula for flexural failure at the midlength of the beam-column can be obtained by using Eqs. (7.77), (7.78), and (7.96) as follow:

$$\frac{\Omega_c P_T}{P_n} + \frac{C_m e_T P_T}{0.6M_n(1-\Omega_c P_T/P_{cr})} = 1.0 \qquad (7.97)$$

By solving for P_T in Eq. (7.97), the following equation for allowable load is obtained :

$$(P_T)_{ASD1} = \frac{1.0}{\dfrac{\Omega_c}{P_n} + \dfrac{C_m e_T}{0.6M_n(1-\Omega_c P_T/P_{cr})}} \qquad (7.98)$$

Equation (7.98) requires a solution using iterations, since the allowable axial load is a function of the actual axial load , P_T.

For flexural failure at the braced points, the interaction formula used for the allowable stress design can be obtained by using Eqs. (7.81), (7.78), and (7.96) as follow:

$$\frac{\Omega_c P_T}{P_{no}} + \frac{e_T P_T}{0.6M_n} = 1.0 \qquad (7.99)$$

By solving for P_T in Eq. (7.99), the following equation for allowable load is obtained :

$$(P_T)_{ASD2} = \frac{1.0}{\dfrac{\Omega_c}{P_{no}} + \dfrac{e_T}{0.6M_n}} \qquad (7.100)$$

For allowable stress design, the interaction formula based on flexural failure without the effect of secondary moment can be obtained by using Eqs. (7.77), (7.78), and (7.96) as follow:

$$\frac{\Omega_c P_T}{P_n} + \frac{e_T P_T}{0.6M_n} = 1.0 \qquad (7.101)$$

The following equation for allowable load is obtained by solving for P_T in Eq. (7.101):

$$(P_T)_{ASD3} = \frac{1.0}{\dfrac{\Omega_c}{P_n} + \dfrac{e_T}{0.6M_n}} \tag{7.102}$$

For LRFD, the interaction formula for flexural failure at the midlength of the beam-column can be obtained by using Eqs. (7.86), (7.88), and (7.96) as follow:

$$\frac{1.2D/L+1.6}{D/L+1}\left\{\frac{P_T}{\phi_c P_n} + \frac{C_m e_T P_T}{\phi_b M_n\left[1-(1.2D/L+1.6)P_T/(D/L+1)\phi_c P_E\right]}\right\} = 1.0 \tag{7.103}$$

By solving for P_T in Eq. (7.103), the following equation for allowable load is obtained :

$$(P_T)_{LRFD1} = \frac{(D/L+1)/(1.2D/L+1.6)}{\dfrac{1}{\phi_c P_n} + \dfrac{C_m e_T}{\phi_b M_n\left[1-(1.2D/L+1.6)P_T/(D/L+1)\phi_c P_E\right]}} \tag{7.104}$$

Equation (7.104) requires a solution by using iterations, since the allowable axial load is also a function of the actual axial load.

For flexural failure at the braced points, the interaction formula used for the LRFD can be obtained by using Eqs. (7.91), (7.87), and (7.96) as follow:

$$\frac{1.2D/L+1.6}{D/L+1}\left[\frac{P_T}{\phi_c P_{no}} + \frac{e_T P_T}{\phi_b M_n}\right] = 1.0 \tag{7.105}$$

By solving for P_T in Eq. (7.105), the following equation for allowable load is obtained :

$$(P_T)_{LRFD2} = \frac{(D/L+1)/(1.2D/L+1.6)}{\dfrac{1}{\phi_c P_{no}} + \dfrac{e_T}{\phi_b M_n}} \tag{7.106}$$

For LRFD, the interaction formula based on flexural failure without the effect of secondary moment can be obtained by using Eqs. (7.86), (7.87), and (7.96) as follow:

$$\frac{1.2D/L+1.6}{D/L+1}\left[\frac{P_T}{\phi_c P_n} + \frac{e_T P_T}{\phi_b M_n}\right] = 1.0 \qquad (7.107)$$

The following equation for allowable load was obtained by solving for P_T in Eq. (7.107):

$$(P_T)_{LRFD3} = \frac{(D/L+1)/(1.2D/L+1.6)}{\dfrac{1}{\phi_c P_n} + \dfrac{e_T}{\phi_b M_n}} \qquad (7.108)$$

The equations to be used for the allowable eccentric axial load for allowable stress design and LRFD are very complex and utilize iterations with multiple variables. The allowable load ratios, $(P_T)_{LRFD}/(P_T)_{ASD}$, for various lengths and eccentricities were studied. Typical channel sections and their section properties used in this study, were obtained from Tables 1 and 2 of Part V of the AISI Cold-Formed Steel Design Manual.

A channel (4 in. x 2 in. x 0.105 in.) with stiffened flanges was studied as a beam-column subjected to an eccentric load applied at each end. Figure 35 shows the allowable load ratio versus eccentricity for the channel with an effective length of 5 ft, D/L = 0.5, and C_m = 1.0. From this figure, it can be seen that the smaller the eccentricity the larger the allowable load ratio and this relationship holds for both positive and negative eccentricities.

The top line in Figure 35 represents the same channel section with a yield point of 50 ksi. The allowable load ratios in this case are slightly greater than that computed with F_y = 33 ksi.

Figure 36 shows the relationship between allowable load ratio and dead-to-live load ratio for the 4 in. deep channel with e = + 1.29 in. The two curves represent yield points of 33 and 50 ksi for the 5 ft long beam-column. The higher yield point steels result in slightly higher values of the allowable load ratio as seen in Figures 35 and 36. From the computer output, the value of F_y has a negligible effect on the allowable load ratio for the same channel with - 0.25 in. < e < + 0.25 in. and effective length equals to 5 ft.

Figure 37 shows the allowable load ratio versus slenderness ratio, KL/r_y, for the channel (4 in. x 2 in. x 0.105 in.) with stiffened flanges and D/L = 1/5. The curves represent yield points of 33 and 50 ksi for the channel with e = + 1.29 in. For F_y = 33 ksi, the allowable load ratio increases slightly as the slenderness ratio increases up to KL/r_y = 160. For KL/r_y > 160, the allowable load ratio decreases as the slenderness ratio increases. The slenderness ratio has a larger effect on the allowable load ratio for the channel with F_y = 50 ksi as compared with F_y = 33 ksi. For F_y = 50 ksi, the allowable load ratio increases as the slenderness ratio increases up to KL/r_y = 130. For KL/r_y > 130, the allowable load ratio decreases as the slenderness ratio increases.

A deeper channel (6 in. x 2.5 in. x 0.105 in.) with stiffened flanges was also studied. The relationship between allowable load ratio and eccentricity for the channel with a length of 5 ft and D/L = 0.5 is shown in Figure 38. The bottom line represents the curve for C_m = 1.0 which would be used for braced frames. For this case, the curve is similar to that shown in Figure 35 for the 4 in. deep channel.

The top line in Figure 38 represents the same channel with $C_m = 0.85$. This value of C_m is used for unbraced frames and beam-columns with restrained ends subjected to transverse loading between its supports. The curve for $C_m = 0.85$ is similar to the curve for $C_m = 1.0$ except that $C_m = 0.85$ results in a higher allowable load ratio than $C_m = 1.0$. The effect of the value of C_m on the allowable load ratio is neglegible for -0.25 in. $< e < +0.25$ in. as shown in Figure 38.

Figure 39 shows the allowable load ratio versus dead-to-live load ratio for the channel used in Figure 38. The curves represent the allowable load ratios for various eccentricities by using $F_y = 33$ ksi and $C_m = 1.0$. It can be seen from this figure that the eccentricity does not affect the shape of the curve but does affect the value of the allowable load ratio.

Channels with unstiffened flanges were studied in a similar manner. The curves obtained for channels with unstiffened flanges are similar to these curves obtained for channels with stiffened flanges. Detailed information can be found in Reference 36.

G. COMPARATIVE STUDY OF STIFFENERS

1. Transverse Stiffeners. The unfactored load applied to the stiffener can be computed for both design methods by using the following formula:

$$P_T = P_{DL}+P_{LL} \tag{7.109}$$

where

P_T = unfactored compressive load

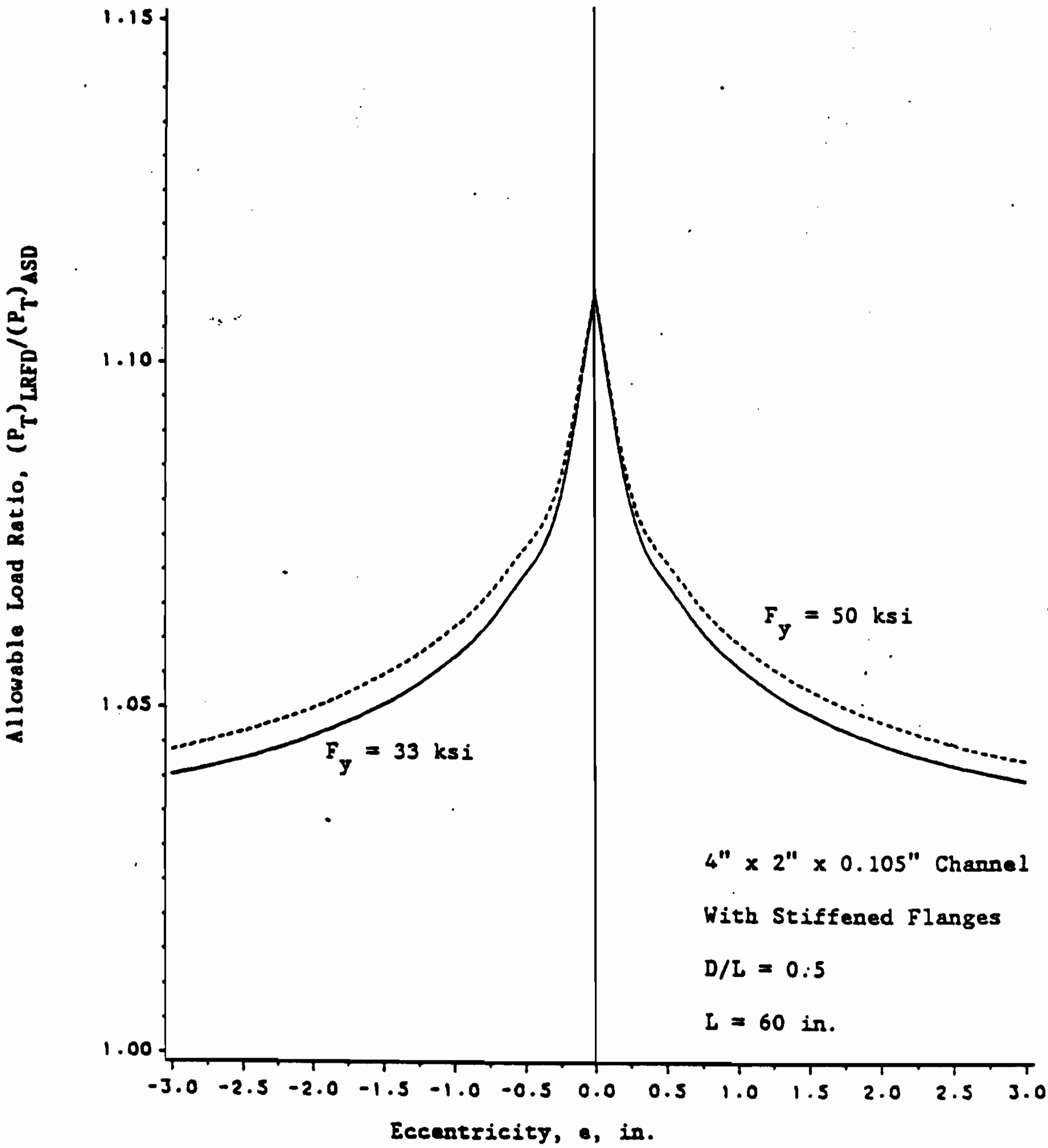

Figure 35 Allowable Load Ratio vs. Eccentricity for Beam-Columns - Case 1

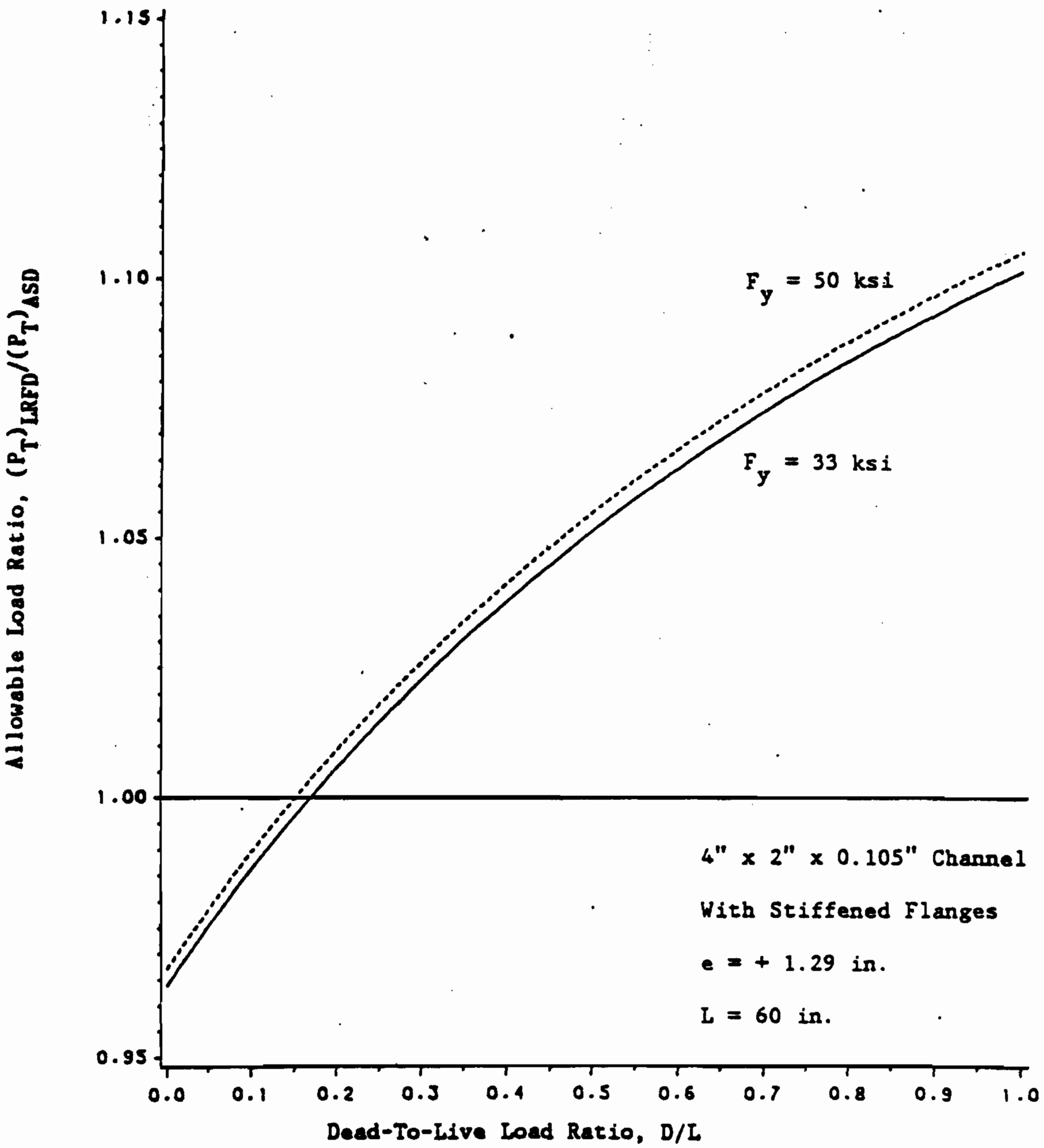

Figure 36 Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case 1

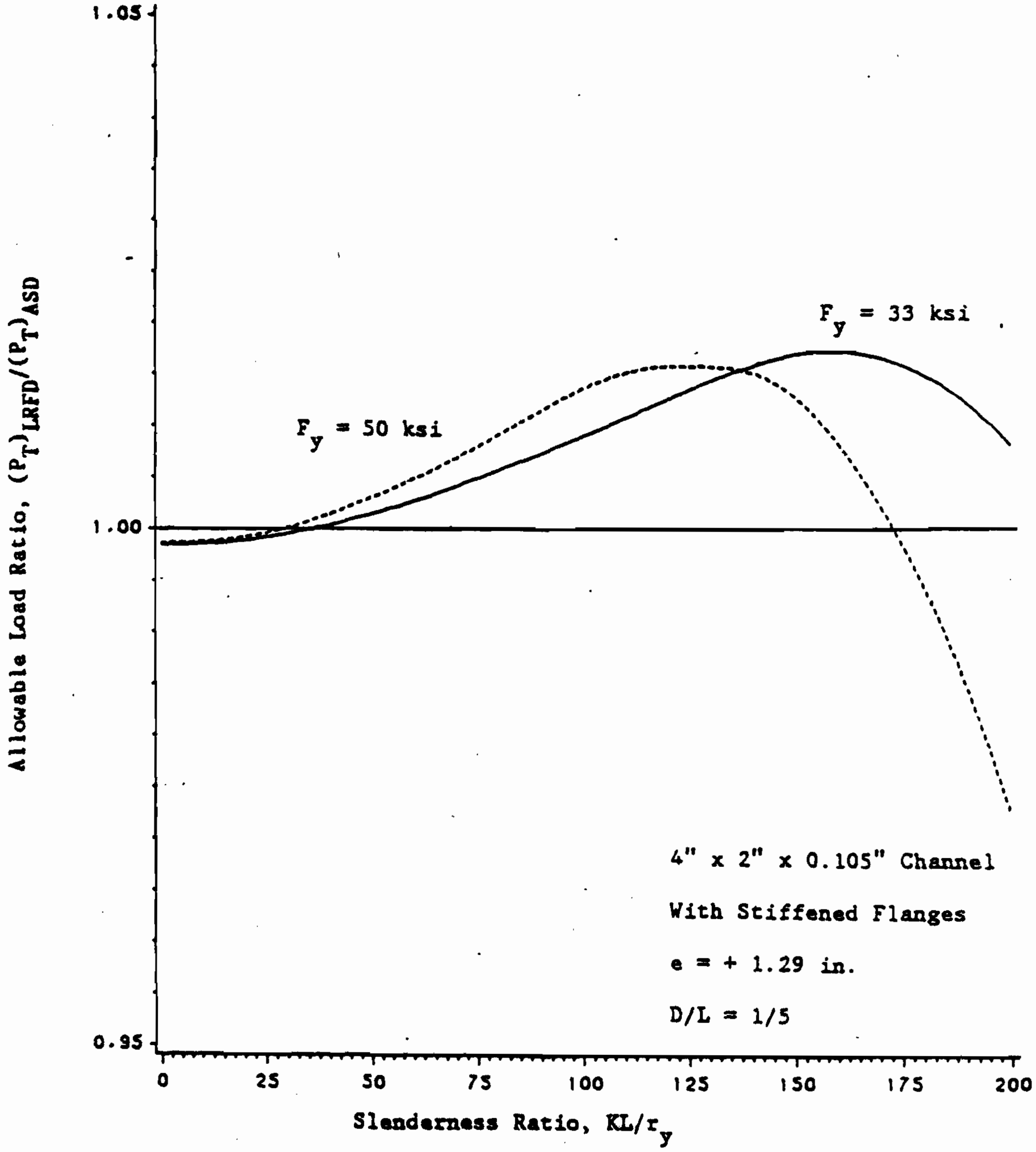

Figure 37 Allowable Load Ratio vs. Slenderness Ratio for Beam-Columns - Case 1

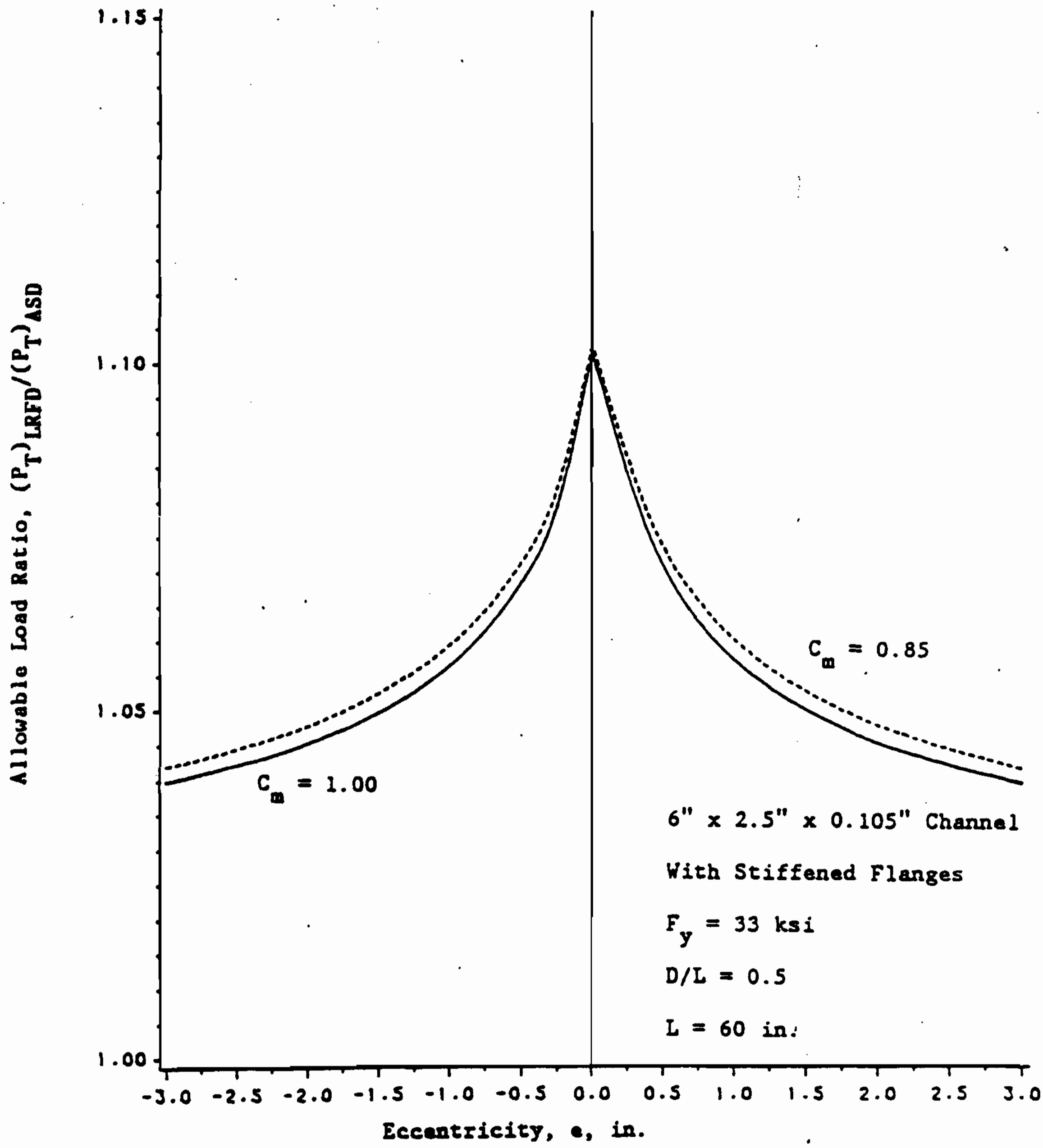

Figure 38 Allowable Load Ratio vs. Eccentricity for Beam-Columns - Case 2

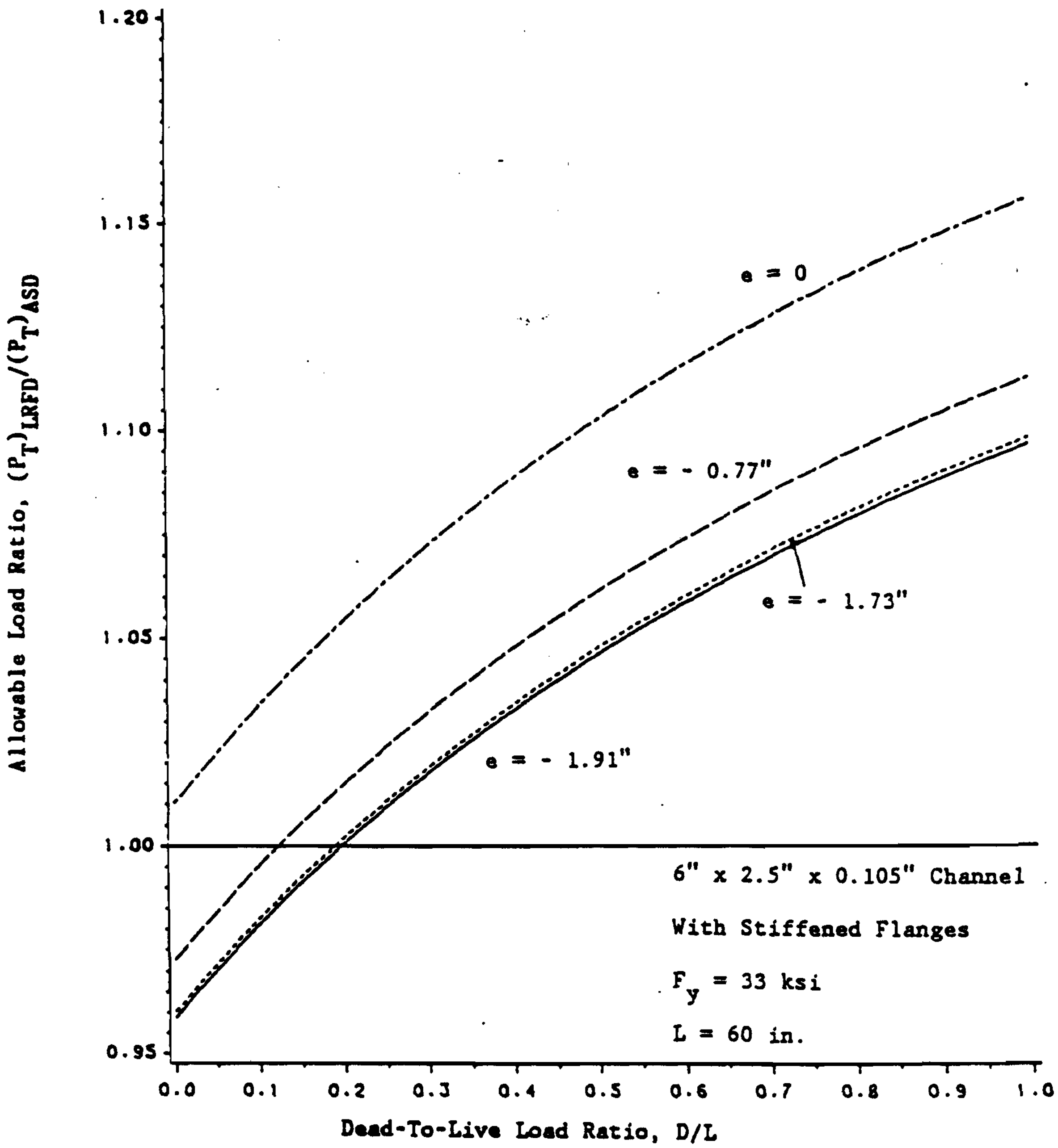

Figure 39 Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case 2

P_{DL} = compressive load due to the nominal axial dead load

P_{LL} = compressive load due to the nominal axial live load

The total unfactored load should be less than or equal to the allowable loads computed from allowable stress design and LRFD. For allowable stress design, the allowable loads are

$$(P_a)_{ASD1} = P_{n1}/\Omega_{st} \tag{7.110}$$

$$(P_a)_{ASD2} = P_{n2}/\Omega_c \tag{7.111}$$

Eq. (7.110) serves to prevent end crushing of the transverse stiffeners while Eq. (7.111) is to prevent column-type buckling of the web-stiffeners.

For LRFD, the allowable axial loads can be computed by using the following equations developed from Eq. (7.4):

$$(P_a)_{LRFD1} = \phi_c P_{n1}(D/L+1)/(1.2D/L+1.6) \tag{7.112}$$

$$(P_a)_{LRFD2} = \phi_c P_{n2}(D/L+1)/(1.2D/L+1.6) \tag{7.113}$$

where

P_{n1} = nominal compression strength for the prevention of end crushing of the transverse stiffeners

P_{n2} = nominal compression strength for the prevention of column-type buckling of the web-stiffeners

In order to study the allowable load ratios, three different cases were considered:

(1) Case 1: $P_{n1} \le P_{n2}$, then Eqs. (7.110) and (7.112) can be used to determine the allowable load ratio as follows:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = \Omega_{st}\phi_c \frac{D/L+1}{1.2D/L+1.6} = 1.7 \frac{D/L+1}{1.2D/L+1.6} \tag{7.114}$$

(2) Case 2: $P_{n1} > P_{n2}$ and $P_{n1}/\Omega_{st} > P_{n2}/\Omega_c$, then Eqs. (7.111) and (7.113) can be used to determine the allowable load ratio as follows:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = \Omega_c \Phi_c \frac{D/L+1}{1.2D/L+1.6} = 0.85\Omega_c \frac{D/L+1}{1.2D/L+1.6} \quad (7.115)$$

where

$$\Omega_c = 5/3+(3/8)R-(1/8)R^3 \quad (7.116)$$

$$R = \sqrt{(F_y/2F_e)} \quad (7.117)$$

(3) Case 3: $P_{n1} > P_{n2}$ and $P_{n1}/\Omega_{st} < P_{n2}/\Omega_c$, then Eqs. (7.110) and (7.113) can be used to determine the allowable load ratio as follows:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = \Omega_{st} \Phi_c \frac{P_{n2}}{P_{n1}} \left[\frac{D/L+1}{1.2D/L+1.6}\right] = 1.7 \frac{P_{n2}}{P_{n1}} \left[\frac{D/L+1}{1.2D/L+1.6}\right] \quad (7.118)$$

Figure 40 shows the allowable load ratio versus dead-to-live load ratio for the compression strength of transverse stiffeners determined by Eq. (7.114). For this case, the LRFD criteria always permit larger allowable loads than the allowable stress design. For D/L = 0.5, the LRFD criteria give an allowable load about 16% greater than the load obtained by using allowable stress design.

Figure 41 shows the allowable load ratio versus dead-to-live load ratio for the compression strength of transverse stiffeners determined by Eq. (7.115). Different curves represent different values of Ω_c. For Ω_c values from 1.67 to 1.92 and D/L = 0.5, the allowable loads determined by LRFD criteria are from 3.2% lower to 11.2% higher than the allowable loads determined by the allowable stress design.

Figure 42 shows the allowable load ratio versus dead-to-live load ratio for the compression strength of transverse stiffeners determined by Eq. (7.118). Different curves represent different values of P_{n2}/P_{n1}.

For P_{n2}/P_{n1} values from 0.835 to 1.0 and D/L = 0.5, the allowable loads determined by LRFD criteria are from 3.2% lower to 16% higher than the allowable loads determined by the allowable stress design.

2. Shear Stiffeners. The unfactored shear force can be calculated for both ASD and LRFD methods by using the following equation.

$$V_T = V_{DL}+V_{LL} \tag{7.119}$$

where

V_T = total unfactored shear force

V_{DL} = shear force due to the nominal dead load

V_{LL} = shear force due to the nominal live load

This total unfactored shear force should be less than or equal to the allowable shear capacity. For allowable stress design, the allowable shear load is

$$(V_a)_{ASD} = V_n/\Omega \tag{7.120}$$

For LRFD, the allowable shear load equation was developed from Eq. (7.4) and is

$$(V_a)_{LRFD} = \phi_v V_n(D/L+1)/(1.2D/L+1.6) \tag{7.121}$$

The allowable shear force, V_a, for allowable stress design is determined from shear yielding with a factor of safety of 1.44, from the critical stress for elastic shear buckling with a factor of safety of 1.71, and from the critical stress for inelastic shear buckling with a factor of safety of 1.67. The limits of the h/t ratio were obtained by equating the formulas for the three shear failure modes for both allowable stress and LRFD criteria. Because each failure mode has a different factor of safety, the h/t limits are slightly different for both design criteria. The allowable shear ratios are:

For $h/t \leq \sqrt{Ek_v/F_y}$ and $\phi_v = 1.0$,

$$\frac{(V_a)_{LRFD}}{(V_a)_{ASD}} = 1.443\phi_v \frac{D/L+1}{1.2D/L+1.6} = 1.443 \frac{D/L+1}{1.2D/L+1.6} \tag{7.122}$$

For $\sqrt{Ek_v/F_y} < h/t \leq 1.38\sqrt{Ek_v/F_y}$ and $\phi_v = 0.90$,

$$\frac{(V_a)_{LRFD}}{(V_a)_{ASD}} = 1.674\phi_v \frac{D/L+1}{1.2D/L+1.6} = 1.507 \frac{D/L+1}{1.2D/L+1.6} \tag{7.123}$$

For $h/t > 1.415\sqrt{Ek_v/F_y}$ and $\phi_v = 0.90$,

$$\frac{(V_a)_{LRFD}}{(V_a)_{ASD}} = 1.712\phi_v \frac{D/L+1}{1.2D/L+1.6} = 1.541 \frac{D/L+1}{1.2D/L+1.6} \tag{7.124}$$

It should be noted that for h/t greater than $1.38\sqrt{Ek_v/F_y}$ and less than $1.415\sqrt{Ek_v/F_y}$, inelastic shear buckling will govern for LRFD.

Figure 43 shows the allowable shear ratio versus dead-to-live load ratio for the three failure modes. For D/L = 0.5, the allowable shears determined according to LRFD may be up to 5% higher than the values obtained from allowable stress design. For D/L < 0.17, LRFD is generally conservative. When D/L > 0.65, LRFD gives larger values of the allowable shear capacity. It can be seen that this figure is identical to Figure 16 which was obtained for the shear strength of the beam webs.

H. COMPARATIVE STUDY OF WALL STUDS AND WALL STUD ASSEMBLIES

1. Wall Studs in Compression. The unfactored load applied to the member can be computed for both design methods by using the following formula:

$$P_T = P_{DL}+P_{LL} \tag{7.125}$$

where

P_T = unfactored compressive load

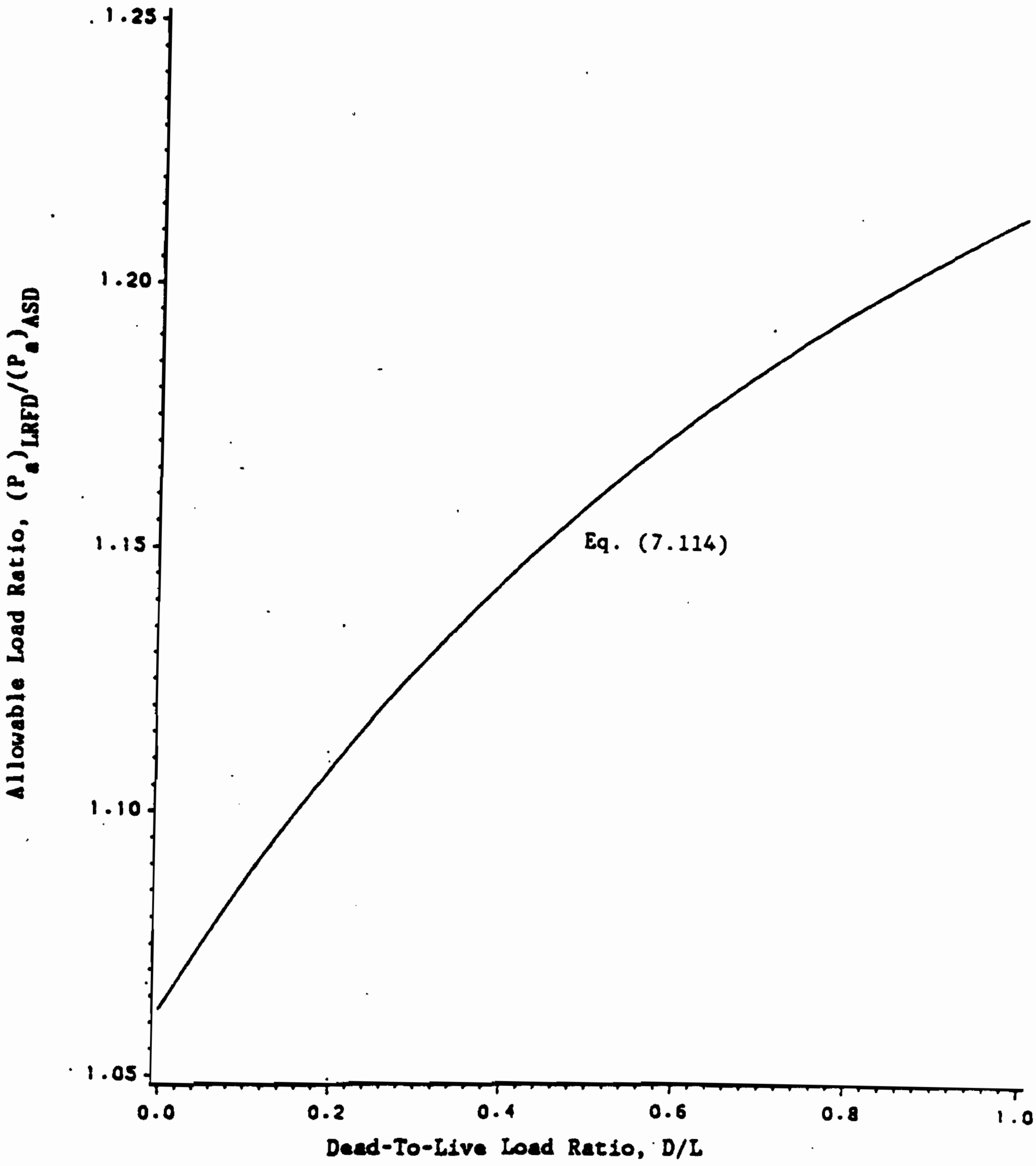

Figure 40 Allowable Load Ratio vs. D/L Ratio for Compression Strength of Transverse Stiffeners-Case 1

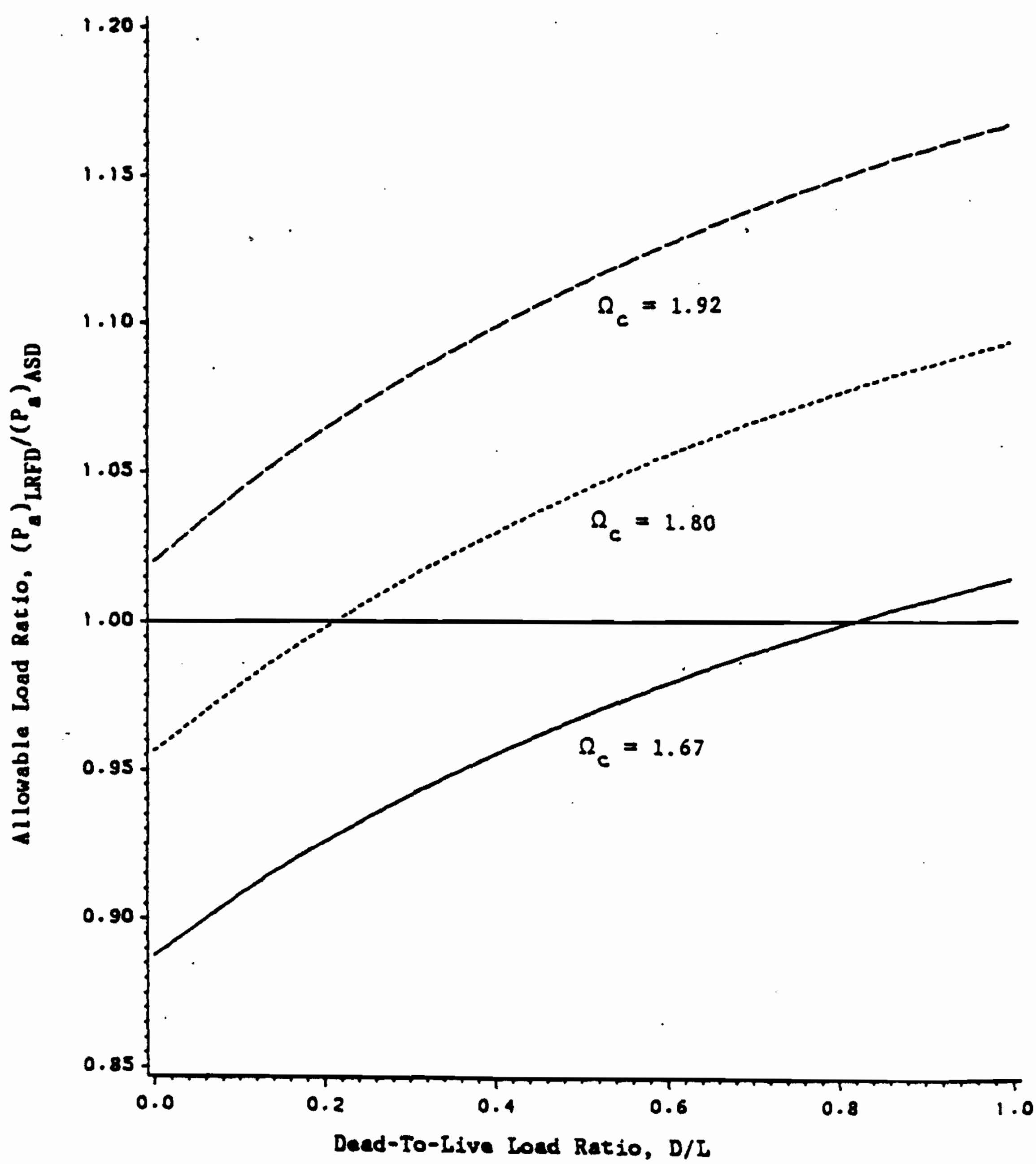

Figure 41 Allowable Load Ratio vs. D/L Ratio for Compression Strength of Transverse Stiffeners-Case 2

