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Load and resistance factor design of cold-formed steel comparative study of design methods for cold-formed steel

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LOAD AND RESISTANCE FACTOR DESIGN OF COLD-FORMED STEEL

COMPARATIVE STUDY OF DESIGN METHODS FOR COLD-FORMED STEEL

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PREFACE

This progress report is the revision of the Eighth Progress Report on the Load and Resistance Factor Design of Cold-Formed Steel. Revisions have been made to reflect some recent changes of resistance factors and design formulas in the proposed LRFD Specification (UMR Civil Engineering Study 88-3) dated February 1988.

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ABSTRACT

Allowable Stress Design is the current method used to design cold-formed steel structural members and connections. In this design approach, factors of safety are used to compute the allowable design stresses which are compared to the actual maximum stresses that will occur in the member during the life of the structure.

In recent years, the Load and Resistance Factor Design (LRFD) method has been developed for the design of hot-rolled steel shapes and the design of cold-formed steel structural members. This method is based on probabilistic and statistical techniques to account for the many uncertainties involved with the actual design. The LRFD criteria use load factors which are applied to the external loads and resistance factors that are applied to the internal resistance capacities of the structure.

The allowable unfactored loads based on each design method for different types of structural members are compared and shown in graphical forms. For structural members with one type of loading, the dead-to-live load ratio contributes to the difference between the two allowable loads. For members with a combination of loads, cross-sectional geometry, loading conditions, material strength, member length, along with dead-to-live load ratio will affect the difference between the allowable loads computed from allowable stress design and LRFD.

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I. INTRODUCTION

A. GENERAL

The 1986 Edition of the Specification for the Design of Cold-Formed Steel Structural Members published by the American Iron and Steel Institute (AISI) applies to steel members cold-formed to shape from carbon or low-alloy steel sheet, strip, plate or bar not more than one inch in thickness and used for load-carrying purposes in buildings[1]. The specification is based upon the allowable stress concept presented in terms of allowable moments and loads. In the design of cold-formed steel members and connections, the actual moments and loads are computed from service loads that include dead, live, snow, wind, and earthquake loads. The allowable moments and loads are determined by dividing the corresponding nominal capacities by appropriate factors of safety recommended by AISI for different types of structural members and connections.

The Load and Resistance Factor Design (LRFD) criteria for steel members and connections have recently been developed by using probabilistic and statistical techniques to account for the uncertainties in design, fabrication, material properties, and applied loads. The LRFD criteria for hot-rolled shapes, built-up members, and connections[2] have been included in the Load and Resistance Factor Design Manual of Steel Construction published by the American Institute of Steel Construction[3]. For cold-formed steel structural members, the Load and Resistance Factor Design Specification was developed from a joint research project entitled "Load and Resistance Factor Design of Cold-

Formed Steel" conducted at the University of Missouri-Rolla, Washington University and the University of Minessota[4-13].

B. PURPOSE OF INVESTIGATION

The primary purpose of this investigation was to study and compare the proposed Load and Resistance Factor Design (LRFD) Criteria for Cold-Formed Steel[13] with the existing Allowable Stress Design (ASD) Criteria included in the 1986 Specification for the Design of Cold-Formed Steel Structural Members[1]. 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. SCOPE OF INVESTIGATION

This study compares the existing Allowable Stress Design Method with the proposed 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 proposed LRFD Specification 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 proposed LRFD criteria were prepared for any combinations of different loads, only dead and live loads were used in this comparison for each type of structural members. Ratios of load carrying capacities were computed and evaluated for different shapes of structural members which are used in typical design situations.

II. REVIEW OF LITERATURE

A. GENERAL

Because of the growing need for a unified approach to structural design for all types of construction materials, many studies have been conducted in recent years. In early 1978, the LRFD criteria for hot-rolled steel shapes[14] were proposed by Galambos as alternative design methods. This proposal was a result of a research project conducted at Washington University under the sponsorship of the American Iron and Steel Institute. This subject was subsequently discussed by Galambos, Ravindra, Yura, Bjorhovde, Cooper, Hansell, Viest, Fisher, Kulak, and Cornell in References 15 through 22. In addition, numerous papers were publisbed in the proceedings of the American Society of Civil Engineers (ASCE) Specialty Conference on Probabilistic Mechanics and Structural Reliability held in January 1979. In Reference 23, Grigoriu, Veneziano, and Cornell discuss the importance of decision making in probability distribution modeling. Chalk and Cortis studied a collection of live load data to develop a probabilistic format for the determination of design live loads for building floors[24].