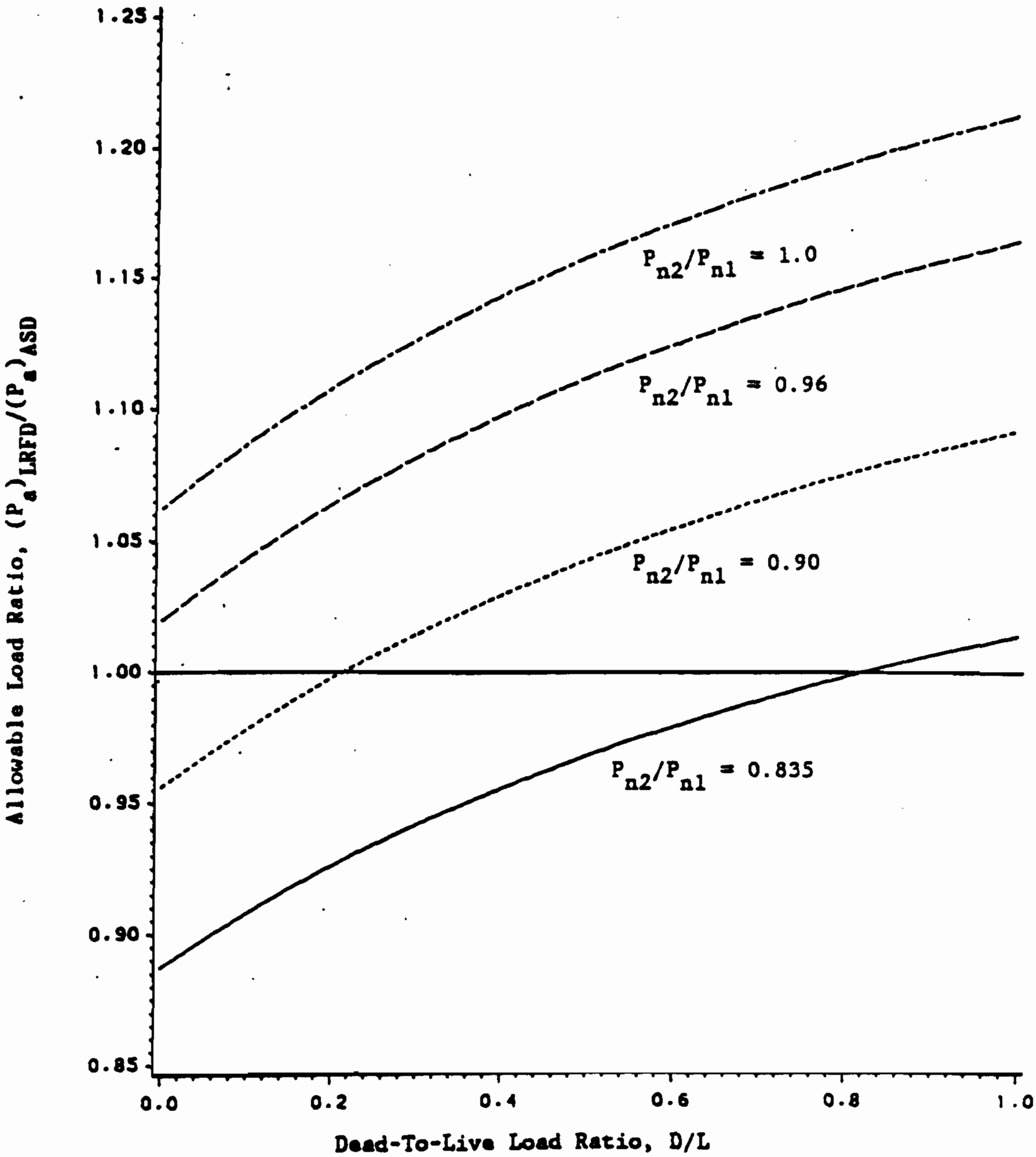

Figure 42 Allowable Load Ratio vs. D/L Ratio for Compression Strength of Transverse Stiffeners-Case 3

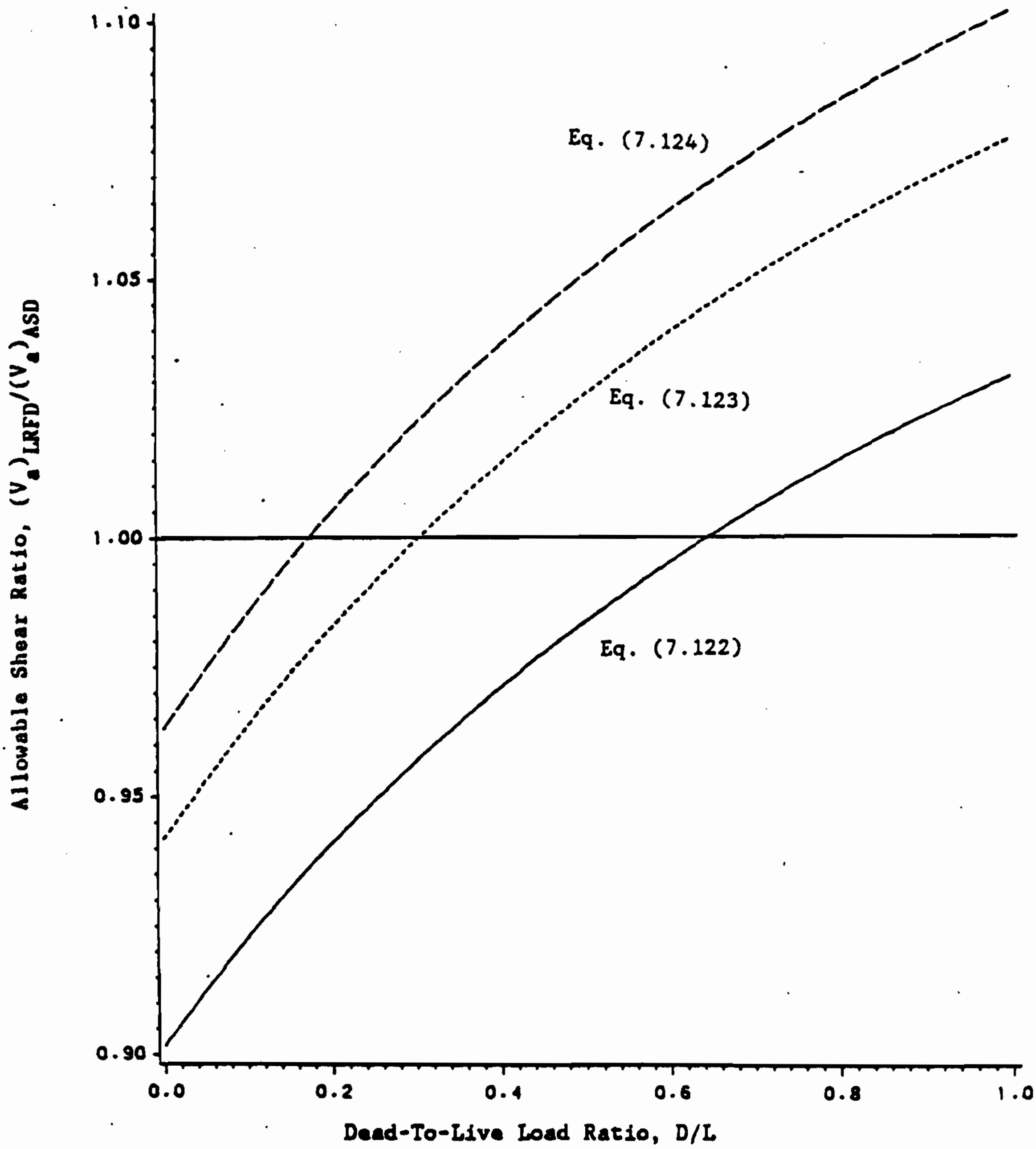

Figure 43 Allowable Shear Ratio vs. D/L Ratio for Shear Strength of Shear Stiffeners

P_{DL} = compressive load due to the nominal axial dead load

P_{LL} = compressive load due to the nominal axial live load

The total unfactored load should be less than or equal to the allowable loads computed from allowable stress design and LRFD. For allowable stress design, the allowable load is

$$(P_a)_{ASD} = P_n/\Omega_c \tag{7.126}$$

For LRFD, the allowable axial load can be computed by using the following equation developed from Eq. (7.4):

$$(P_a)_{LRFD} = \phi_c P_n(D/L+1)/(1.2D/L+1.6) \tag{7.127}$$

Then, the allowable load ratio can be determined as follows:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = \frac{\phi_c P_n}{P_n/\Omega_c}\left[\frac{D/L+1}{1.2D/L+1.6}\right] = 0.85\Omega_c \frac{D/L+1}{1.2D/L+1.6} \tag{7.128}$$

For fully effective sections having wall thickness greater than 0.09 in. and $F_e > F_y/2$,

$$\Omega_c = 5/3+(3/8)R-(1/8)R^3 \tag{7.129}$$

Therefore, the allowable load ratio is

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 0.85\left(\frac{5}{3} + \frac{3}{8}R - \frac{1}{8}R^3\right)\frac{D/L+1}{1.2D/L+1.6} \tag{7.130}$$

For all other cases, Ω_c = 1.92 = 23/12, therefore the allowable load ratio is

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 0.85(23/12)\frac{D/L+1}{1.2D/L+1.6} = 1.629\frac{D/L+1}{1.2D/L+1.6} \tag{7.131}$$

Figure 44 shows the allowable load ratio versus dead-to-live load ratio for the wall studs used to develop Eq. (7.131). For this case, the LRFD criteria always permit larger allowable loads than the allowable stress design. For D/L = 0.5, the LRFD criteria gives an allowable load about 11% greater than the load obtained by using allowable stress design.

Figure 45 shows the allowable load ratio versus dead-to-live load ratio for the wall studs used to develop Eq. (7.130). Different curves represent different values of R. For R varies from 0 to 1 and D/L = 0.5, the allowable loads determined by LRFD criteria are from 3.2% lower to 16% higher than the allowable loads determined by the allowable stress design.

2. Wall Studs in Bending. The unfactored moment can be calculated by using Eq. (7.132) for both methods (ASD and LRFD).

$$M_{TL} = M_{DL}+M_{LL} \tag{7.132}$$

where

M_{TL} = total unfactored moment

M_{DL} = moment due to the nominal dead load

M_{LL} = moment due to the nominal live load

For allowable stress design, the allowable moment is determined from the nominal section strength with a factor of safety of 1.67. Therefore, the allowable moment for beams is

$$(M_a)_{ASD} = M_n/\Omega_f = M_n/1.67 \tag{7.133}$$

For LRFD, the allowable moment can be computed by using the following equation developed from Eq. (7.4).

$$(M_a)_{LRFD} = \phi_b M_n(D/L+1)/(1.2D/L+1.6) \tag{7.134}$$

The ratio of the allowable moments is

$$\frac{(M_a)_{LRFD}}{(M_a)_{ASD}} = 1.67\phi_b \frac{D/L+1}{1.2D/L+1.6} \tag{7.135}$$

For sections with stiffened or partially stiffened compression flanges, $\phi_b = 0.95$

$$\frac{(M_a)_{LRFD}}{(M_a)_{ASD}} = 1.58 \frac{D/L+1}{1.2D/L+1.6} \tag{7.136}$$

Figure 46 shows the allowable moment ratio versus dead-to-live load ratio for wall studs with stiffened compression flanges. For D/L = 1/25 both design methods will give the same value of allowable moment. However, LRFD will be conservative for D/L < 1/25 and unconservative for D/L > 1/25 as compared with the allowable stress design method.

For sections with unstiffened compression flanges, $\phi_b = 0.90$

$$\frac{(M_a)_{LRFD}}{(M_a)_{ASD}} = 1.50 \frac{D/L+1}{1.2D/L+1.6} \tag{7.137}$$

Figure 47 shows the allowable moment ratio versus the dead-to-live load ratio for this case. The two design methods give the same value for D/L = 1/3. For D/L = 0.5, the allowable moment based on LRFD is about 2.3% larger than the value obtained from allowable stress design. When the dead-to-live load ratio for cold-formed steel is less than 1/3, the LRFD criteria are found to be conservative for sections with unstiffened compression flanges as compared with the allowable stress design method.

3. Wall Studs With Combined Axial Load and Bending. Wall studs made by channel sections bending about the x-axis were considered. A typical design example was selected and the allowable axial loads were calculated by using three interaction equations for each design method. The example used a wall stud with equal moments applied to each end so that the member is bent in single curvature. Since the end moments are independent of the axial load, the ratio of the unfactored applied moment to the nominal

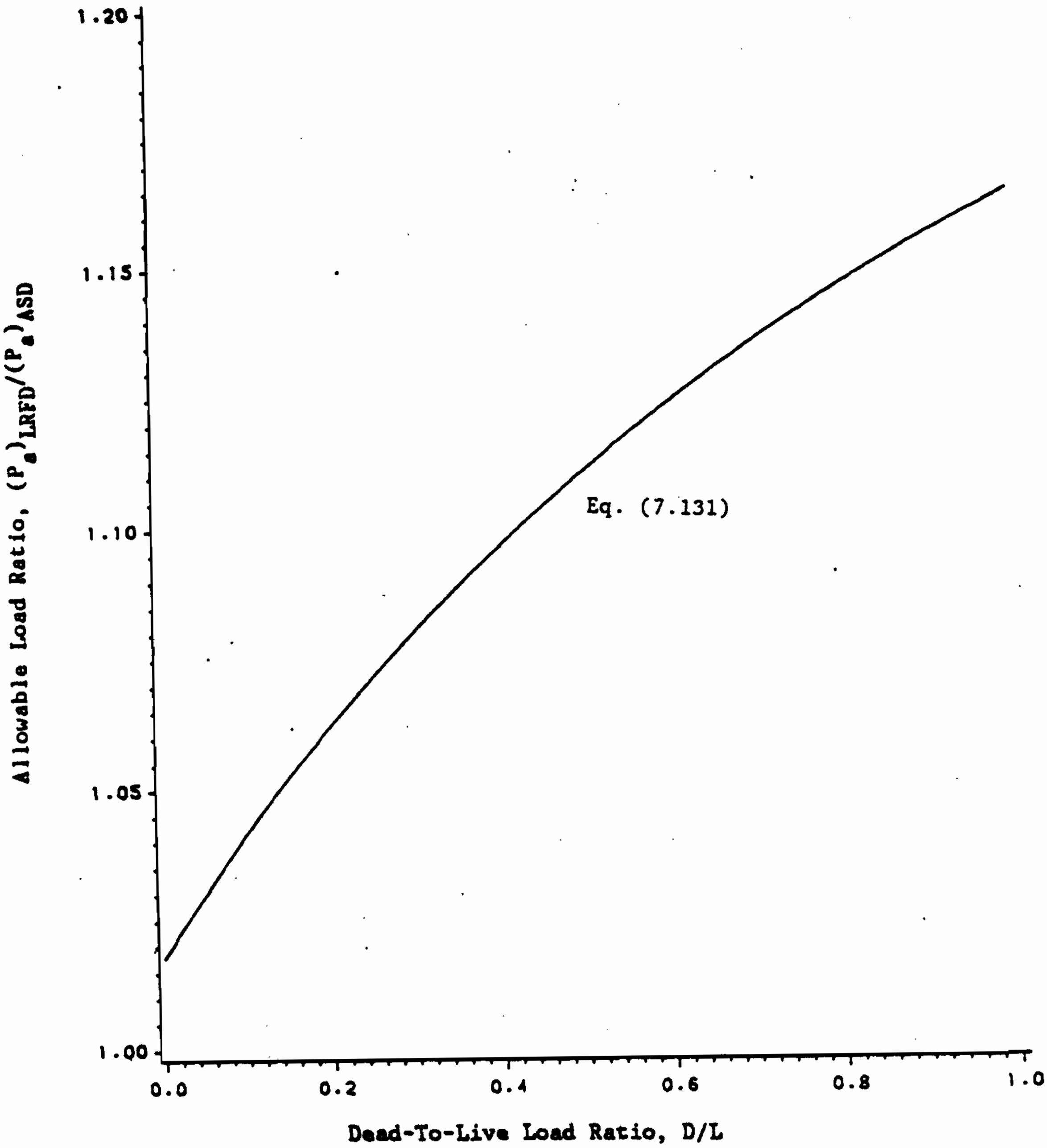

Figure 44 Allowable Load Ratio vs. D/L Ratio for Wall Studs in Compression-Case 1

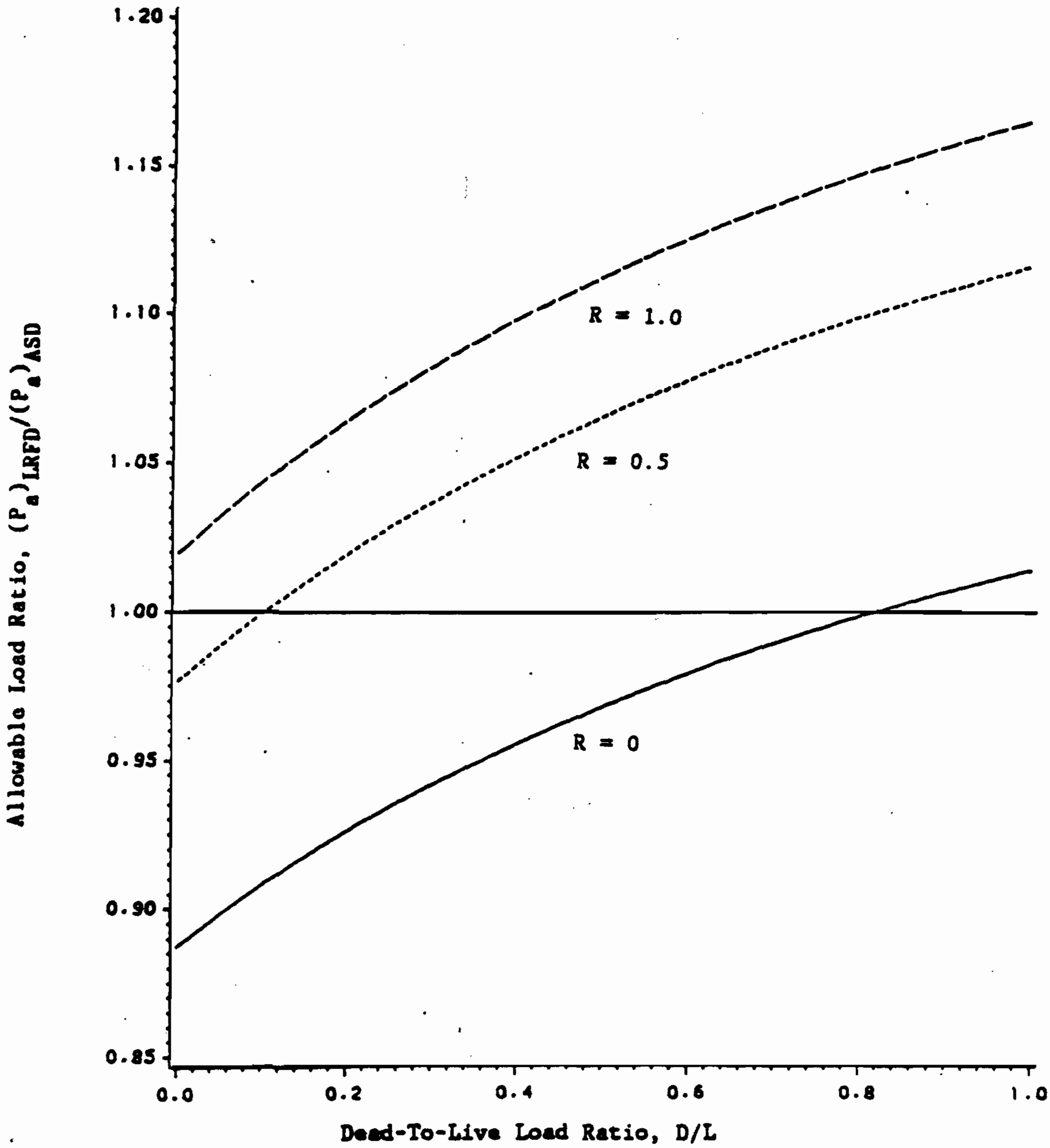

Figure 45 Allowable Load Ratio vs. D/L Ratio for Wall Studs in Compression-Case 2