During the period from 1979 to 1982, Ellingwood studied statistical information in reinforced concrete[25,26], wood[27], and masonry[28] structures for developing a probability-based limit states design criteria. In a recent study sponsored by the National Bureau of Standards, Galambos, Ellingwood, MacGregor, and Cornell developed a set of load factors, load combinations, and methodology for material specification groups[29-31]. More recently, the ASCE Committee on Fatigue and Fracture Reliability published a series of reports on fatigue reliability[32-34].

With regard to cold-formed steel design, a study on reliability based criteria for temporary cold-formed steel buildings was conducted by Knab and Lind[35] in 1975. A joint research project entitled "Load and Resistance Factor Design of Cold-Formed Steel" was conducted by Rang, Supornsilaphachai, Snyder, Pan, Galambos, and Yu at the University of Missouri-Rolla and Washington University since 1976. This project was also under the sponsorship of AISI. References 4 through 8 summarized the studies of the LRFD criteria for cold-formed steel tension members, beams, columns, beam-columns, and connections. The research findings have been discussed at various engineering and specialty conferences and published in several conference proceedings[36-40]. In September 1985, the Tentative Recommendations on the LRFD Criteria for Cold-Formed Steel Structural Members and Commentary[10] were prepared according to the 1980 edition of the AISI Specification for the allowable stress design. These tentative recommendations were updated in 1987[13] on the basis of the 1986 edition of the AISI Specification[1].

In Canada, the Canadian Standards Association uses the limit states design principles in their standard for cold-formed steel[41].

B. LOAD AND RESISTANCE FACTOR DESIGN CRITERIA

The Load and Resistance Factor Design Specification for Cold-Formed Steel[13] is based on the first-order principles of probabilistic theory. The general format for the LRFD criteria is

$$\phi R_n \geq \sum_{k=1}^{j} \gamma_k Q_{kn} \qquad \text{(II.1)}$$

In the above,

ϕ = resistance factor

R_n = nominal resistance

γ_k = load factors

Q_{kn} = nominal load effects

On the left side of Eq. (II.1), the resistance factor, ϕ, is a nondimensional factor less than or equal to one that accounts for the uncertainties in calculating the nominal resistance. The nominal resistance of the structure is the predicted ultimate resistance or load determined from design formulas using specified mechanical properties of material and section properties. It could be a bending moment, axial load, shear force, or an interaction formula when load combinations are presented.

On the right side of the equation, factor γ is a nondimensional load factor used to reflect the possiblity of overloads and uncertainties in computing the load effect. Each load factor applies to a nominal load effect Q_n and the subscript k corresponds to different types of loads. Only dead and live load effects were used to develop the LRFD criteria for cold-formed steel.

Instead of a safety factor, a safety index is used to determine structural reliability. The safety index, β, indicates the probability of failure as shown in Figure 1. The distribution of the R/Q ratio was assumed to be lognormal. The safety index can be determined by using Eq. (II.2):[4,42]