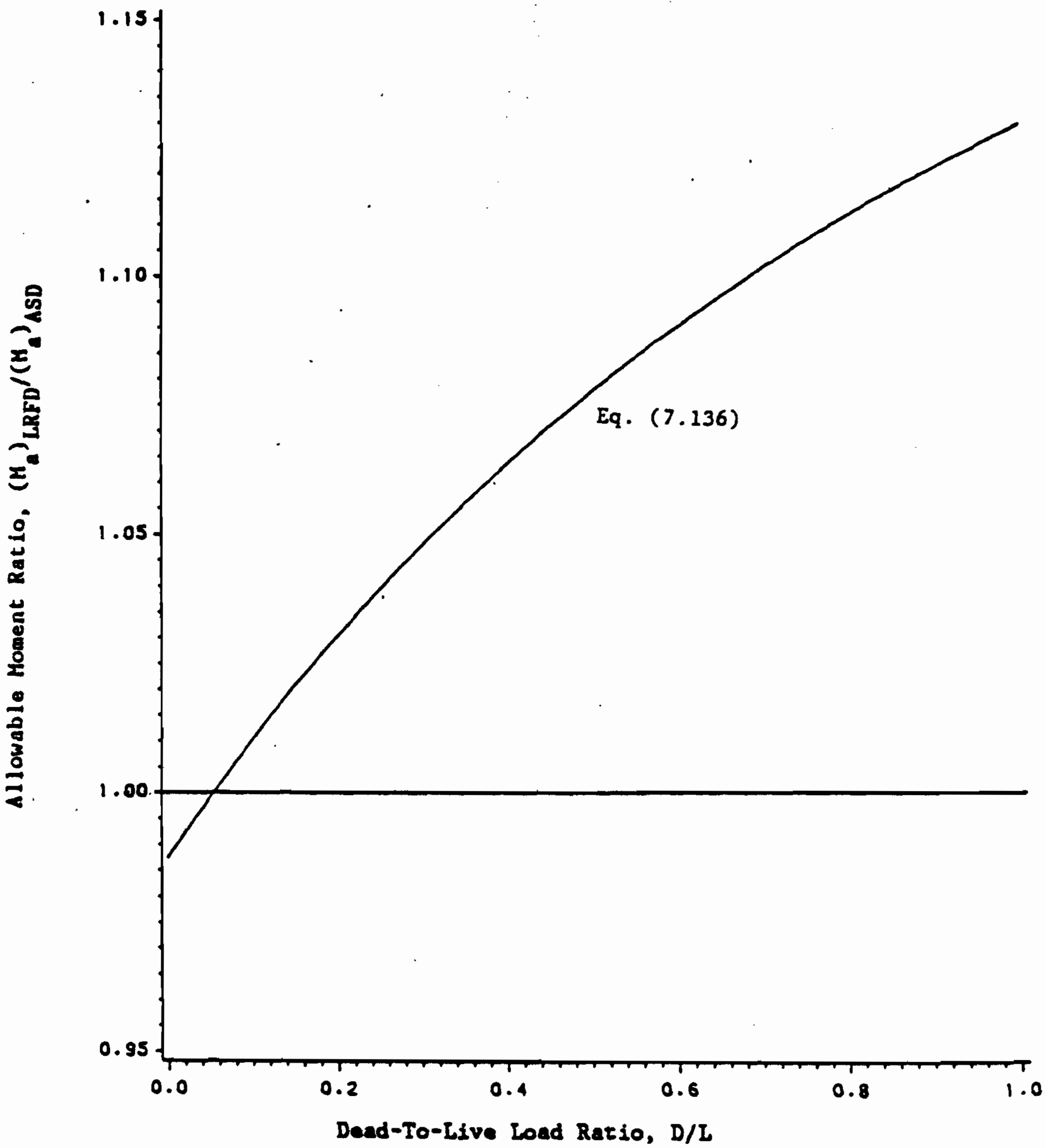

Figure 46 Allowable Moment Ratio vs. D/L Ratio for Wall Studs in Bending-Case 1

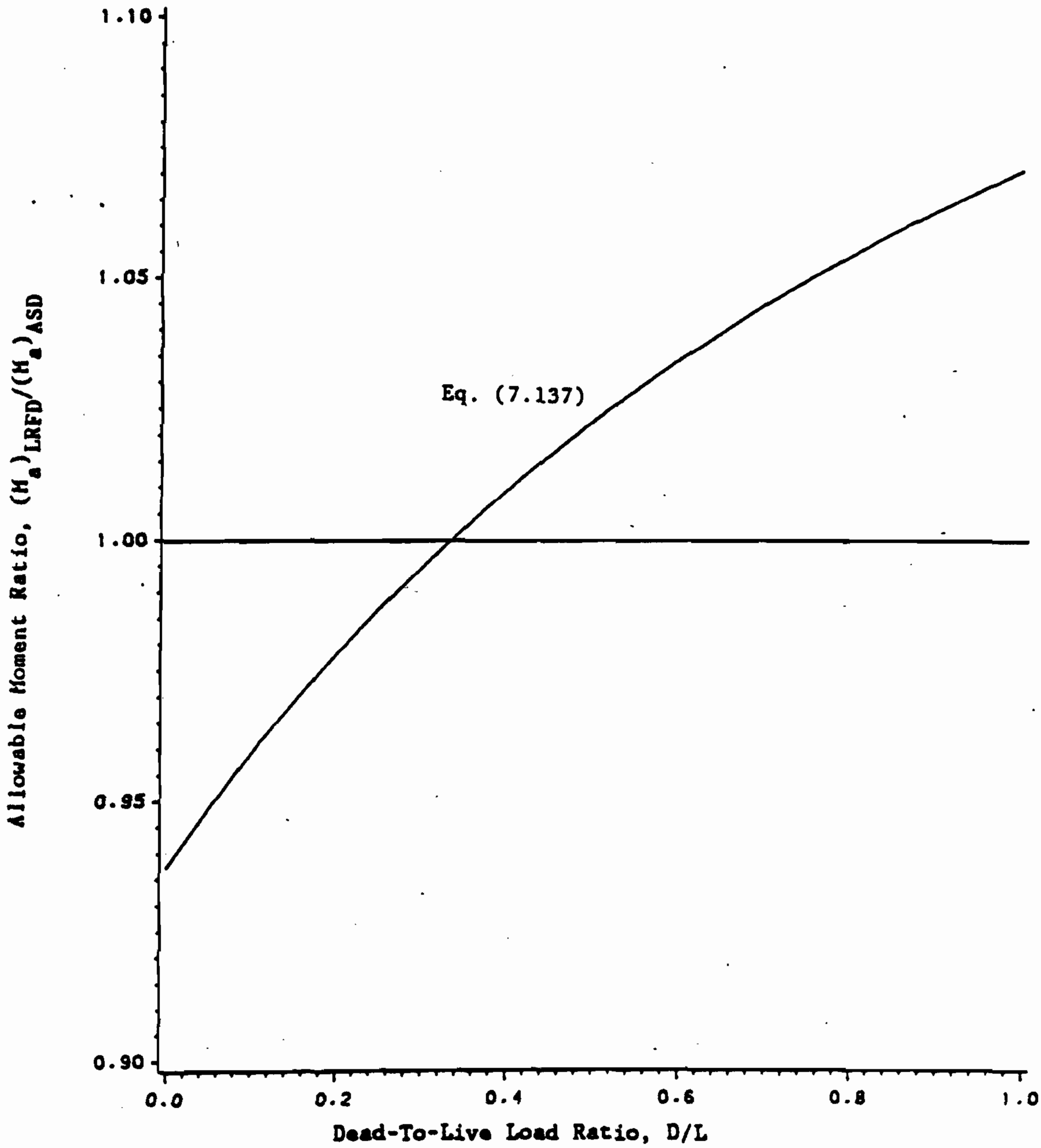

Figure 47 Allowable Moment Ratio vs. D/L Ratio for Wall Studs in Bending-Case 2

moment capacity based on section strength, M_T/M_{no}, was considered to be a parameter in the equations for determining the allowable loads.

For allowable stress design the allowable axial loads were computed as follows:

$$\frac{P}{P_a} = \frac{P_T}{P_n/\Omega_c} = \frac{\Omega_c P_T}{P_n} \qquad (7.138)$$

$$\frac{M}{M_{ao}} = \frac{M_T}{0.6M_{no}} = \frac{M_T/M_{no}}{0.6} \qquad (7.139)$$

where

P_T = applied unfactored axial load

M_T = applied unfactored bending moment at each end of the member

Ω_c = factor of safety of axially loaded compression members

The use of Eqs. (7.138) and (7.139) results in the following interaction formula:

$$\frac{\Omega_c P_T}{P_n} + \frac{C_m(M_T/M_{no})}{0.6(1-\Omega_c P_T/P_{cr})} = 1.0 \qquad (7.140)$$

By solving for P_T in the first term of Eq. (7.140), the following equation for allowable load is obtained :

$$(P_T)_{ASD1} = \left[1- \frac{C_m(M_T/M_{no})}{0.6(1-\Omega_c P_T/P_{cr})}\right] \frac{P_n}{\Omega_c} \qquad (7.141)$$

Equation (7.141) is based the failure at the midlength of the beam-column and requires a solution by iterations.

The following expression was used to solve for the allowable load based on the failure at the braced points:

$$\frac{P}{P_{ao}} = \frac{P_T}{P_{no}/\Omega_c} = \frac{\Omega_c P_T}{P_{no}} \qquad (7.142)$$

The use of Eqs. (7.142) and (7.139) results in the following interaction formula:

$$\frac{\Omega_c P_T}{P_{no}} + \frac{(M_T/M_{no})}{0.6} = 1.0 \tag{7.143}$$

By solving for P_T in Eq. (7.143), the following equation for allowable load is obtained :

$$(P_T)_{ASD2} = \left[1 - \frac{(M_T/M_{no})}{0.6}\right] \frac{P_{no}}{\Omega_c} \tag{7.144}$$

Equation (7.144) is based on the failure at the braced points.

When $P/P_a \leq 0.15$, the following interaction formula can be written by using Eqs. (7.138) and (7.139) :

$$\frac{\Omega_c P_T}{P_n} + \frac{M_T/M_{no}}{0.6} = 1.0 \tag{7.145}$$

By solving for P_T in Eq. (7.145), the following equation for allowable load is obtained :

$$(P_T)_{ASD3} = \left[1 - \frac{M_T/M_{no}}{0.6}\right] \frac{P_n}{\Omega_c} \tag{7.146}$$

Equation (7.146) is based on the flexural failure when the effect of the secondary moment is neglected.

For LRFD, the allowable axial loads were computed in accordance with Eq. (7.4) as follows :

$$\frac{P_u}{\phi_c P_n} = \frac{1.2D/L+1.6}{D/L+1} \left[\frac{P_T}{\phi_c P_n}\right] \tag{7.147}$$

$$\frac{M_u}{\phi_b M_{no}} = \frac{1.2D/L+1.6}{D/L+1} \left[\frac{M_T/M_{no}}{\phi_b}\right] \tag{7.148}$$

$$\frac{P_u}{\phi_c P_E} = \frac{1.2D/L+1.6}{D/L+1}\left[\frac{P_T}{\phi_c P_E}\right] \tag{7.149}$$

The use of Eqs. (7.147), (7.148), and (7.149) results in the following interaction formula:

$$\frac{1.2D/L+1.6}{D/L+1}\left\{\frac{P_T}{\phi_c P_n} + \frac{C_m(M_T/M_{no})}{\phi_b[1-(1.2D/L+1.6)P_T/(D/L+1)\phi_c P_E]}\right\} = 1.0 \tag{7.150}$$

By solving for P_T in the first term of Eq. (7.150), the following equation for allowable load is obtained :

$$(P_T)_{LRFD1} = \left\{\frac{D/L+1}{1.2D/L+1.6} - \frac{C_m(M_T/M_{no})}{\phi_b[1-(1.2D/L+1.6)P_T/(D/L+1)\phi_c P_E]}\right\}\phi_c P_n \tag{7.151}$$

Equation (7.151) is based on the flexural failure at the midlength of the beam-column and requires a solution by iterations.

The following expression was used to solve for the allowable load based on the failure at the braced points:

$$\frac{P_u}{\phi_c P_{no}} = \frac{1.2D/L+1.6}{D/L+1}\left[\frac{P_T}{\phi_c P_{no}}\right] \tag{7.152}$$

The use of Eqs. (7.152) and (7.148) results in the following interaction formula:

$$\frac{1.2D/L+1.6}{D/L+1}\left[\frac{P_T}{\phi_c P_{no}} + \frac{(M_T/M_{no})}{\phi_b}\right] = 1.0 \tag{7.153}$$

By solving for P_T in Eq. (7.153), the following equation for allowable load is obtained :

$$(P_T)_{LRFD2} = \left[\frac{D/L+1}{1.2D/L+1.6} - \frac{(M_T/M_{no})}{\phi_b}\right]\phi_c P_{no} \tag{7.154}$$

Equation (7.154) is based on the failure at the braced points.

When $P_u/(\phi_c P_n) \leq 0.15$, the following interaction formula can be written by using Eqs. (7.147) and (7.148) :

$$\frac{1.2D/L+1.6}{D/L+1}\left[\frac{P_T}{\phi_c P_n} + \frac{(M_T/M_{no})}{\phi_b}\right] = 1.0 \qquad (7.155)$$

By solving for P_T in Eq. (7.155), the following equation for allowable load is obtained :

$$(P_T)_{LRFD3} = \left[\frac{D/L+1}{1.2D/L+1.6} - \frac{(M_T/M_{no})}{\phi_b}\right]\phi_c P_n \qquad (7.156)$$

Equation (7.156) is based on the flexural failure when the effect of the secondary moment is neglected.

Equations (7.141), (7.144), and (7.146), for determining the allowable axial load based on allowable stress design, and Eqs. (7.151), (7.154), and (7.156), for determining the allowable axial load based on LRFD, are very complex and utilize iterations with multiple variables. The allowable load ratios, $(P_T)_{LRFD}/(P_T)_{ASD}$, for various lengths combined with different applied end moment ratios, M_T/M_{no}, with respect to the bending strength of the member were studied. The wall studs used in this study use 1/2 in. gypsum board with No. 6 type S-12 self-drilling screws at 12 in. spacing and the spacing of the channel is 24 in.. Typical channel sections and their section properties used in this study were obtained from Tables 1 and 2 of Part V of the AISI Cold-Formed Steel Design Manual.

A channel section (7 in. x 2.75 in. x 0.075 in.) with stiffened flanges was studied with a yield point of 50 ksi. Figure 48 shows the allowable load ratio versus dead-to-live load ratio for a 15 ft length with various end moment ratios, M_T/M_{no}. For a D/L ratio around 0.05, the

LRFD criteria give an allowable load that is about 3% more than the value computed from allowable stress design for all end moment ratios indicated in the figure. For other values of the D/L ratio, the difference between the allowable loads computed by using these two methods depends on the end moment ratio as shown in Figure 48. For D/L > 0.05, the larger the end moment ratio, the higher the allowable load ratio. For example, for D/L = 0.5, the $(P_T)_{LRFD}/(P_T)_{ASD}$ ratios are 1.202 and 1.131 for M_T/M_{no} = 0.3 and 0.1, respectively.

Figure 49 shows the relationship between allowable load ratio and dead-to-live load ratio for end moment ratio of 0.2. The different curves in the figure represent different lengths of the 7 in. x 2.75 in. x 0.075 in. channel section. With end moment ratio of 0.2 and D/L = 0.5, ASD would provide conservative values up to 16.2%, for effective lengths equal to 10 ft, 12 ft, 15 ft, and 20 ft, as compared with the LRFD method. It can also be seen that effective length has a negligible effect on the allowable load ratio.

A shallower channel section (4 in. x 2 in. x 0.075 in.) with stiffened flanges was also studied for an effective length of 10 ft. Figure 50 shows the allowable load ratio versus dead-to-live load ratio for various end moment ratios. The curves without star symbols are for F_y = 33 ksi and the curves with star symbols are for F_y = 50 ksi. They are the same as those shown in Figure 48 for the 7 in. deep channel section. For this case, the yield point of steel would not affect the allowable load ratio. For D/L = 0.5 and M_T/M_{no} = 0.1, the allowable load computed from LRFD is 13.4% greater than the value determined from allowable stress design. However, for D/L = 0.5 and M_T/M_{no} = 0.3, the allowable load computed from LRFD is 20.7% higher than the value computed from allowable stress design.

The curves with and without star symbols in Figure 51 are for C_m = 0.85 and 1.0, respectively, and for F_y = 33 ksi. The value of 0.85 is used for unbraced wall studs and wall studs 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 Figure 51. It can be seen that for D/L < 0.05, the allowable load ratios computed for C_m = 0.85 are larger than those for C_m = 1.0.

Channel sections with unstiffened flanges were studied in a similar manner. In general, the allowable load ratios computed for channels with unstiffened flanges (ϕ_b = 0.90) are smaller than those computed for channels with stiffened flanges (ϕ_b = 0.95). Detailed information can be found in Reference 36.

I. COMPARATIVE STUDY OF WELDED CONNECTIONS

The allowable load per weld for allowable stress design is $(P_a)_{ASD}$ computed from the following equation:

$$(P_a)_{ASD} = P_n/\Omega_w \tag{7.157}$$

where

Ω_w = factor of safety for arc welded connections

= 2.50

For the LRFD criteria, the allowable load per weld can be calculated from the following equation developed from Eq. (7.4):

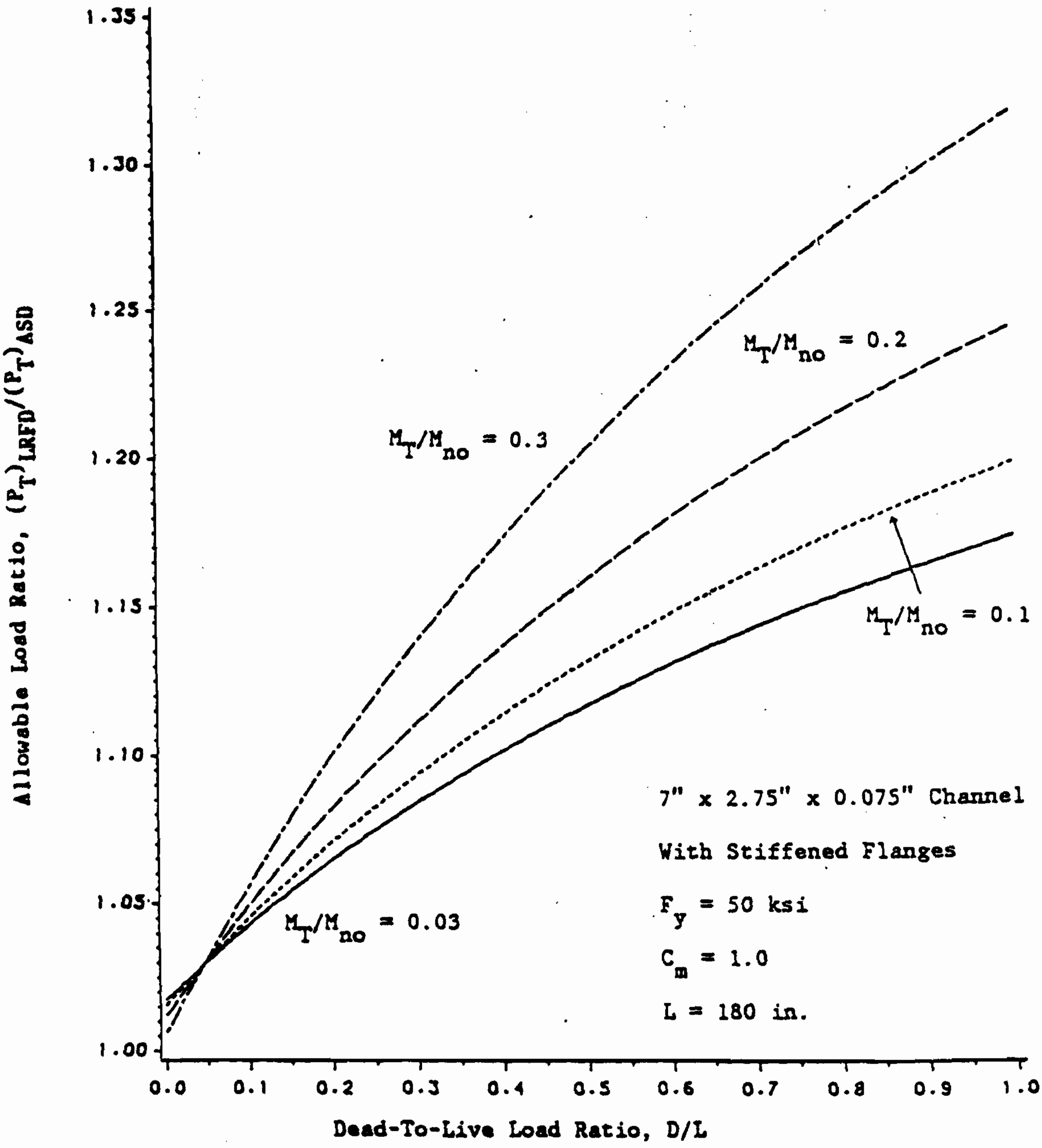

Figure 48 Allowable Load Ratio vs. D/L Ratio for Wall Studs With Combined Axial Load and Bending-Case A

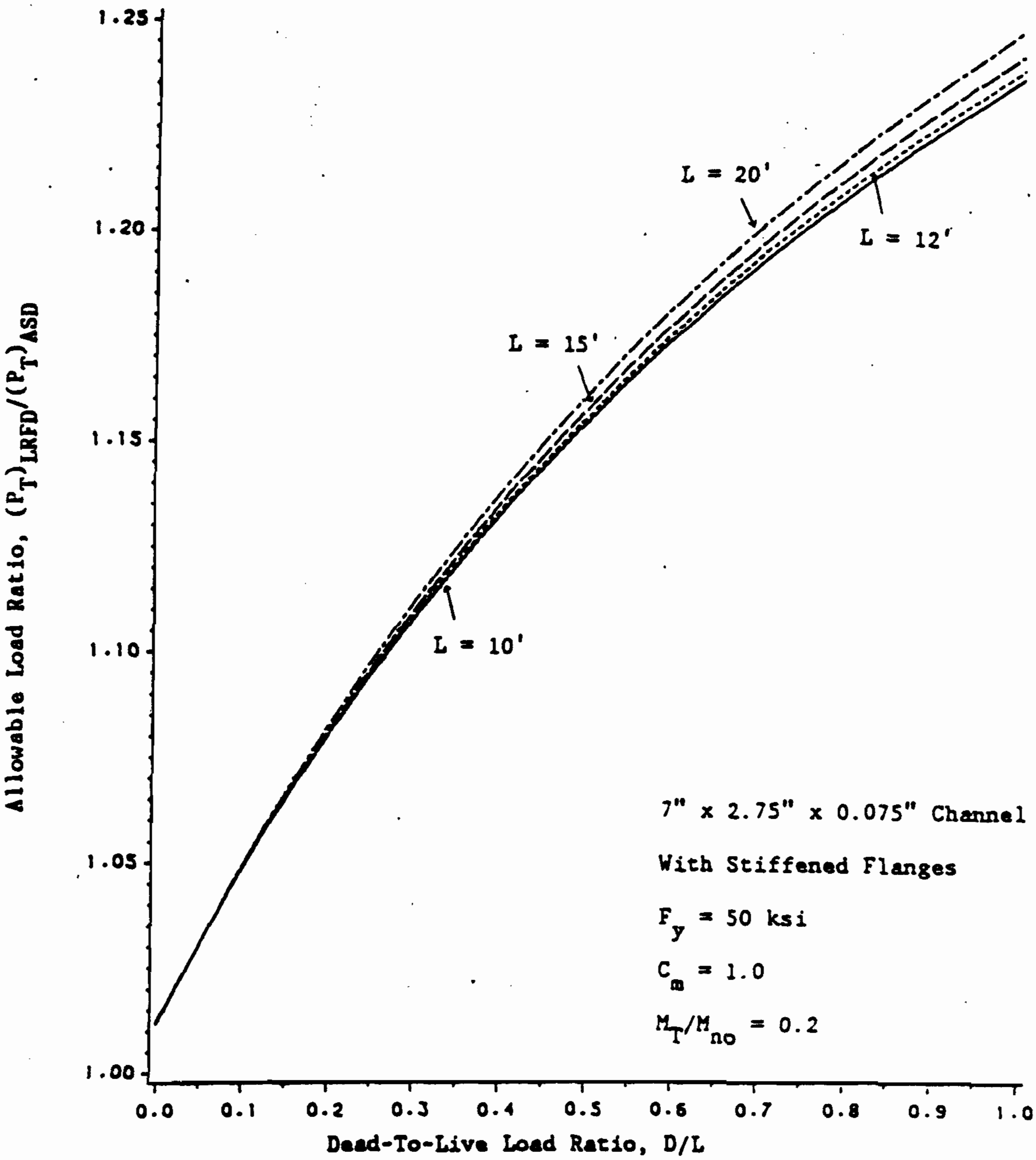

Figure 49 Allowable Load Ratio vs. D/L Ratio for Wall Studs With Combined Axial Load and Bending-Case B

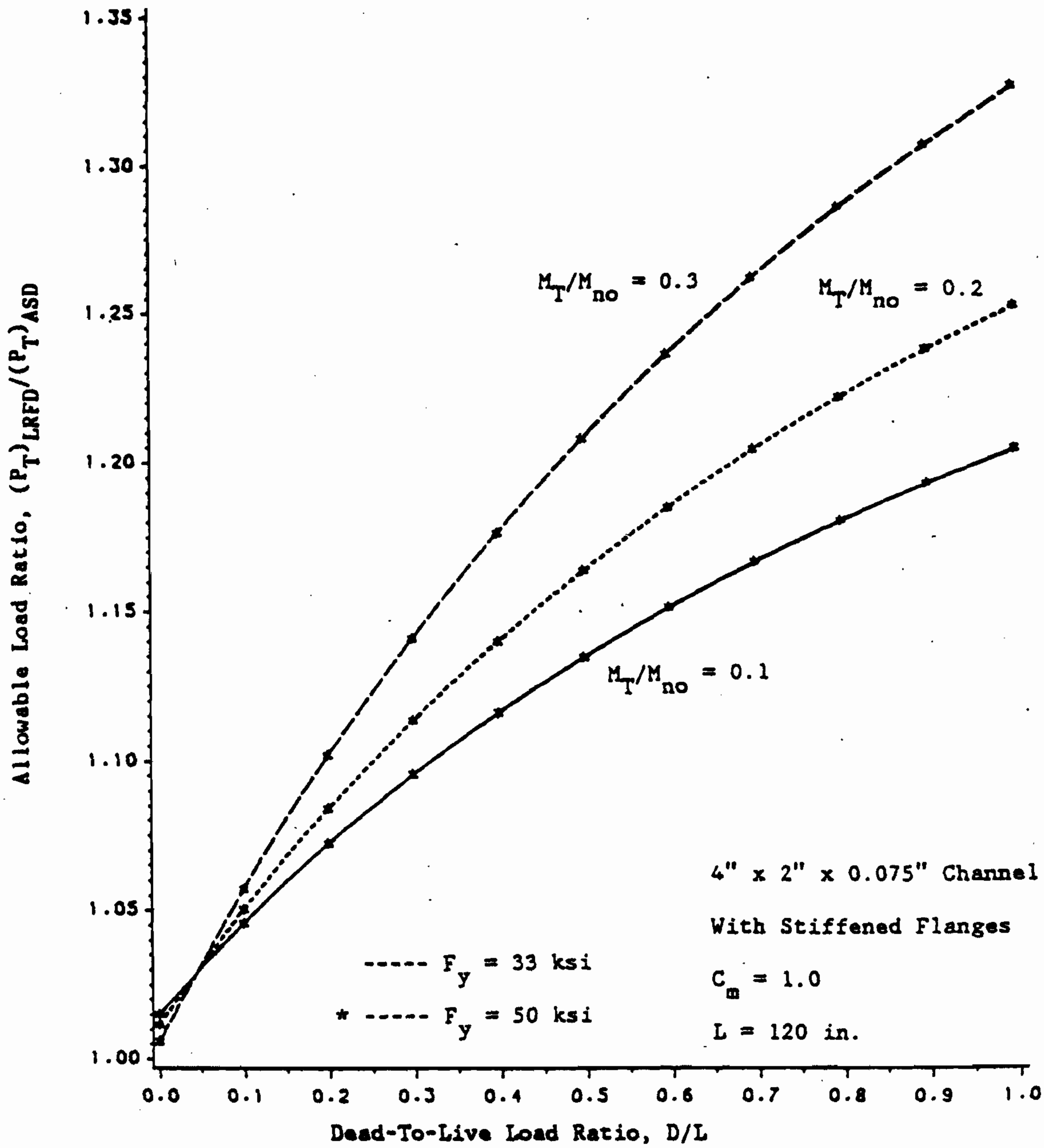

Figure 50 Allowable Load Ratio vs. D/L Ratio for Wall Studs With Combined Axial Load and Bending-Case C

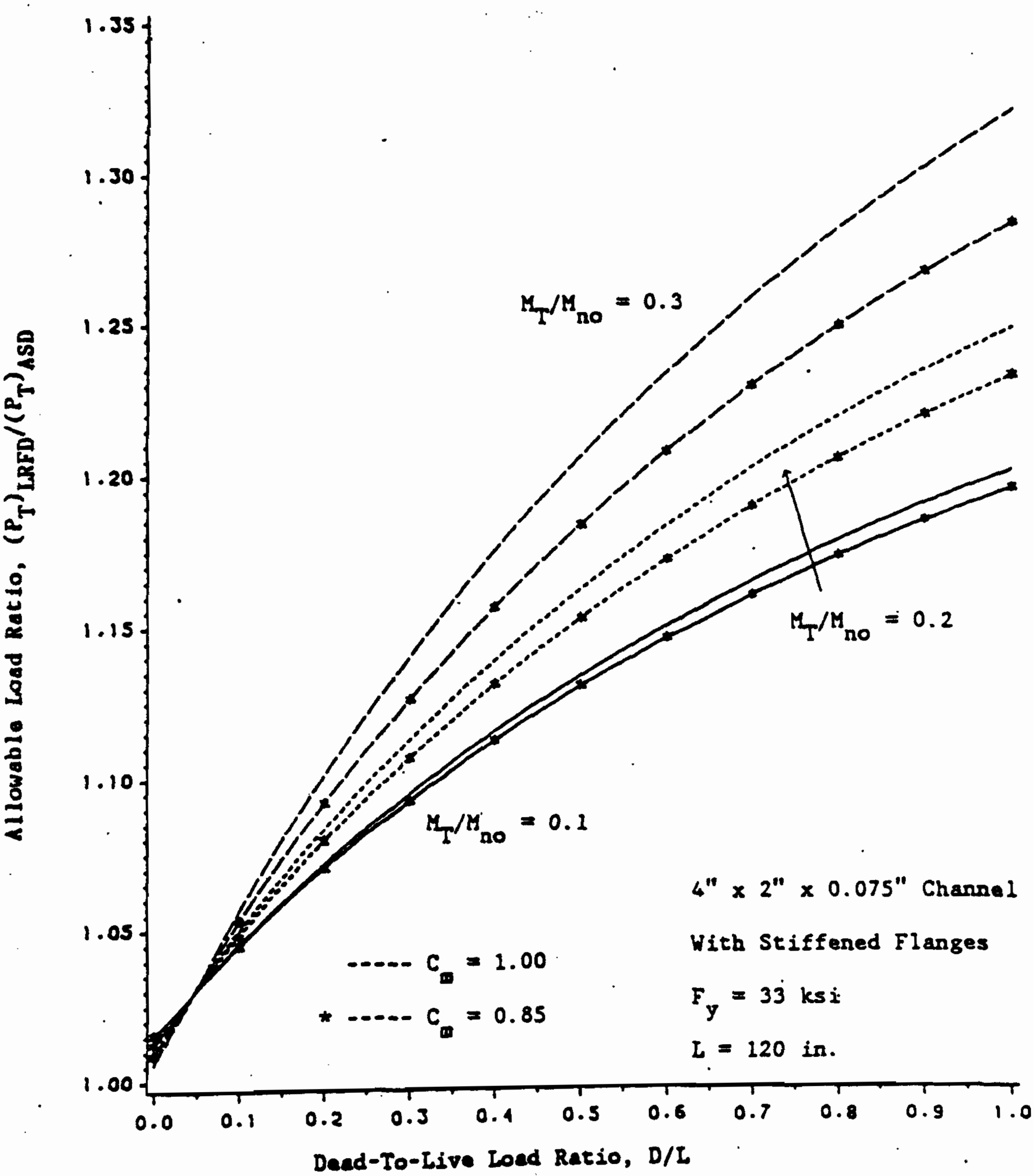

Figure 51 Allowable Load Ratio vs. D/L Ratio for Wall Studs With Combined Axial Load and Bending-Case D

$$(P_a)_{LRFD} = \phi P_n(D/L+1)/(1.2D/L+1.6) \tag{7.158}$$

1. Arc Spot Welds. For the determination of nominal shear strength based on shearing of the welds, Eq. (5.152) is used in the LRFD criteria. For allowable stress design, the nominal shear strength is determined with a coefficient of 0.625, in lieu of 0.589, in Eq. (5.152). Therefore, the allowable load ratio based on shearing of arc spot welds and $\phi = 0.60$ is as follows:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = \frac{0.589}{0.625}(2.5\phi)\frac{D/L+1}{1.2D/L+1.6} = 1.414\frac{D/L+1}{1.2D/L+1.6} \tag{7.159}$$

Figure 52 shows the allowable load ratio versus dead-to-live load ratio determined from Eq. (7.159) for weld shear failure of arc spot welds. For D/L = 0.5, the allowable load per spot determined from the LRFD criteria is 3.6% less than the value obtained from allowable stress design. As shown in the figure, LRFD is conservative for shear failure in arc spot welds for D/L < 0.9.

Equations (5.153), (5.154), and (5.155) are based on failure in the plate, and are used in both ASD and LRFD criteria. The allowable load ratios for plate failure are as follows:

For $(d_a/t) \le 0.815\sqrt{(E/F_u)}$ and $\phi = 0.60$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.5\phi\frac{D/L+1}{1.2D/L+1.6} = 1.50\frac{D/L+1}{1.2D/L+1.6} \tag{7.160}$$

For $0.815\sqrt{(E/F_u)} < (d_a/t) < 1.397\sqrt{(E/F_u)}$ and $\phi = 0.50$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.5\phi\frac{D/L+1}{1.2D/L+1.6} = 1.25\frac{D/L+1}{1.2D/L+1.6} \tag{7.161}$$

For $(d_a/t) \ge 1.397\sqrt{(E/F_u)}$ and $\phi = 0.50$,

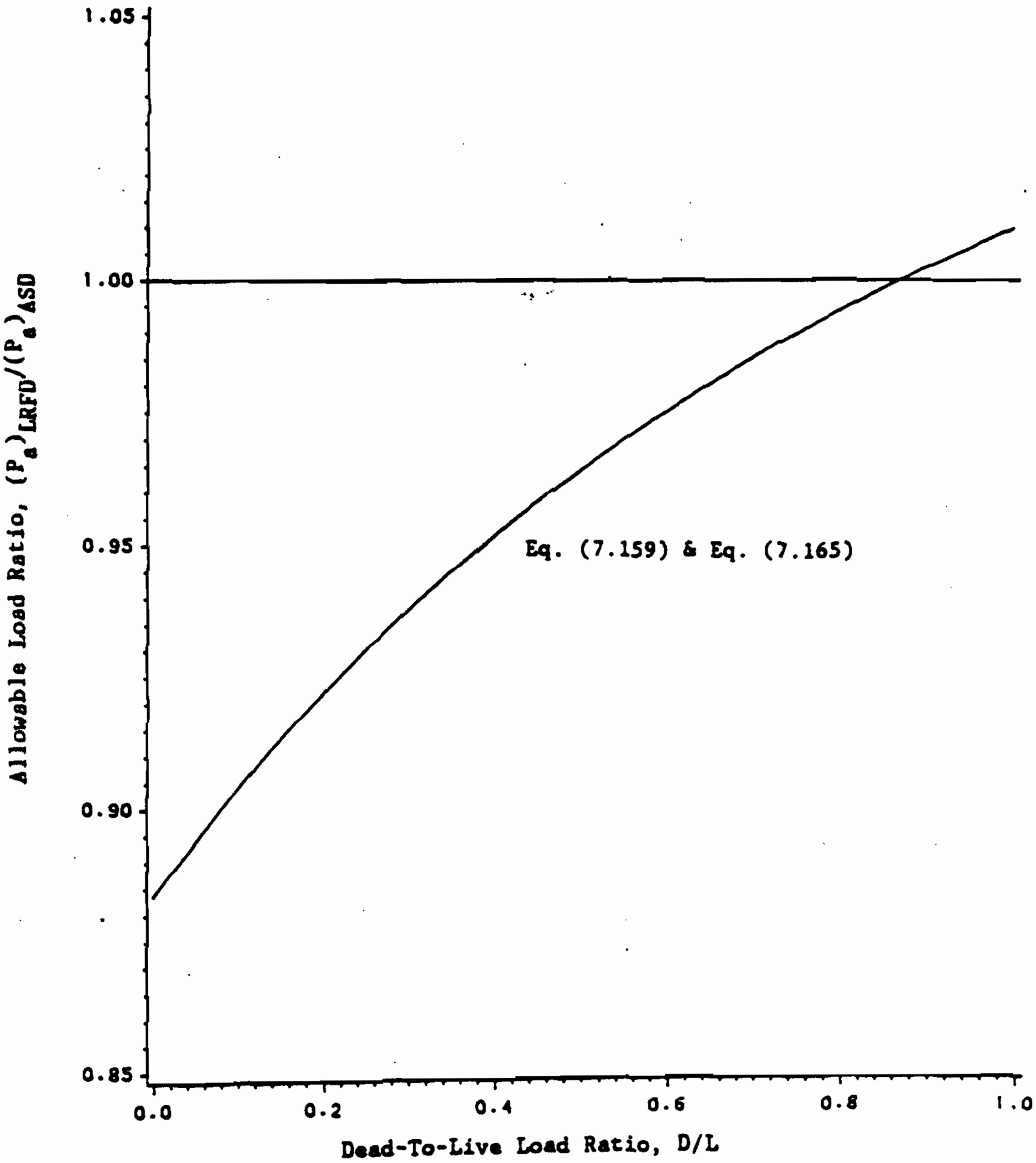

Figure 52 Allowable Load Ratio vs. D/L Ratio for Shear Failure of Arc Spot and Arc Seam Welds

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.5\phi \frac{D/L+1}{1.2D/L+1.6} = 1.25 \frac{D/L+1}{1.2D/L+1.6} \qquad (7.162)$$

Equations (7.160), (7.161), and (7.162) are shown in Figure 53 and are based on plate failure of arc spot welds. As seen from the figure, for D/L = 0.5, the allowable load ratios computed from LRFD and ASD vary from about 0.85 to 1.02 depending upon the d_a/t ratio used in the connection. For the typical range of D/L ratios used in cold-formed steel construction, LRFD is conservative for the design of arc spot welds when compared with allowable stress design.

For the determination of nominal tensile strength of arc spot welds, Eq. (5.158) is used for both ASD and LRFD criteria. In order to study the comparison between LRFD and ASD criteria, two types of load combination are considered:

For dead and live load combination with $\phi = 0.65$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.5\phi \frac{D/L+1}{1.2D/L+1.6} = 1.625 \frac{D/L+1}{1.2D/L+1.6} \qquad (7.163)$$

For counteracting loads (dead and wind loads) with $\phi = 0.65$, the required nominal load ratios can be computed by using the following equation developed from Eq. (7.25):

$$\frac{(P_n)_{ASD}}{(P_n)_{LRFD}} = (0.75)(2.5)\phi \frac{1-D/W}{1.17-0.9D/W} = 1.219 \frac{1-D/W}{1.17-0.9D/W} \qquad (7.164)$$

where

$(P_n)_{ASD}$ = required nominal load based on ASD criteria

$(P_n)_{LRFD}$ = required nominal load based on LRFD criteria

Figure 54 shows the allowable load ratio versus dead-to-live load ratio for tensile strength of arc spot welds determined by Eq. (7.163).

As shown in this figure, the LRFD criteria always result in higher values of allowable load than ASD criteria. For D/L = 0.2, the difference between the allowable loads is 6%.

Figure 55 shows the required nominal load ratio versus dead-to-wind load ratio for tensile strength of arc spot welds determined by Eq. (7.164). It can be seen that for $D/W < 0.15$, LRFD criteria will result in lower values of required nominal load than ASD criteria. For $D/W > 0.15$, LRFD is conservative as compared with ASD criteria.

2. Arc Seam Welds. For the determination of nominal shear strength based on shearing of the welds, Eq. (5.159) is used in the LRFD criteria. For allowable stress design, the nominal shear strength is determined with a coefficient of 2.5 in lieu of 0.75π in Eq. (5.159). Therefore, the allowable load ratio based on shear failure of arc seam welds and $\phi = 0.60$ is as follows:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 0.75\pi\phi \frac{D/L+1}{1.2D/L+1.6} = 1.414 \frac{D/L+1}{1.2D/L+1.6} \qquad (7.165)$$

Equation (7.165) is identical to Eq. (7.159) which is the allowable load ratio for arc spot welds based on weld shearing. Figure 52 shows the relationship between allowable load ratio and dead-to-live load ratio for this type of failure. As shown in the figure, LRFD is conservative for shear failure of arc seam welds compared with allowable stress design for $D/L < 0.90$.

Equation (5.160) is based on plate tearing and is used in both ASD and LRFD criteria. The allowable load ratio for plate failure and $\phi = 0.60$ is as follows:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.5\phi \frac{D/L+1}{1.2D/L+1.6} = 1.50 \frac{D/L+1}{1.2D/L+1.6} \qquad (7.166)$$

Figure 56 shows the allowable load ratio versus dead-to-live load ratio determined from Eq. (7.166) for plate tearing failure. Both design methods result in the same value of allowable load for a D/L ratio of 1/3. The allowable load based on LRFD is 2.3% greater than the value based on allowable stress design for D/L = 0.5. However, LRFD is conservative for D/L < 1/3 compared with allowable stress design.

3. Fillet Welds. Equations (5.164), (5.165), and (5.166) are based on plate tearing and are used in both ASD and LRFD criteria. The allowable load ratio can be computed using the following formula:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.5\phi \frac{D/L+1}{1.2D/L+1.6} \qquad (7.167)$$

For longitudinal loading with L/t < 25, the resistance factor is 0.60. Therefore, the allowable load ratio can be computed using the following equation:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.50 \frac{D/L+1}{1.2D/L+1.6} \qquad (7.168)$$

For longitudinal loading with L/t ≥ 25, the resistance factor is 0.55. Therefore, the following equation can be used to calculate the allowable load ratio:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.375 \frac{D/L+1}{1.2D/L+1.6} \qquad (7.169)$$

For transverse loading with $\phi = 0.6$, Eq. (7.170) can be used to calculate the allowable load ratio.