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

where

R_m = mean value of resistances

Q_m = mean value of load effects

V_R = coefficient of variation of resistances

V_Q = coefficient of variation of load effects

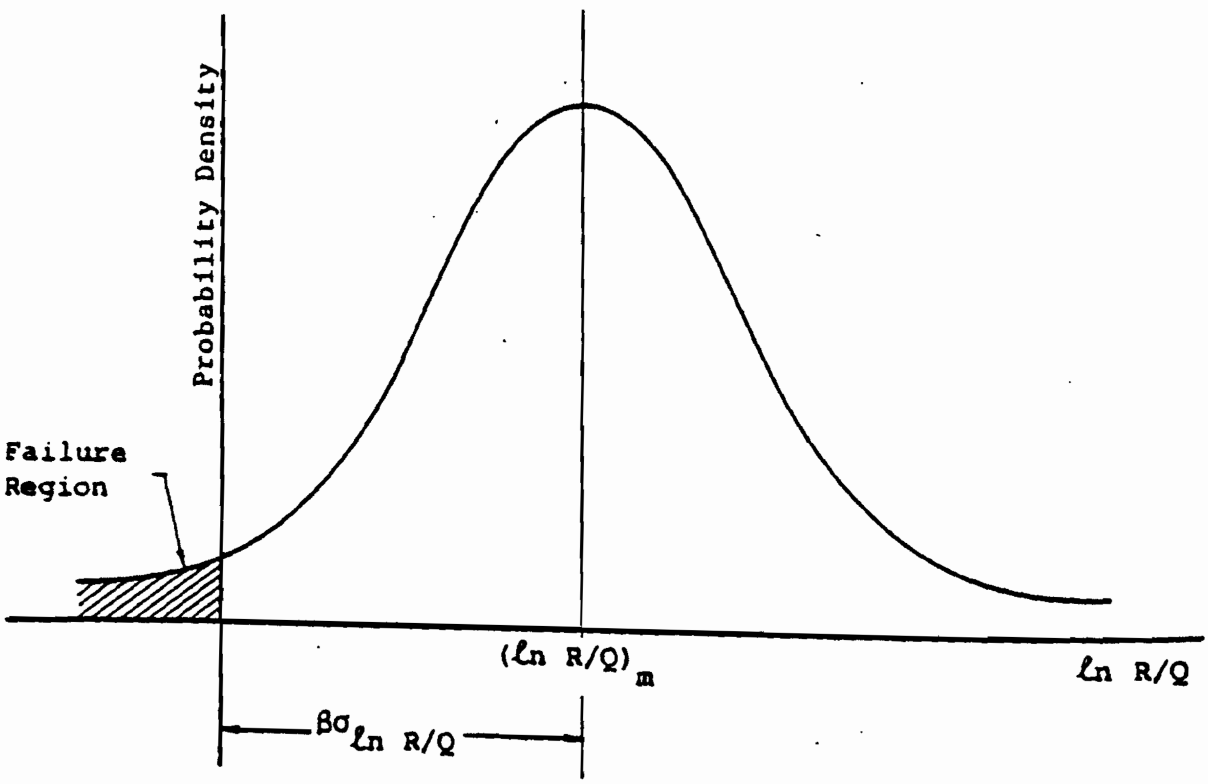

Figure 1. Probability Distribution of ℓn R/Q

The target values of safety index used in the development of the LRFD Specification for cold-formed structural members and connections are 2.5 and 3.5, respectively. A probability of failure of 9.8×10^{-3} is obtained from the cumulative lognormal distribution for the value of safety index equal to 2.5[42].

Unlike the traditional design methods, the resistance of the structure is considered to be a random variable because of variations in mechanical properties and fabrication uncertainties involved in calculations of the resistance. The mean value of the resistances was assumed to be a product of several values as given in Eq. (II.3).

$$R_m = R_n M_m F_m P_m \tag{II.3}$$

where M_m, F_m and P_m are the mean values of nondimensional variables reflecting the uncertainties in mechanical properties, sectional properties, and calculation of the resistance.

In Eq. (II.3), M is the material factor which is determined by the ratio of the tested mechanical properties to the specified values. Mechanical properties include yield point, modulus of elasticity, and tensile strength values. The fabrication factor, F, accounts for variations of geometric dimensions and uncertainties caused by initial imperfections and tolerances. The professional factor, P, accounts for uncertainties that results from the use of approximations and simplifications of complex design formulas based on ideal situations. It is obtained from the ratio of the tested failure loads to the predicted failure loads computed from design formulas.

From statistical studies of applied loads and reliability calculations[30,31], the following load combinations and load factors were used for cold-formed steel:[43]

1. $1.4D_n+L_n$
2. $1.2D_n+1.6L_n+0.5(L_{rn}$ or S_n or $R_n)$
3. $1.2D_n+1.6(L_{rn}$ or S_n or $R_n)+(0.5L_n$ or $0.8W_n)$
4. $1.2D_n+1.3W_n+0.5L_n+0.5(L_{rn}$ or S_n or $R_n)$
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

L_{rn} = nominal roof live load

R_n = nominal roof rain load

S_n = nominal snow load

W_n = nominal wind load (Exception: For wind load on individual purlins, girts, wall panels and roof decks, multiply the load factor for W_n by 0.9)

Exception: The load factor on L_n in combination (3), (4), and (5) shall be equal to 1.0 for garages, areas occupied as places of public assembly, and all areas where the live load is greater than 100 psf.

For roof and floor construction, the load combination for dead load, weight of wet concrete, and construction load including equipment, workmen and formwork is suggested in Section A5.1.(2)(a) of the Commentary.[13]

When the structural effects of F, H, P, or T are significant, they shall be considered in design as the following factored loads: 1.3F, 1.6H, 1.2P, and 1.2T, where

F = loads due to fluids with well-defined pressures and maximum heights

H = loads due to the weight and lateral pressure of soil and water in soil

P = loads, forces, and effects due to ponding

T = self-straining forces and effects arising from contraction or expansion resulting from temperature changes, shrinkage, moisture changes, creep in component materials, movement due to differential settlement, or combinations thereof

The preceding load combinations are listed in Section A5.1.4 of the LRFD Specification[13] and should be used in the computation of the load effects. The combination of dead and live load with an assumed dead-to-live load ratio of 1/5 were used to develop the LRFD criteria for cold-formed steel.

The coefficient of variation of the resistances, V_R, is related to the coefficient of variation of M, F, and P as follows:

$$V_R = \sqrt{V_M^2+V_F^2+V_P^2} \qquad \text{(II.4)}$$

The coefficient of variation of the load effects, V_Q, can be computed from the nominal dead-to-live load ratio and the coefficient of variation of the dead and live loads. For a dead-to-live load ratio equal to 1/5, V_Q is equal to 0.21.

The resistance factor can be obtained from the following equation developed in Reference 10.

$$\phi = \frac{1.521 M_m F_m P_m}{\exp(\beta\sqrt{V_R^2+V_Q^2})} \qquad \text{(II.5)}$$

All statistical data and calculations for material factors, fabrication factors, professional factors, coefficients of variation of resistances, and resistance factors can be found in References 4 through 13.

In the LRFD criteria, the factored nominal resistance for design 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:

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

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

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

Therefore,

$$c(D_n+L_n) \leq \frac{R_n}{(1.2D_n/L_n+1.6)/[\phi(D_n/L_n+1)]} \qquad \text{(II.6)}$$

where c is the deterministic influence coefficient to transform the load to load effect.

From Eq. (II.6), the factor of safety against the nominal resistance used in the LRFD is:

$$(F.S.)_{LRFD} = (1.2D_n/L_n+1.6)/[\phi(D_n/L_n+1)] \qquad \text{(II.7)}$$

Equation (II.6) was used in this study to compare the AISI Specification for allowable stress design and the Load and Resistance Factor Design Specification. The results are presented and discussed in Chapters III through IX.

III. TENSION MEMBERS

A. ALLOWABLE STRESS DESIGN (ASD)

According to Section C2 of the AISI Specification[1], cold-formed steel tension members should be designed to satisfy the following requirement:

"For axially loaded tension members, the applied tensile force shall not exceed T_a determined as follows:

$$T_a = T_n/\Omega_t \qquad \text{(III.1)}$$

where

T_n = Strength of member when loaded in tension

$= A_n F_y$ (III.2)

Ω_t = Factor of safety for tension

$= 1.67$

A_n = Net area of the cross section

F_y = Design yield stress."

B. LOAD AND RESISTANCE FACTOR DESIGN (LRFD)

Based on Section C2 of the proposed LRFD Specification[13], the following provisions are used for the design of cold-formed steel tension members:

"For axially loaded tension members, the factored nominal tensile strength, ϕT_n, shall be determined as follows:

$\phi = 0.95$

$$T_n = A_n F_y \qquad \text{(III.3)}$$

where

ϕ = Resistance factor for tension

T_n = Nominal strength of member when loaded in tension

A_n = Net area of the cross section

F_y = Design yield stress."