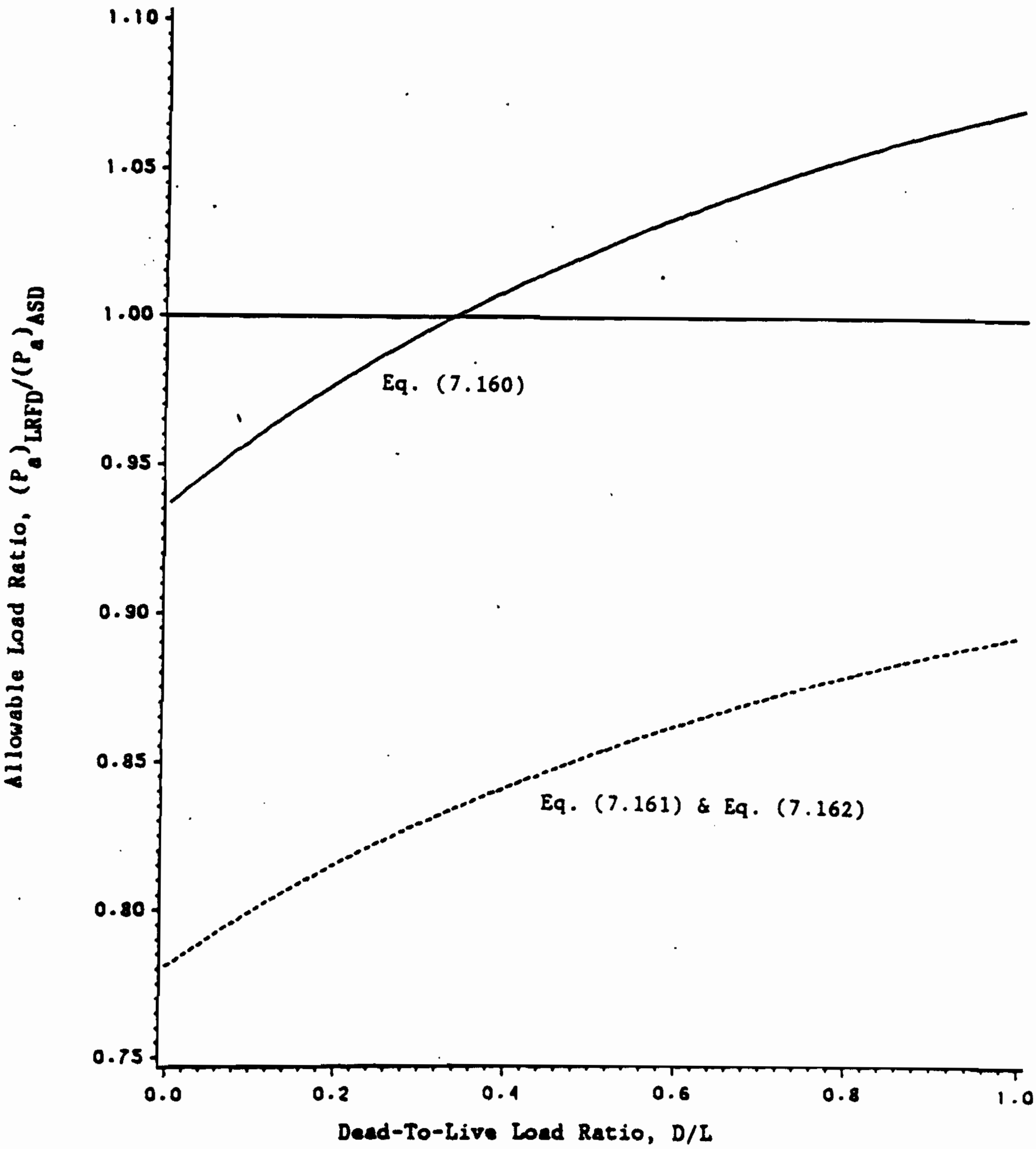

Figure 53 Allowable Load Ratio vs. D/L Ratio for Plate Tearing of Arc Spot Welds

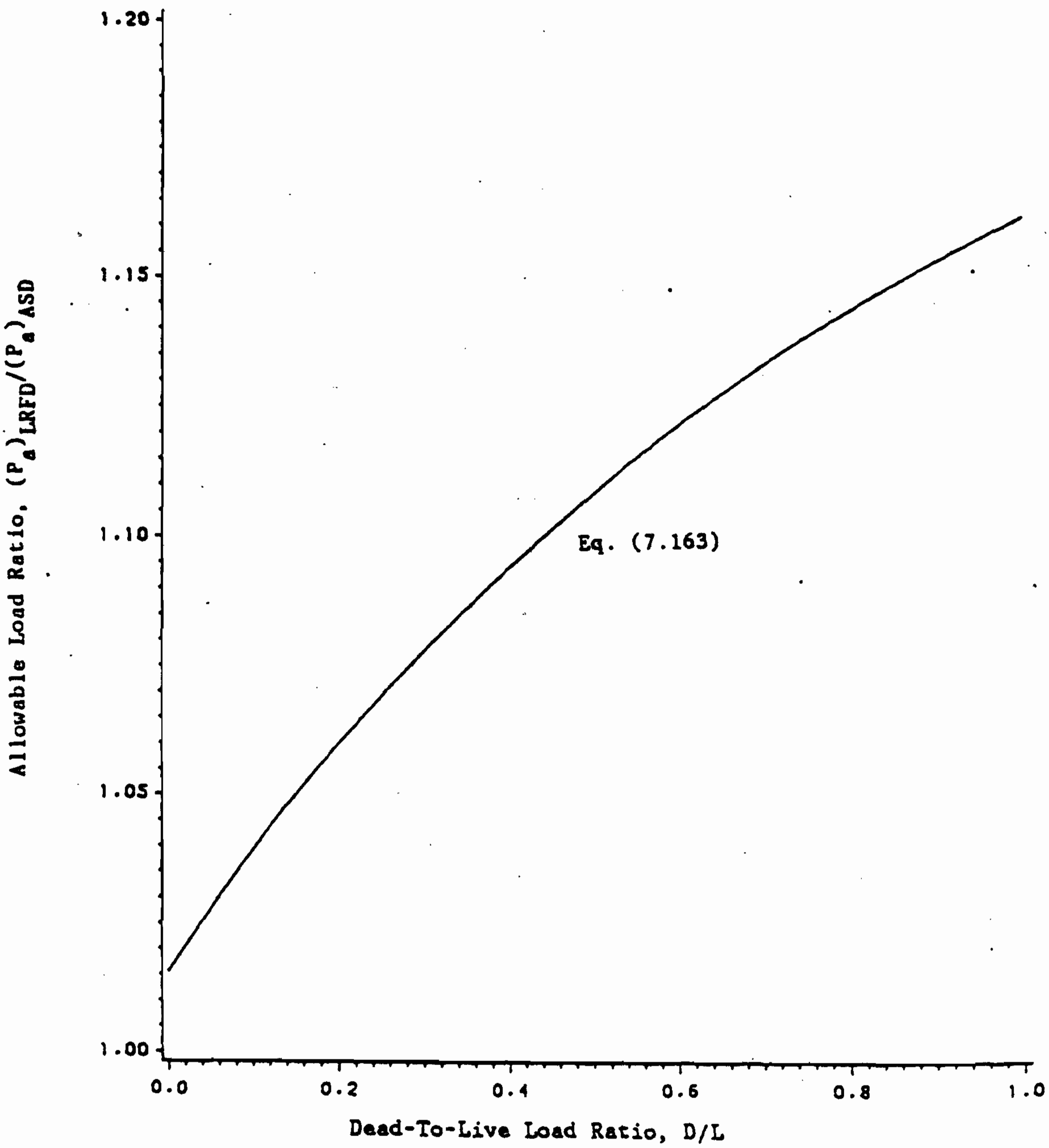

Figure 54 Allowable Load Ratio vs. D/L Ratio for Tensile Strength of Arc Spot Welds

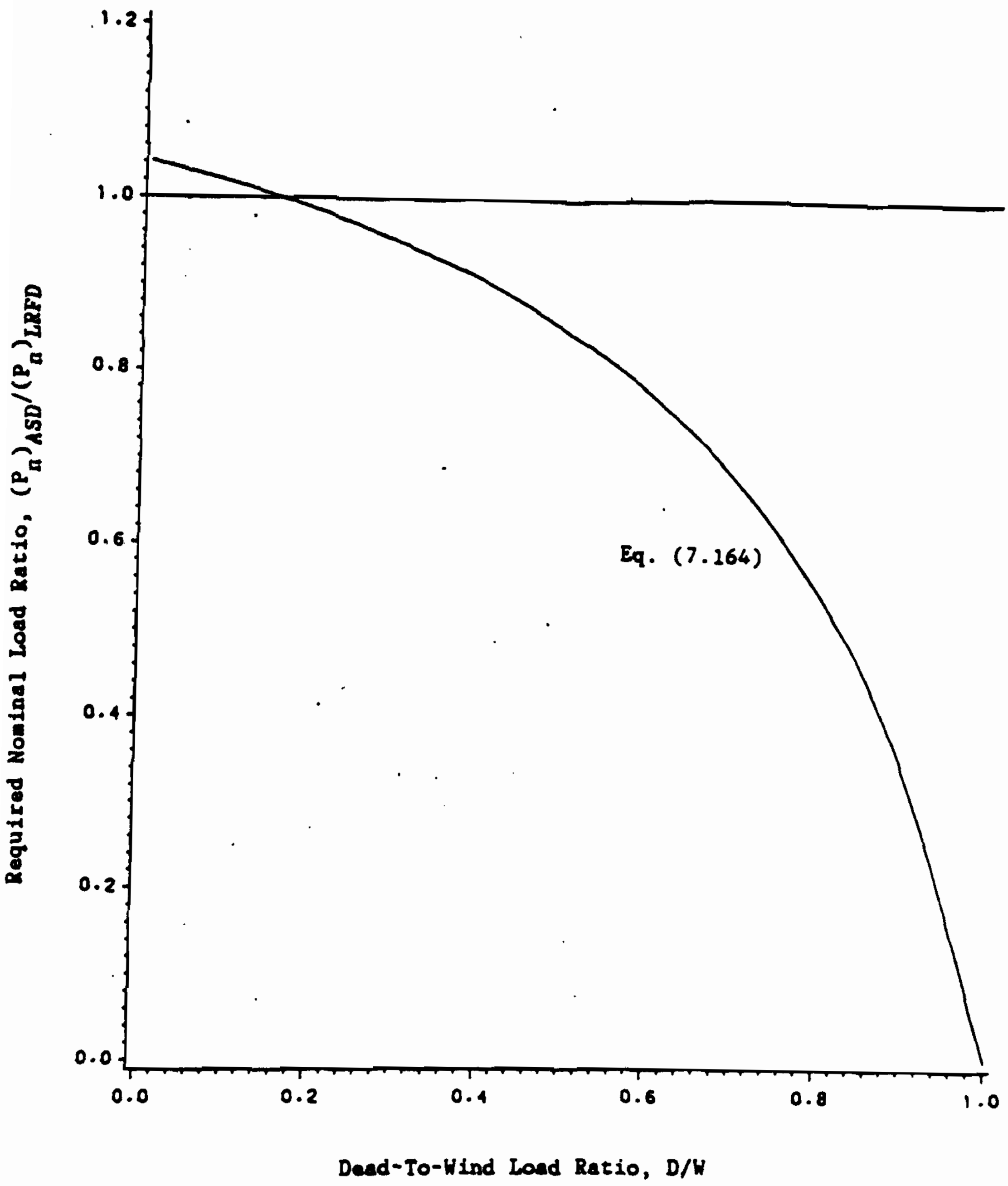

Figure 55 Required Nominal Load Ratio vs. D/W Ratio for Tensile Strength of Arc Spot Welds

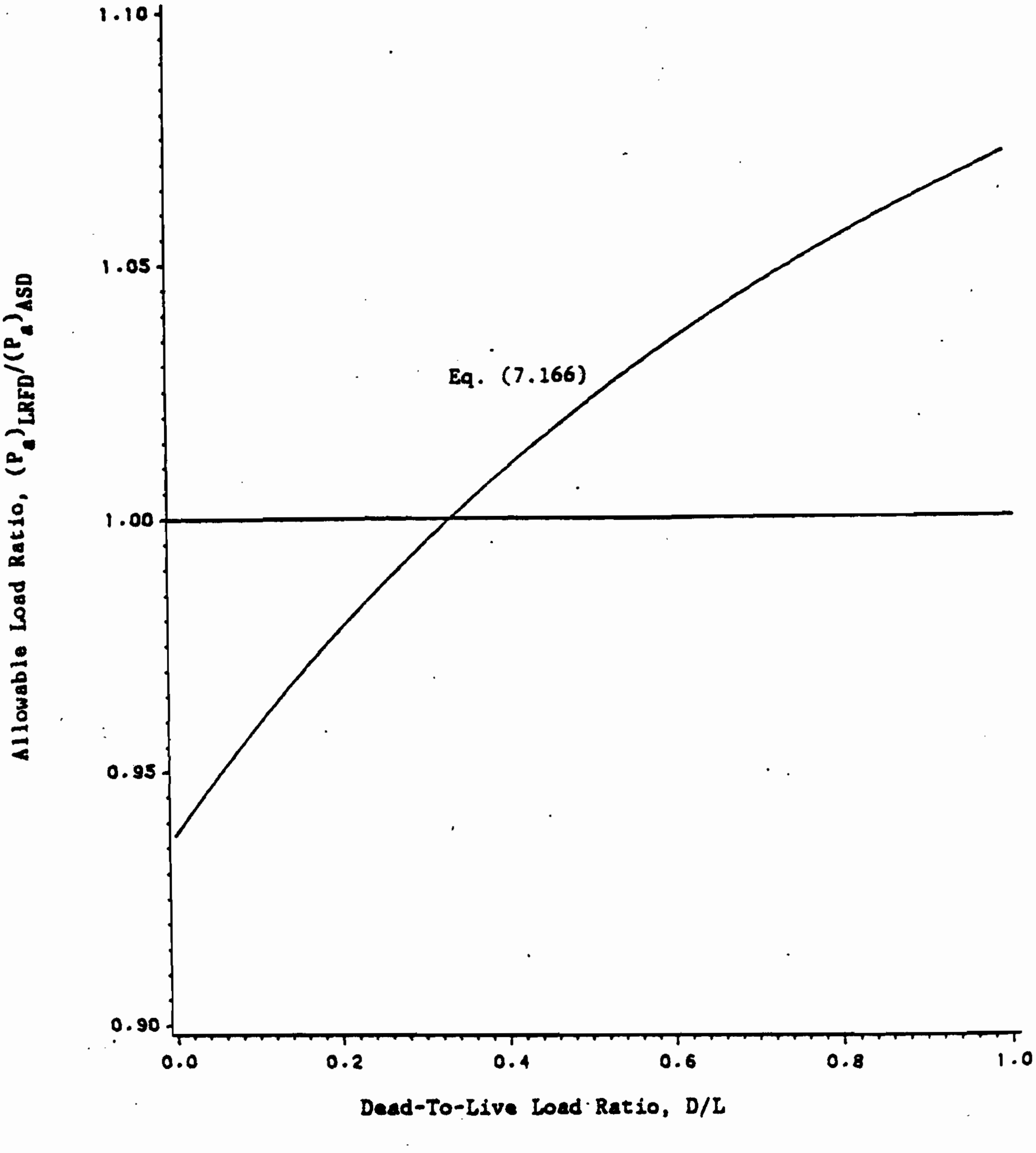

Figure 56 Allowable Load Ratio vs. D/L Ratio for Plate Tearing of Arc Seam Welds

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.50 \frac{D/L+1}{1.2D/L+1.6} \tag{7.170}$$

The relationship between allowable load ratio and dead-to-live load ratio is shown in Figure 57 for plate tearing failure based on Eqs. (7.168), (7.169), and (7.170). For longitudinally loaded fillet welds, with $L/t < 25$ and $D/L = 0.5$, the allowable load computed from LRFD is 2.3% higher than the value computed from allowable stress design. For longitudinally loaded fillet welds, with $L/t \geq 25$ and $D/L = 0.5$, the allowable load computed from LRFD is 6.1% lower than the value computed from allowable stress design.

For transverse loading of fillet welds, the allowable load based on the LRFD criteria is also 2.3% higher than the value based on allowable stress design for $D/L = 0.5$.

When the thickness of the plate is greater than 0.15 in., weld shearing has to be checked. Equation (5.167) is used in both ASD and LRFD criteria. The allowable load ratio can be computed using the following formula with $\phi = 0.60$:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.5\phi \frac{D/L+1}{1.2D/L+1.6} = 1.50 \frac{D/L+1}{1.2D/L+1.6} \tag{7.171}$$

The relationship between allowable load ratio and dead-to-live load ratio for weld failure of fillet welds is shown in Figure 58. From the figure, for $D/L = 0.5$, LRFD criteria result in an allowable load 2.3% larger than the value computed from allowable stress design.

4. Flare Groove Welds. Equations (5.168), (5.169), and (5.170) are based on plate failure, and are used in both ASD and LRFD criteria. The allowable load ratio can be computed using the following formula:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.5\phi \frac{D/L+1}{1.2D/L+1.6} \qquad (7.172)$$

For flare-bevel groove welds loaded in the transverse direction, and ϕ = 0.55, the following equation can be used for allowable load ratio:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.375 \frac{D/L+1}{1.2D/L+1.6} \qquad (7.173)$$

For flare groove welds loaded in the longitudinal direction, and ϕ = 0.55, the allowable load ratio can be computed as follows:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.375 \frac{D/L+1}{1.2D/L+1.6} \qquad (7.174)$$

Figure 59 shows the relationship between allowable load ratio and dead-to-live load ratio computed from Eqs. (7.173) and (7.174). For transverse loading of flare-bevel groove welds and D/L = 0.5, the allowable load computed from LRFD is 6.3% lower than the value computed from allowable stress design. The same is true for flare groove welds loaded in the longitudinal direction. As shown in the figure, the LRFD criteria for flare groove welds are slightly conservative for the values of D/L ratios generally used in cold-formed steel construction.

For flare groove welds on sheets thicker than 0.15 in., weld shearing may govern the design. Equation (5.171) is used in both ASD and LRFD criteria. With ϕ = 0.60, the allowable load ratio can be computed as follows:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.5\phi \frac{D/L+1}{1.2D/L+1.6} = 1.50 \frac{D/L+1}{1.2D/L+1.6} \qquad (7.175)$$

Equation (7.175) is identical to Eq. (7.171) which is the allowable load ratio for fillet welds based on the same type of failure. Figure 58 shows the allowable load ratio versus dead-to-live load ratio for weld failure of fillet and flare groove welds. The allowable load ratio based on LRFD is 2.3% larger than the value based on allowable stress design for D/L = 0.5.

5. Resistance Welds. The allowable loads per spot weld for allowable stress design were derived from the nominal values listed in Table XVI using a factor of safety of 2.5. Therefore, the following equation for allowable load ratio can be used for $\phi = 0.65$:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.5\phi \frac{D/L+1}{1.2D/L+1.6} = 1.625 \frac{D/L+1}{1.2D/L+1.6} \quad (7.176)$$

The relationship between allowable load ratio and dead-to-live load ratio is shown in Figure 60 for resistance welds. As shown from the figure, LRFD criteria always result in higher values of allowable load than allowable stress design. For D/L = 0.5, the difference between the allowable loads is 10.8%.

J. COMPARATIVE STUDY OF BOLTED CONNECTIONS

The allowable load per bolt for allowable stress design can be determined as $P_a = P_n/\Omega$. For the LRFD criteria, the allowable load per bolt can be calculated from the following equation developed from Eq. (7.4):

$$(P_a)_{LRFD} = \phi P_n(D/L+1)/(1.2D/L+1.6) \quad (7.177)$$

1. Spacing and Edge Distance. For allowable stress design, the allowable load can be computed for a given edge distance by using the following equations:

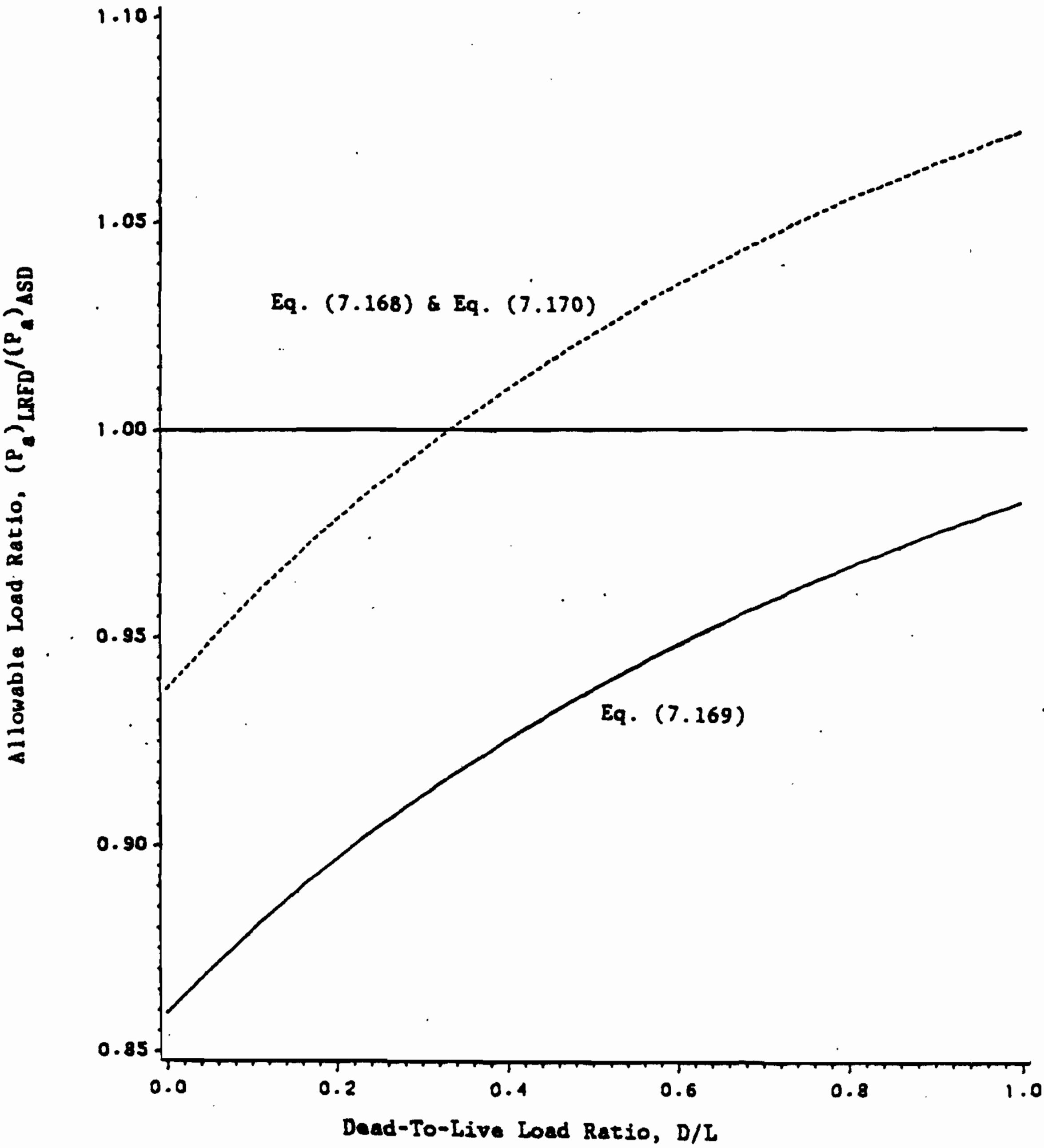

Figure 57 Allowable Load Ratio vs. D/L Ratio for Plate Tearing of Fillet Welds

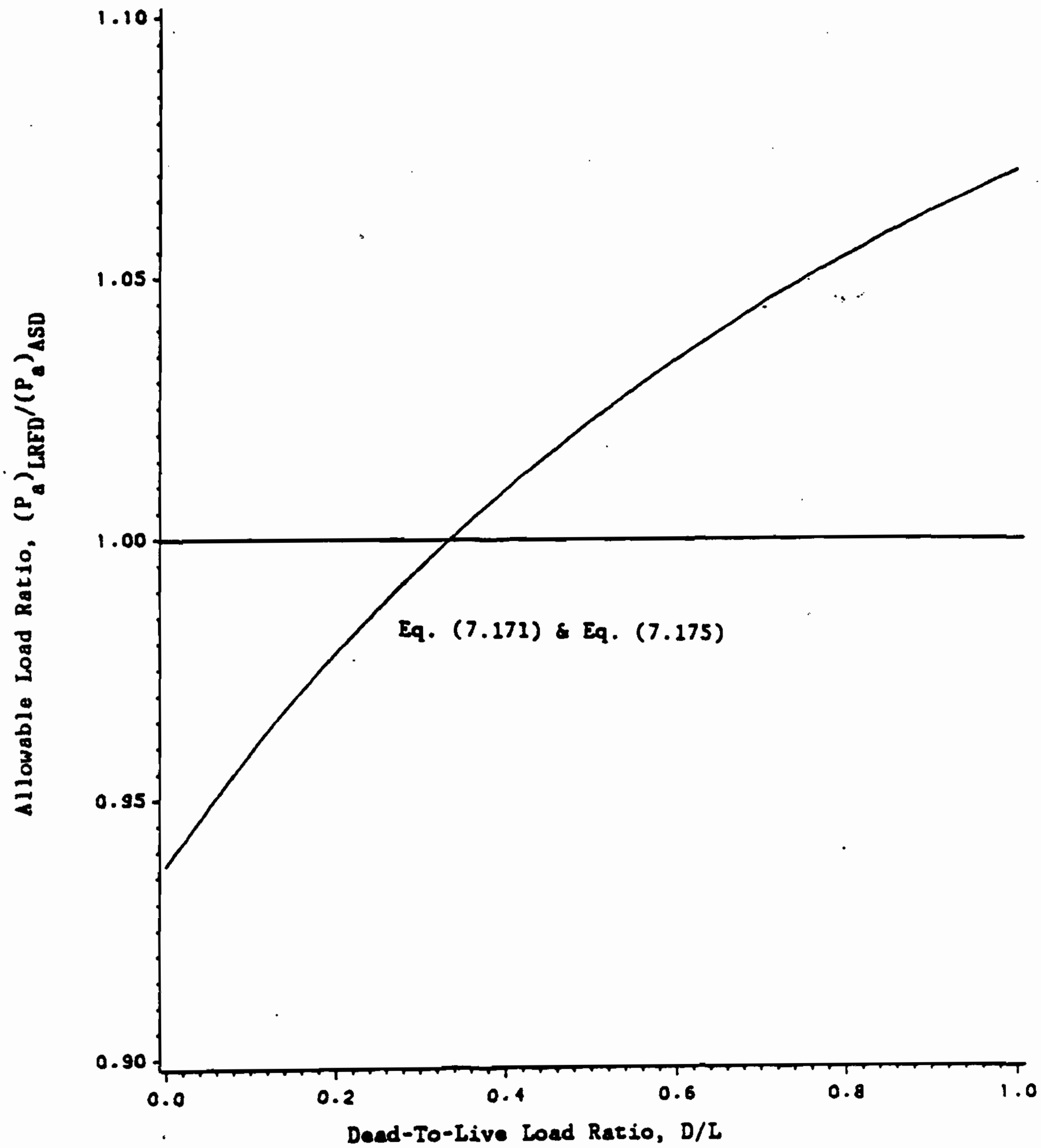

Figure 58 Allowable Load Ratio vs. D/L Ratio for Weld Failure of Fillet and Flare Groove Welds