C. COMPARISON

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:

$$P_T = P_{DL}+P_{LL} \tag{III.4}$$

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

$$(P_a)_{ASD} = A_nF_y/\Omega_t = A_nF_y/1.67 \tag{III.5}$$

For LRFD, the allowable load can be calculated by using Eq. (II.6).

$$(P_a)_{LRFD} = \phi T_n(D/L+1)/(1.2D/L+1.6) \tag{III.6}$$

Because $T_n = A_nF_y$, Eq. (III.6) can be rewritten as

$$(P_a)_{LRFD} = \phi A_nF_y(D/L+1)/(1.2D/L+1.6) \tag{III.7}$$

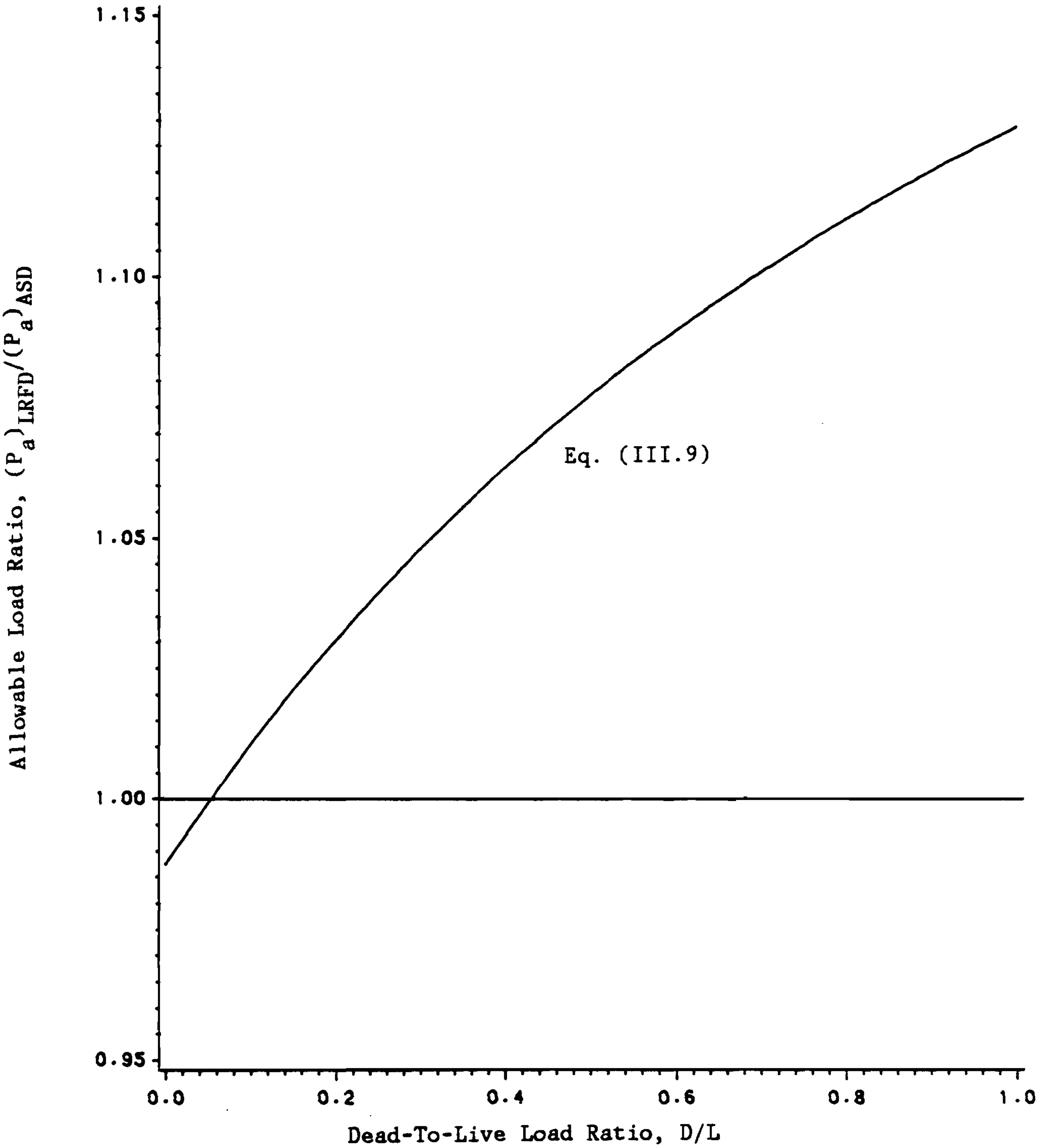

Figure 2. Allowable Load Ratio vs. D/L Ratio for Tension

where D/L is the ratio of the nominal dead load to the nominal live load. From Eq. (III.7) 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. (III.5) and (III.7), the allowable load ratio for tension members is

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.67\phi \frac{D/L+1}{1.2D/L+1.6} \tag{III.8}$$

For the value of $\phi = 0.95$

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.58 \frac{D/L+1}{1.2D/L+1.6} \tag{III.9}$$

Figure 2 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.

IV. FLEXURAL MEMBERS

A. GENERAL

Cold-formed steel flexural members have several possible modes of failure. In the design of beams, consideration should first be given to the section strength based on either initiation of yielding in the effective section or the inelastic reserve capacity as applicable. For beams with inadequate lateral bracing, lateral buckling may limit the moment-resisting capacity. Beam webs have to be designed for shear and combined bending and shear. Because of highly localized concentrations of stress resulting from applied concentrated loads or reactions, web crippling and combined bending and web crippling have to be checked. Excessive deflection due to service live load could also be a problem.

B. STRENGTH FOR BENDING ONLY

1. Allowable Stress Design. Based on Section C3.1 of the AISI Specification, the following provisions are used for the design of cold-formed steel flexural members based on bending strength:

"In flexural members, the applied moment uncoupled from axial load, shear, and local concentrated forces or reactions shall not exceed the allowable M_a calculated as follows:

$$M_a = M_n/\Omega_f \qquad \text{(IV.1)}$$

where

M_n = Smaller of the nominal moment strength calculated according to Sections C3.1.1 and C3.1.2

Ω_f = Factor of Safety for bending

= 1.67

C3.1.1 Nominal Section Strength

Section strength 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.

(a) Procedure I - Based on Initiation of Yielding

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

$$M_n = S_e F_y \tag{IV.2}$$

where

F_y = Design yield stress as determined in Section A5.2.1

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

(b) Procedure II - Based on Inelastic Reserve Capacity

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

(1) The member is not subject to twisting or to lateral, torsional, or torsional-flexural buckling.

(2) The effect of cold forming is not included in determining the yield point F_y.

(3) The ratio of the depth of the compressed portion of the web to its thickness does not exceed λ_1.

(4) The shear force does not exceed $0.35F_y$ times the web area, h x t.

(5) The angle between any web and the vertical does not exceed 30 degrees.

The nominal moment 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_ye_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 $w/t \le \lambda_1$

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

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

where

$$\lambda_1 = 1.11/\sqrt{F_y/E} \quad \text{(IV.3)}$$

$$\lambda_2 = 1.28/\sqrt{F_y/E} \quad \text{(IV.4)}$$

(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 defined in Section B3.1 shall be used in calculating section properties. M_n shall be calculated considering equilibrium of stresses, assuming an ideally elasticplastic stress-strain curve which is the same in tension as in compression, assuming small deformation and assuming that plane sections remain plane during bending. Combined bending and web crippling shall be checked by provisions of Section C3.5.

C3.1.2 Lateral Buckling Strength

For the laterally unbraced segments of doubly- or singly-symmetric sections subjected to lateral buckling, M_n shall be determined as follows:

$$M_n = S_c(M_c/S_f) \qquad \text{(IV.5)}$$

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-sections bent about the centroidal axis perpendicular to the web (x-axis):

For $M_e \geq 2.78M_y$

$$M_c = M_y \qquad \text{(IV.6)}$$

For $2.78M_y > M_e > 0.56M_y$

$$M_c = (10/9)M_y(1\text{-}10M_y/36M_e) \qquad \text{(IV.7)}$$

For $M_e \leq 0.56M_y$

$$M_c = M_e \qquad \text{(IV.8)}$$

where

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

$= S_f F_y$ (IV.9)

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

$= \pi^2 E C_b (d I_{yc}/L^2)$ for doubly-symmetric I-sections (IV.10)

$= \pi^2 E C_b d I_{yc}/(2L^2)$ for point-symmetric Z-sections (IV.11)

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 $M_e > 0.5M_y$

$M_c = M_y(1-M_y/4M_e)$ (IV.12)

For $M_e \leq 0.5M_y$

$M_c = M_e$ (IV.13)

where

M_y is as defined in (a) above

M_e = Elastic critical moment

M_e $= C_b r_0 A\sqrt{\sigma_{ey}\sigma_t}$ 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

(IV.14)

$M_e \quad = C_s A\sigma_{ex}[j+C_s\sqrt{j^2+r_0^2(\sigma_t/\sigma_{ex})}]/C_{TF}$

for bending centrodial axis perpendicular to the symmetry axis (IV.15)