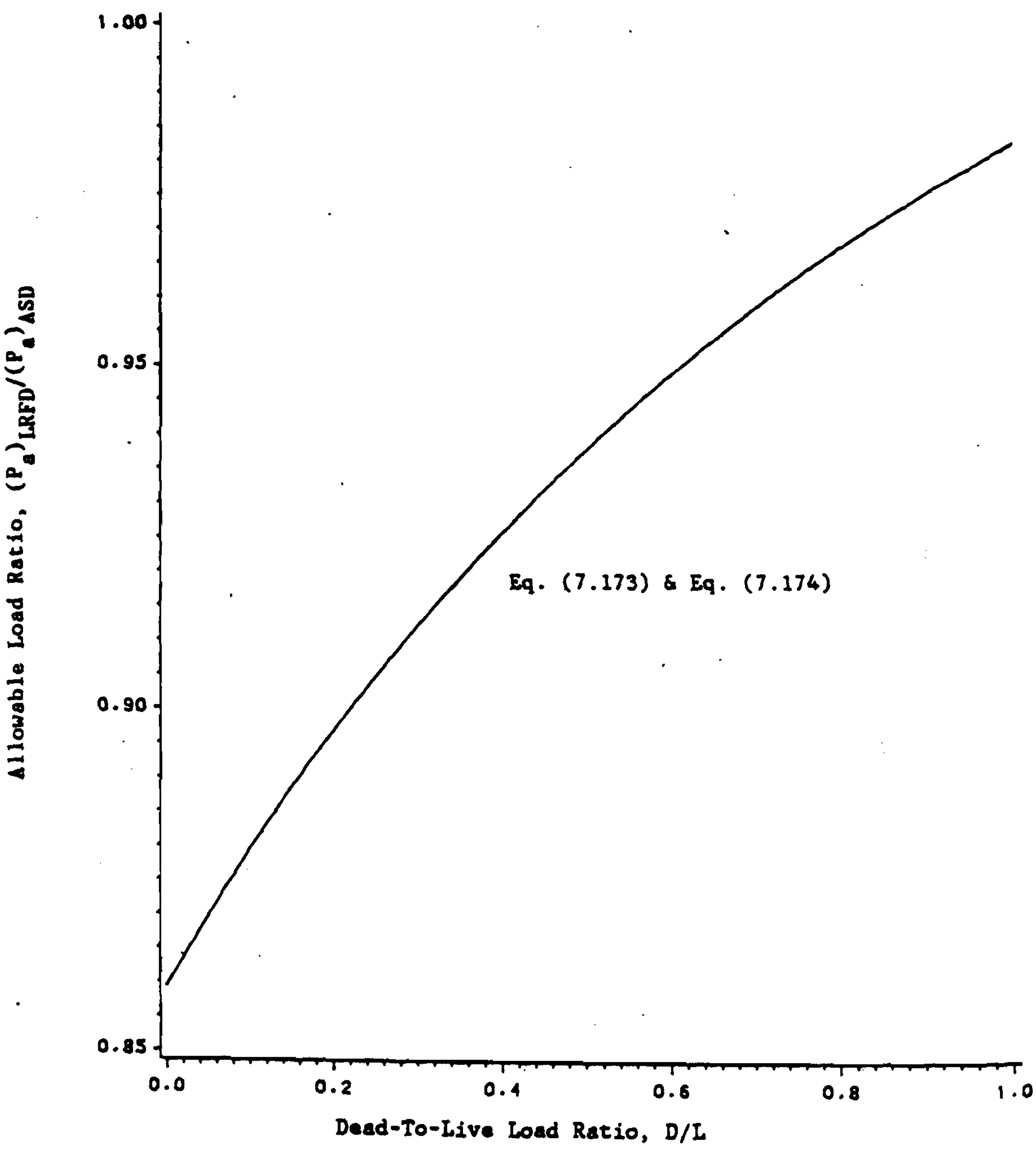

Figure 59 Allowable Load Ratio vs. D/L Ratio for Plate Tearing of Flare Groove Welds

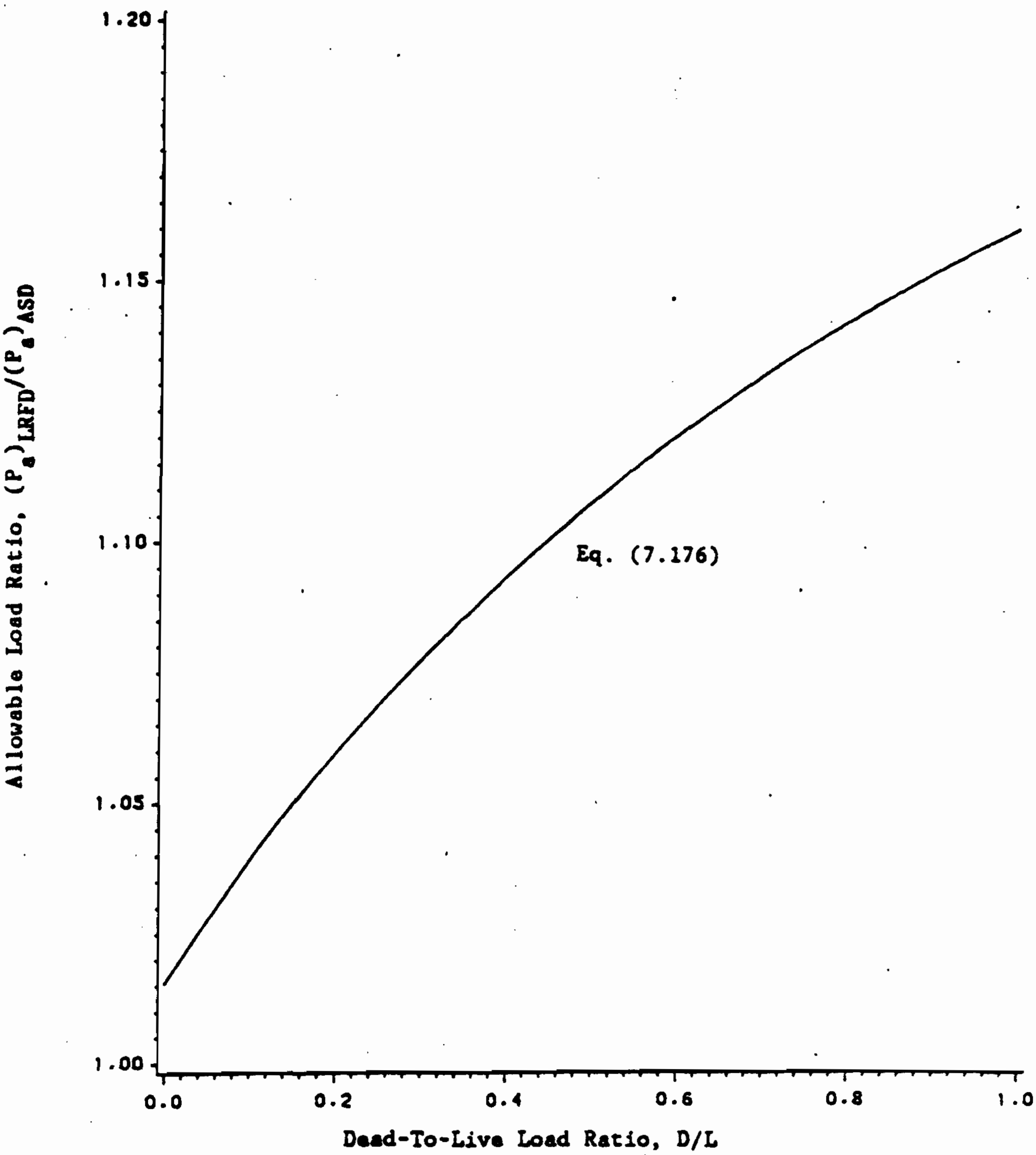

Figure 60 Allowable Load Ratio vs. D/L Ratio for Resistance Welds

For $F_u/F_{sy} \geq 1.15$,

$$(P_a)_{ASD} = 0.5teF_u \tag{7.178}$$

For $F_u/F_{sy} < 1.15$,

$$(P_a)_{ASD} = 0.45teF_u \tag{7.179}$$

The allowable load for LRFD can be computed using Eq. (7.177). The allowable loads from Eqs. (7.178) and (7.179) were derived from the nominal strength in Eq. (5.172) using a factor of 2.00 and 2.22, respectively. Therefore, the allowable load ratios based on plate shearing around the bolt can be computed from the following:

For $F_u/F_{sy} \geq 1.15$, $\phi = 0.70$:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.4 \frac{D/L+1}{1.2D/L+1.6} \tag{7.180}$$

For $F_u/F_{sy} < 1.15$, $\phi = 0.60$:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.332 \frac{D/L+1}{1.2D/L+1.6} \tag{7.181}$$

Figure 61 shows the relationships between allowable load ratio and dead-to-live load ratio for Eqs. (7.180) and (7.181). For D/L = 0.5, the allowable loads based on the LRFD criteria are from 4.5% to 9.2% lower than the values based on allowable stress design.

2. <u>Tension in Connected Parts</u>. For allowable stress design, the allowable tension on the net section can be computed by Eq. (7.182).

$$(P_a)_{ASD} = A_n F_t / \Omega_t \tag{7.182}$$

For LRFD, the allowable tension on the net section can be computed using Eq. (7.177).

The allowable load for double shear connections with washers based on allowable stress design was derived from the nominal tensile load, and a factor of safety of 2.0. For single shear connections without washers, a factor of safety of 2.22 was used for allowable stress design. The yielding criteria for the net section was studied in Section VII.C. The allowable load ratios can be computed as follows:

For double shear connections with washers and $\phi = 0.65$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.0\phi \frac{D/L+1}{1.2D/L+1.6} = 1.30 \frac{D/L+1}{1.2D/L+1.6} \tag{7.183}$$

For single shear connections with washers and $\phi = 0.55$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.22\phi \frac{D/L+1}{1.2D/L+1.6} = 1.221 \frac{D/L+1}{1.2D/L+1.6} \tag{7.184}$$

For connections without washers and $\phi = 0.65$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.22\phi \frac{D/L+1}{1.2D/L+1.6} = 1.443 \frac{D/L+1}{1.2D/L+1.6} \tag{7.185}$$

Figure 62 shows the allowable load ratio versus dead-to-live load ratio for the three cases represented by Eqs. (7.183), (7.184), and (7.185). As shown in the figure, the criteria for tension on the net section result in a wide range of allowable load ratios. For D/L = 0.5, the allowable loads based on the LRFD criteria are from 1.8% to 16.7% lower than the values based on allowable stress design. The difference depends on the use of washers and the type of connections. Figure 62 also shows that LRFD is very conservative for connections with washers under the bolt head and nut, compared with allowable stress design.

3. <u>Bearing</u>. The allowable load based on allowable stress design can be computed using the following equation:

$$(P_a)_{ASD} = F_p td/\Omega_b \tag{7.186}$$

For LRFD, Eq. (7.177) can be used to calculate the allowable load.

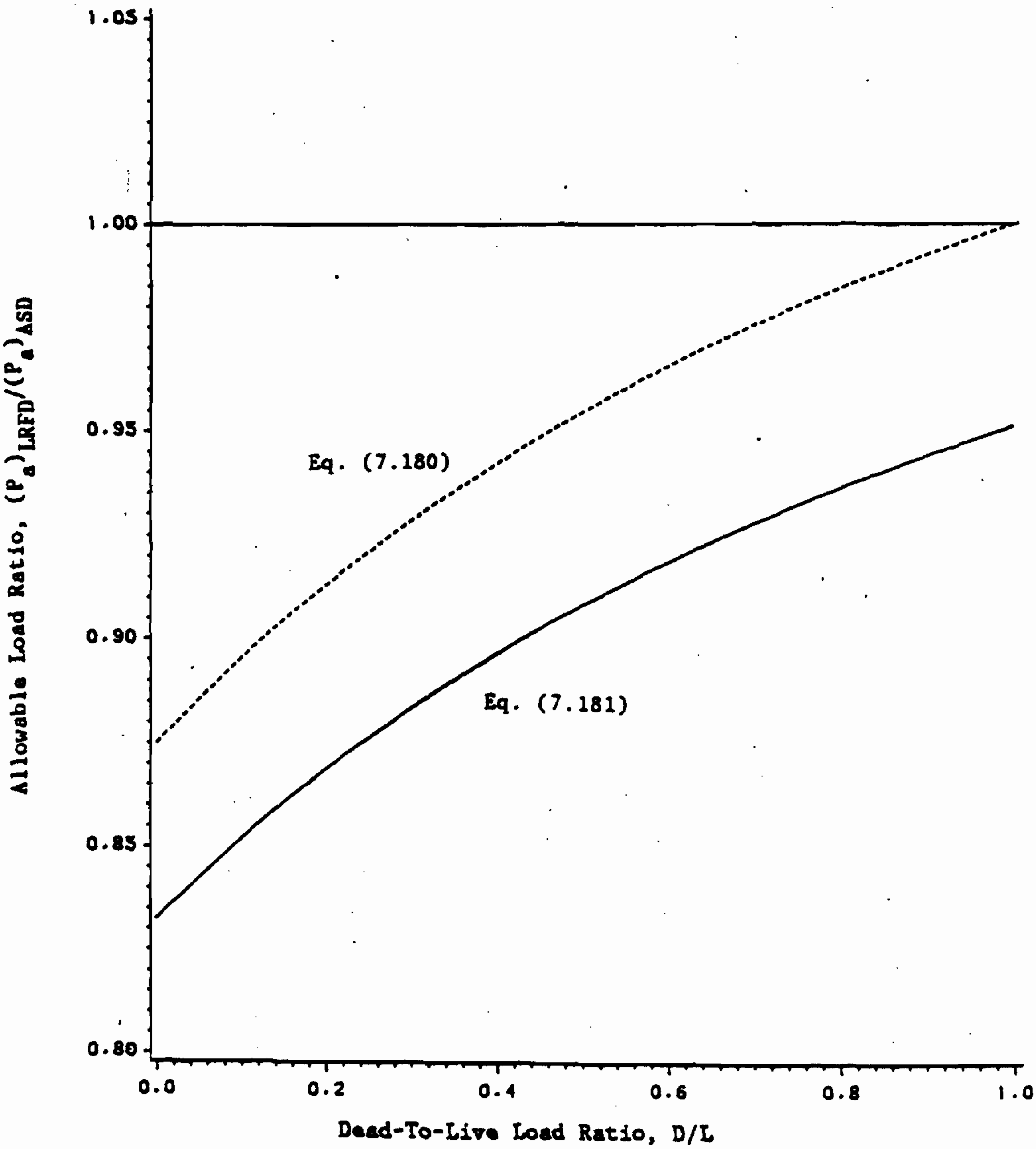

Figure 61 Allowable Load Ratio vs. D/L Ratio for Minimum Edge Distance of Bolts

The factor of safety used in the development of the allowable stress design formulas was 2.22. Therefore, the allowable load ratios can be computed as follows:

(1) Connections with washers:

For inside sheets of double shear connections with

$F_u/F_{sy} \geq 1.15$ and $\phi = 0.55$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.221 \frac{D/L+1}{1.2D/L+1.6} \tag{7.187}$$

For inside sheets of double shear connections with

$F_u/F_{sy} < 1.15$ and $\phi = 0.65$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.443 \frac{D/L+1}{1.2D/L+1.6} \tag{7.188}$$

For single shear and outside sheets of double shear connections with $\phi = 0.60$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.332 \frac{D/L+1}{1.2D/L+1.6} \tag{7.189}$$

(2) Connections without washer or with only one washer:

For inside sheets of double shear connections with

$F_u/F_{sy} \geq 1.15$ and $\phi = 0.70$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.554 \frac{D/L+1}{1.2D/L+1.6} \tag{7.190}$$

For single shear and outside sheets of double shear connections with $F_u/F_{sy} \geq 1.15$ and $\phi = 0.65$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.443 \frac{D/L+1}{1.2D/L+1.6} \tag{7.191}$$

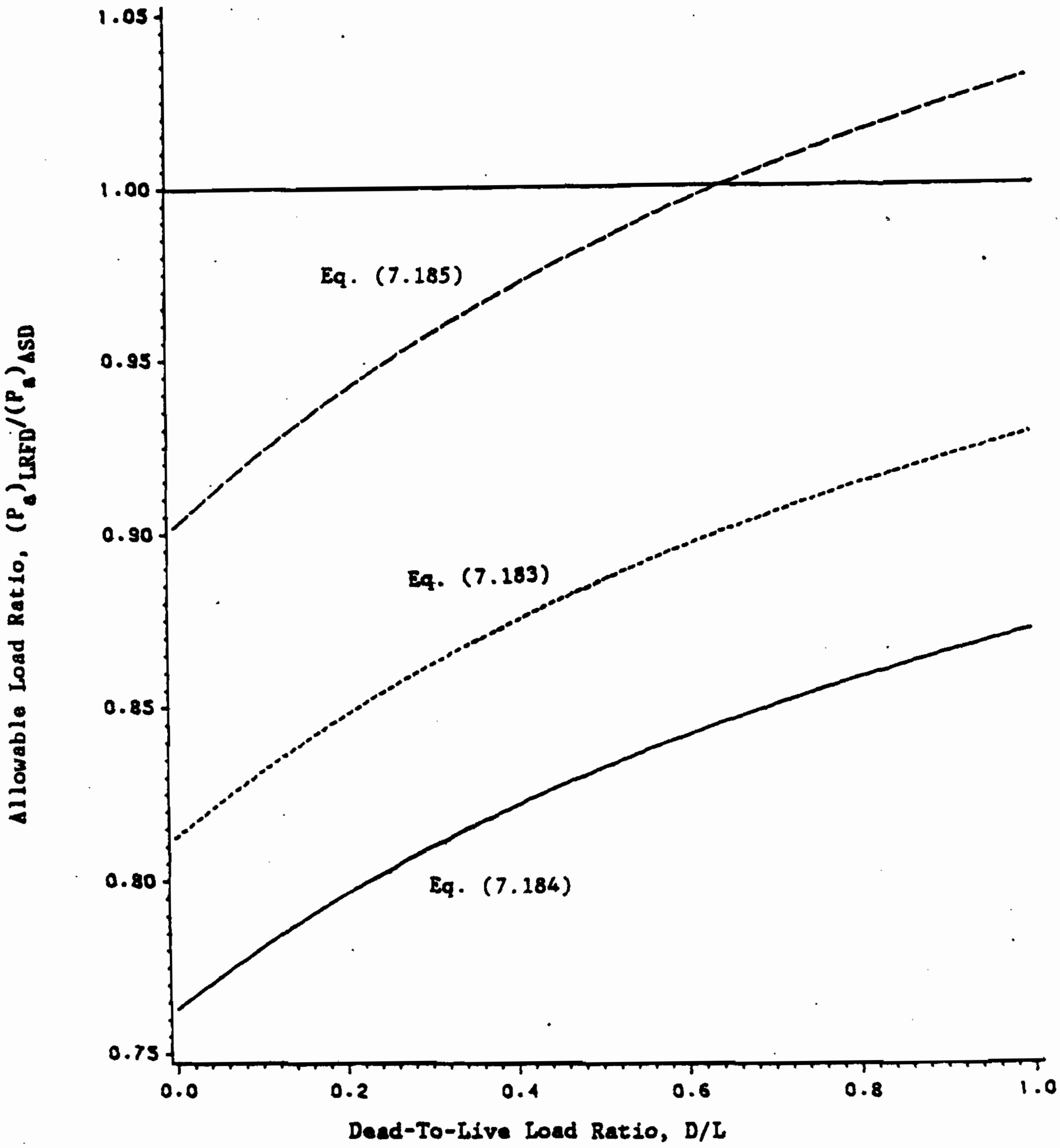

Figure 62 Allowable Load Ratio vs. D/L Ratio for Tension on Net Section

The relationships between allowable load ratio and dead-to-live load ratio for Eqs. (7.187) through (7.191) are shown in Figure 63. As shown in the figure, the criteria for bearing strength of bolted connections result in a wide range of values for allowable load ratio. For D/L = 0.5, the allowable loads based on LRFD are from 6% higher to 16.7% lower than the values obtained from allowable stress design. The difference between the allowable loads will depend upon the use of the washers, the shear conditions, and the F_u/F_{sy} ratio. Inside sheets of double shear bolted connection with washers designed using LRFD are very conservative as compared with allowable stress design.

4. Shear and Tension in Bolts. The allowable load based on allowable stress design can be computed as follows:

$$(P_a)_{ASD} = A_b F \qquad (7.192)$$

where

F is allowable stress given by F_v, F_t, or F_t' in Tables XXXIII and XXXIV

For LRFD, Eq. (7.177) can be used to calculate the allowable load.

Therefore, the allowable load ratio for shear or tension of bolts is:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = \frac{\phi A_b F_n}{A_b F}\left[\frac{D/L+1}{1.2D/L+1.6}\right] = \frac{\phi F_n}{F}\left[\frac{D/L+1}{1.2D/L+1.6}\right] \qquad (7.193)$$

Equation (7.193) can be expressed in the following form:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = (K_b)\,\frac{D/L+1}{1.2D/L+1.6} \qquad (7.194)$$

where

$$K_b = \phi F_n/F \qquad (7.195)$$

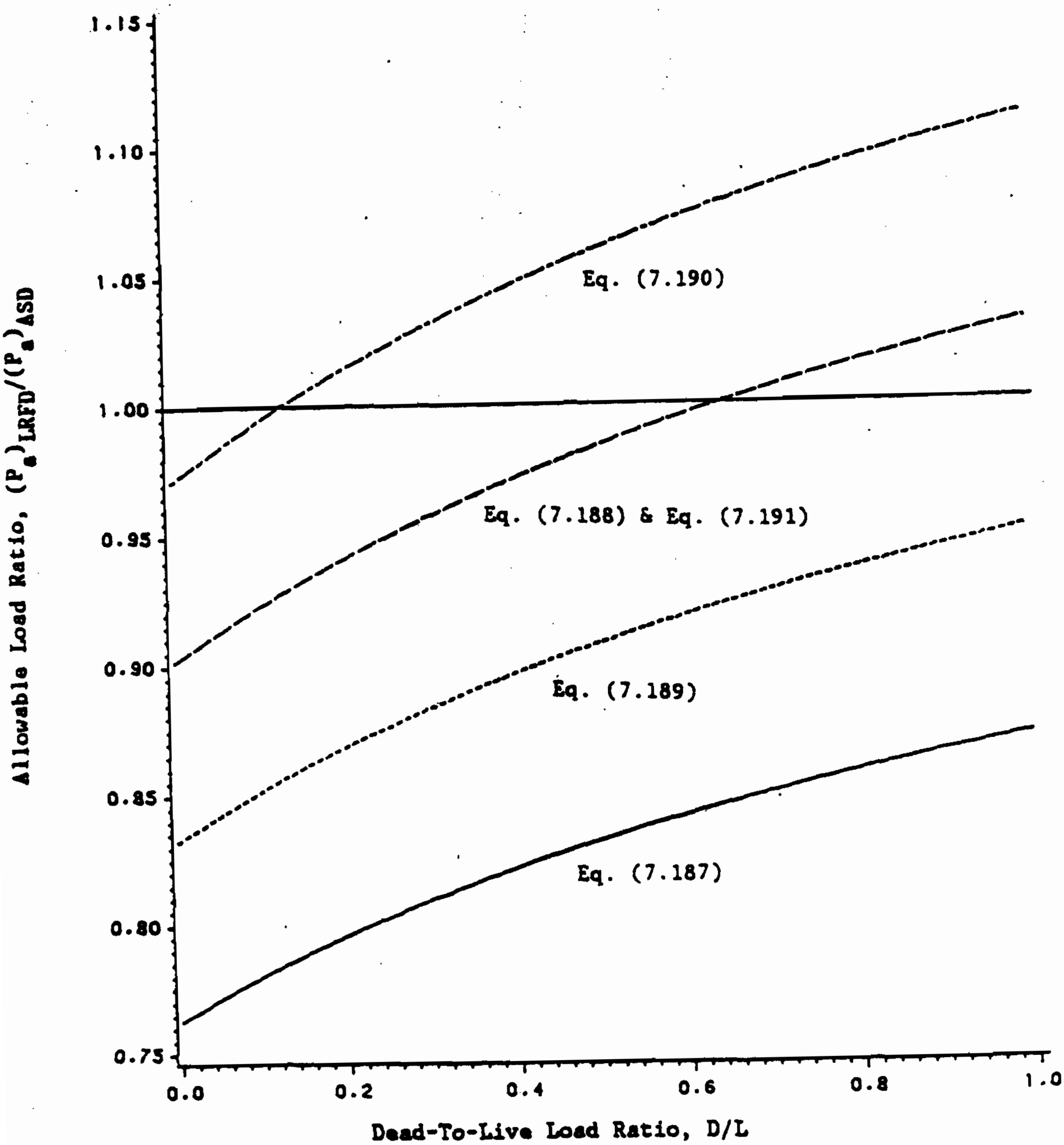

Figure 63 Allowable Load Ratio vs. D/L Ratio for Bearing Strength of Bolted Connections

Table XXXV lists the values of K_b calculated from the values of F, and F_n, provided in Tables XXXIII and XXII, and values of ϕ determined in Section V.I.2.

Figure 64 shows the allowable load ratio versus dead-to-live load ratio for A325 bolts based on shear and tension strengths. As seen from this figure, for D/L = 0.5, the allowable tensile load based on LRFD design is 4.6% larger than the value based on allowable stress design. Also for D/L = 0.5, when threads are included in the shear plane, the allowable shear load based on LRFD design is 13.9% larger than the value based on allowable stress design; when threads are not included in the shear plane, the allowable shear load based on LRFD design is 6.4% larger than the value based on allowable stress design. It can also be seen from this figure that LRFD design will always result in a larger allowable shear load than allowable stress design when threads are included in the shear plane.

All other cases listed in Table XXXV were also studied. In general, the curves obtained for all cases are similar to those shown in Figure 64 except that the curves will be shifted up or down depend on the values of K_b. Detailed information can be found in Reference 36.

When bolts are subject to a combination of shear and tension, the unfactored shear force can be calculated for both ASD and LRFD methods using the following equation:

$$V_T = V_{DL} + V_{LL} \tag{7.196}$$

where

V_T = total unfactored shear force

V_{DL} = shear force due to the nominal dead load

V_{LL} = shear force due to the nominal live load

The factored shear force for LRFD design can be expressed as Eq. (7.197) by using Eq. (7.4):

$$V_u = V_T \frac{1.2D/L+1.6}{D/L+1} \tag{7.197}$$

Therefore, the allowable load ratio for tensile strength when bolts are subject to a combination of shear and tension can be developed as follows by using Eq. (7.193):

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = \frac{\phi\left(C_1 - D_1 f_v \dfrac{1.2D/L+1.6}{D/L+1}\right)}{C - Df_v} \frac{D/L+1}{1.2D/L+1.6} \tag{7.198}$$

where

ϕ = 0.75

$$f_v = V_T/A_b \tag{7.199}$$

C and D are tabulated in Table XXXIV

C_1 and D_1 are tabulated in Table XXIII

Figure 116 shows the allowable load ratio versus dead-to-live load ratio for A325 bolts when threads are included in shear plane. The different curves in this figure represent different unfactored shear stresses f_v. For D/L ratio around 0.18, both design methods would result in the same allowable tensile load for the unfactored shear stresses f_v shown in the figure. For D/L > 0.18, the larger the unfactored shear stress, the higher the allowable load ratio. For example, for D/L = 0.5, the $(P_a)_{LRFD}/(P_a)_{ASD}$ ratios are 1.162 and 1.066 for f_v = 21 ksi and 7 ksi, respectively.

All other cases included in Tables XXXIV and XXIII were studied in similar manner. The results are similar to those obtained for A325 bolts when threads are included in shear plane. Detailed information can be found in Reference 36.

Table XXXIII

Allowable Shear and Tension Stresses for Bolts

Description of Bolts	Allowable shear Stress, F_v, ksi		Allowable Tension Stress, F_t, ksi
	Threads not Excluded from Shear Plane	Threads Excluded from Shear Plane	
A325 Bolts	21	30	44
A354 Grade B Bolts (1/4 in. ≤ d < 1/2 in.)	24	40	49
A449 Bolts (1/4 in. ≤ d < 1/2 in.)	18	30	40
A490 Bolts	28	40	54
A307 Bolts, Grade A (1/4 in. ≤ d < 1/2 in.)	9		18
A307 Bolts, Grade A (d ≥ 1/2 in.)	10		20

Table XXXIV

Allowable Tension Stress, F_t', for Bolts

Subject to the Combination of Shear and Tension

Description of Bolts	Threads Not Excluded from Shear Planes	Threads Excluded from Shear Planes
A325 Bolts	$55 - 1.8f_v \leq 44$	$55 - 1.4f_v \leq 44$
A354 Grade BD Bolts	$61 - 1.8f_v \leq 49$	$61 - 1.4f_v \leq 49$
A449 Bolts	$50 - 1.8f_v \leq 40$	$50 - 1.4f_v \leq 40$
A490 Bolts	$68 - 1.8f_v \leq 54$	$68 - 1.4f_v \leq 54$
A307 Bolts, Grade A		
When 1/4in. $\leq d < 1/2$ in.	$23 - 1.8f_v \leq 18$	
When $d \geq 1/2$in.	$26 - 1.8f_v \leq 20$	

Note: The general form for formulas listed in this table can be written as $C - Df_v \leq Z$

Table XXXV

K_b Values for Standard Bolts

Description of Bolts	Shear Strength: Threads not Excluded from Shear Plane	Shear Strength: Threads Excluded from Shear Plane	Tension Strength
A325 Bolts	1.671	1.560	1.534
A354 Grade B Bolts (1/4 in. ≤ d < 1/2 in.)	1.598	1.463	1.546
A449 Bolts (1/4 in. ≤ d < 1/2 in.)	1.697	1.560	1.519
A490 Bolts	1.567	1.463	1.563
A307 Bolts, Grade A (1/4 in. ≤ d < 1/2 in.)	1.733		1.688
A307 Bolts, Grade A (d ≥ 1/2 in.)	1.755		1.688

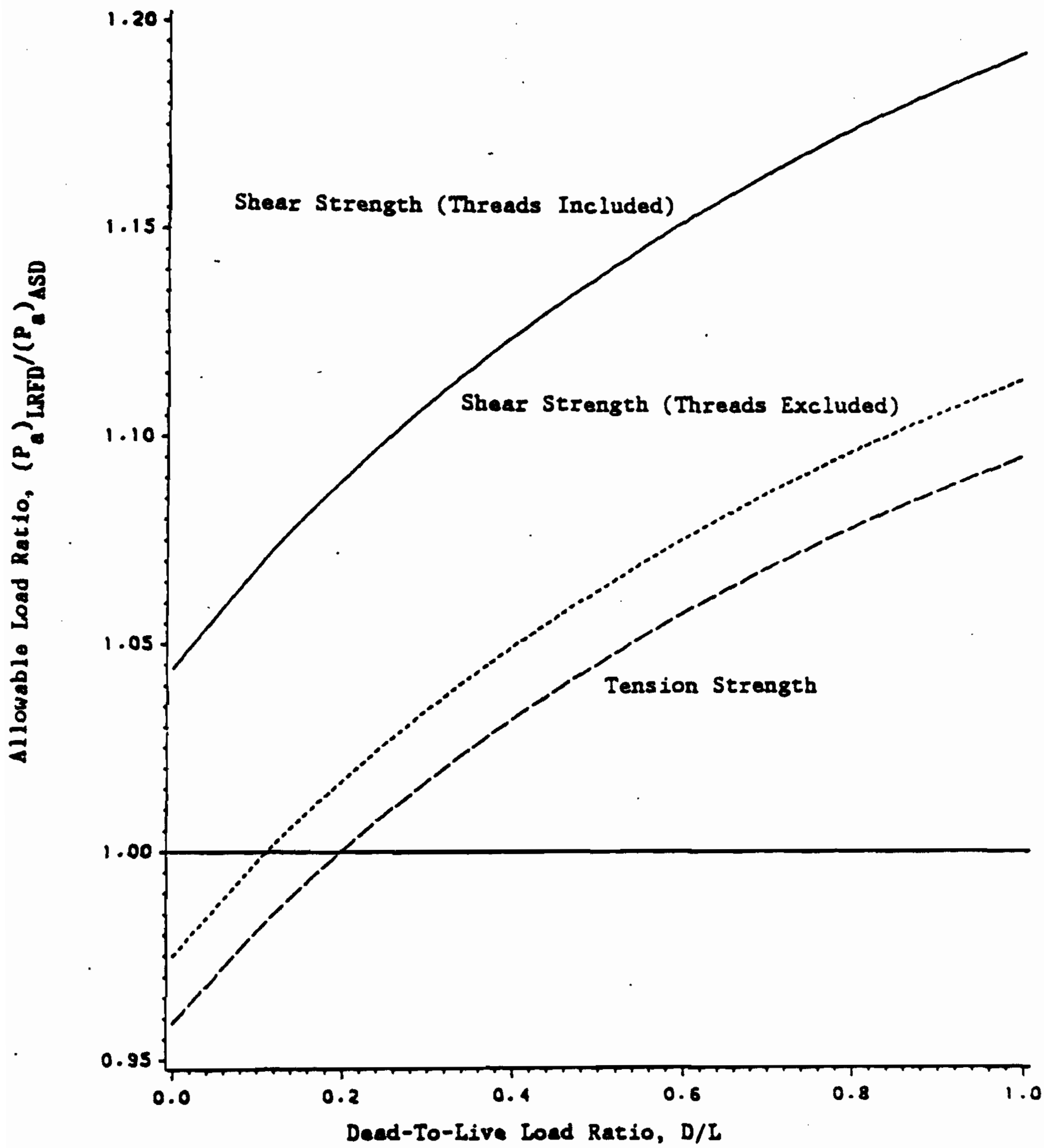

Figure 64 Allowable Load Ratio vs. D/L Ratio for Shear or Tension Strength on A325 Bolts

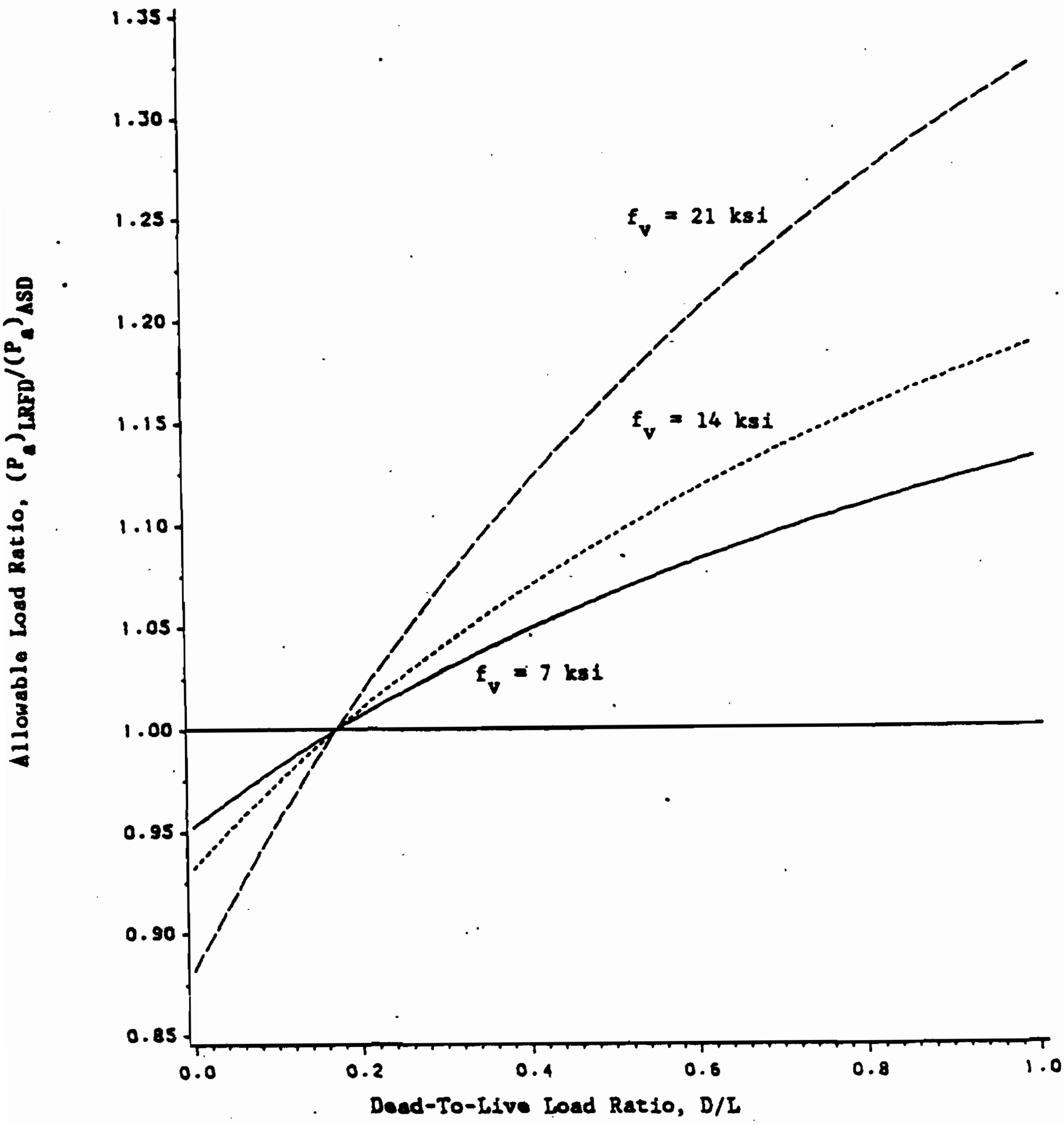

Figure 65 Allowable Load Ratio vs. D/L Ratio for Tension Strength on A325 Bolts Subject to the Combination of Shear and Tension (Threads are Included in Shear Plane)

VIII. CONCLUSIONS

A. GENERAL

The Allowable Stress Design (ASD) method has long been used for the design of cold-formed steel structural members in the United States, and other countries. Recently, the Load and Resistance Factor Design (LRFD) method has been successfully applied to the design of steel buildings using hot-rolled shapes, and built-up members fabricated from steel plates.

In order to develop the reliability-based design criteria for cold-formed steel members, a joint research project entitled "Load and Resistance Factor Design (LRFD) of Cold-Formed Steel" was conducted at the University of Missouri-Rolla, Washington University, and the University of Minnesota. This study included the selection of a reliability analysis model; the evaluation of load factors; the calibration of the design provisions; the determination of resistance factors; and the comparative study of design methods for cold-formed steel structural members. Based on the results of this study, the new LRFD Specification for the design of cold-formed steel structural members and connections has been developed.

B. SUMMARY OF THE STUDY

The LRFD criteria for cold-formed steel structural members were based on the limit states of strength and serviceability of thin-walled steel structural members. The mean-value first-order second-moment reliability analysis and the advanced reliability analysis were used as basic methods in the development of the LRFD criteria.

As the first step of the investigation, numerous technical papers and research reports relative to the theoretical concepts of the structural reliability were reviewed in Section II. This section also contains the statistical data on material properties and cross sectional properties, determination of target reliability indices, and recommended formulas for the determination of structural reliability.

The selection of load factors and load combinations to be used in the LRFD criteria for cold-formed steel members is included in Section III. Load factors and load combinations recommended in the 1982 ANSI code were adopted and modified on the basis of the special circumstances inherent in cold-formed steel structures.

For the purpose of facilitating the steps used in the calibration of various provisions of the AISI Specification, the calibration procedures were formulated and summarized in Section IV. All the resistance factors, as well as, reliability indices used for various design provisions were based on the formulas derived in Section IV.

The development of the LRFD criteria for cold-formed steel members and connections is presented in Section V. This section contains the determination of nominal strengths and corresponding resistance factors for tension members, flexural members, concentrically loaded compression members, beam-column members, stiffeners, wall studs, welded connections, and bolted connections.

Due to the small number of test data, the calibration procedure derived in Section IV can not be applied directly to the evaluation procedure of tests for determining structural performance. In Section VI, the LRFD procedure for determining structural performance on the basis of special tests was developed by using a reduction factor applied to the coefficient of variation of the professional factor V_P.

Finally, comparative studies of both ASD and LRFD methods for cold-formed steel members were conducted and presented in Section VII. This comparison involved studies of different variables used for the design of various types of structural members and discussions of different load-carrying capacities determined by these two methods.

C. CONCLUSIONS

The following conclusions can be drawn from the development of the LRFD criteria, and the comparative study of design methods for cold-formed steel structural members and connections.

1. Development of the LRFD Criteria. In the development of the LRFD criteria for cold-formed steel members and connections, the load factors and load combinations were based on the 1982 ANSI Code, while the nominal strengths used for various design provisions were based on the 1986 AISI Specification. The resistance factors were determined on the basis of the target reliability indices using a calibration procedure. By using the LRFD criteria, designs can achieve a more consistent reliability than the ASD criteria.

2. Comparative Study of Design Methods. In the comparative study, it was found that the D/L and D/W ratios have a significant effect on the allowable load ratio, $(P_a)_{LRFD}/(P_a)_{ASD}$. In general, the allowable load ratio increases as the D/L ratio increases or D/W ratio decreases. Because

cold-formed steel is usually used for light-weight members, the D/L and D/W ratios of such members are expected to be lower than the ratios used for other building materials. In this study, D/L = 0.2 and D/W = 0.1 are used as the representative values for cold-formed steel structures.

In addition to the effect of D/L and D/W ratios on the allowable load ratio, the resistance factors used in the LRFD criteria, and the factors of safety used in the allowable stress design, also contribute to the differences between these two different methods. For members subjected to a combination of loads, or load effects, the differences between the ASD and LRFD methods are affected by the cross-sectional geometry, loading condition, material strength, and member length.

The LRFD method is a more rational approach for structural design as compared with the ASD method. Therefore, the research findings obtained from the comparative study of these two methods can provide a useful reference for future improvement of the current AISI ASD Specification.

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