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

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

$$\sigma_{ex} = \pi^2E/(K_xL_x/r_x)^2 \quad \text{(IV.16)}$$

$$\sigma_{ey} = \pi^2E/(K_yL_y/r_y)^2 \quad \text{(IV.17)}$$

$$\sigma_t = 1/(Ar_0^2)[GJ+\pi^2EC_w/(K_tL_t)^2] \quad \text{(IV.18)}$$

A = Full cross-sectional area

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

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

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 (Section C5), 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 (Section C5), C_{TF} shall be taken as unity.

r_0 = Polar radius of gyration of the cross section about the shear center

$$= \sqrt{r_x^2+r_y^2+x_0^2} \qquad \text{(IV.19)}$$

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_0 = 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

$j = 1/(2I_y)(\int_A x^3 dA + \int_A xy^2 dA) - x_0$ (IV.20)"

2. <u>LRFD Criteria.</u> Based on Section C3.1 of the LRFD Specification[13], for flexural members subjected only to bending moment, the factored nominal bending strength, ϕM_n, shall be the smaller of the values calculated according to nominal section strength and lateral buckling strength.

For nominal section strength, the factored nominal bending strength, ϕM_n, shall be determined with $\phi = 0.95$ and 0.90 for sections with stiffened compression flanges and unstiffened compression flanges, respectively, and the nominal section strength, M_n, calculated exactly the same as that specified in Section C3.1.1 of the AISI Specification.

For lateral buckling strength, the factored nominal strength of the laterally unbraced segments of doubly- or singly-symmetric sections subjected to lateral buckling, ϕM_n, shall be determined with $\phi = 0.90$ and M_n calculated exactly the same as that specified in Section C3.1.2 of the AISI Specification.

3. <u>Comparison.</u> The unfactored moment can be calculated by using Eq. (IV.21) for both methods (ASD and LRFD) for comparison.

$$M_{TL} = M_{DL} + M_{LL} \qquad \text{(IV.21)}$$

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

$$(M_a)_{ASD} = M_n/\Omega_f = M_n/1.67 \qquad \text{(IV.22)}$$

For LRFD, the allowable moment can be computed by using the following equation developed from Eq. (II.6).

$$(M_a)_{LRFD} = \phi M_n(D/L+1)/(1.2D/L+1.6) \qquad \text{(IV.23)}$$

The ratio of the allowable moments for both nominal section strength and lateral buckling strength is

$$\frac{(M_a)_{LRFD}}{(M_a)_{ASD}} = 1.67\phi \frac{D/L+1}{1.2D/L+1.6} \qquad \text{(IV.24)}$$

For nominal section strength of sections with stiffened compression flanges, $\phi = 0.95$

$$\frac{(M_a)_{LRFD}}{(M_a)_{ASD}} = 1.58 \frac{D/L+1}{1.2D/L+1.6} \qquad \text{(IV.25)}$$

Figure 3 shows the allowable moment ratio versus dead-to-live load ratio for beams with 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, $\phi = 0.90$

$$\frac{(M_a)_{LRFD}}{(M_a)_{ASD}} = 1.50 \frac{D/L+1}{1.2D/L+1.6} \qquad \text{(IV.26)}$$

Figure 4 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 for cold-formed steel is less than 1/3, the LRFD criteria are found to be conservative for nominal section strength of sections with unstiffened compression flanges and lateral buckling as compared with the allowable stress design method.

C. STRENGTH FOR SHEAR ONLY

There are three possible modes of shear failure in beam webs. For a relatively small h/t ratio, shear yielding will be the failure mode. For webs with large h/t ratios, the webs will fail in elastic shear buckling. For moderate values of h/t, the shear buckling will be in the inelastic range.

1. Allowable Stress Design. The shear force at any section shall not exceed the allowable shear, V_a, specified in Section C3.2 of the AISI Specification as follows:

(a) For $h/t \leq 1.38\sqrt{Ek_v/F_y}$

$$V_a = 0.38t^2\sqrt{k_vF_yE} \leq 0.4F_yht \qquad \text{(IV.27)}$$

(b) For $h/t > 1.38\sqrt{Ek_v/F_y}$

$$V_a = 0.53Ek_vt^3/h \qquad \text{(IV.28)}$$

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:

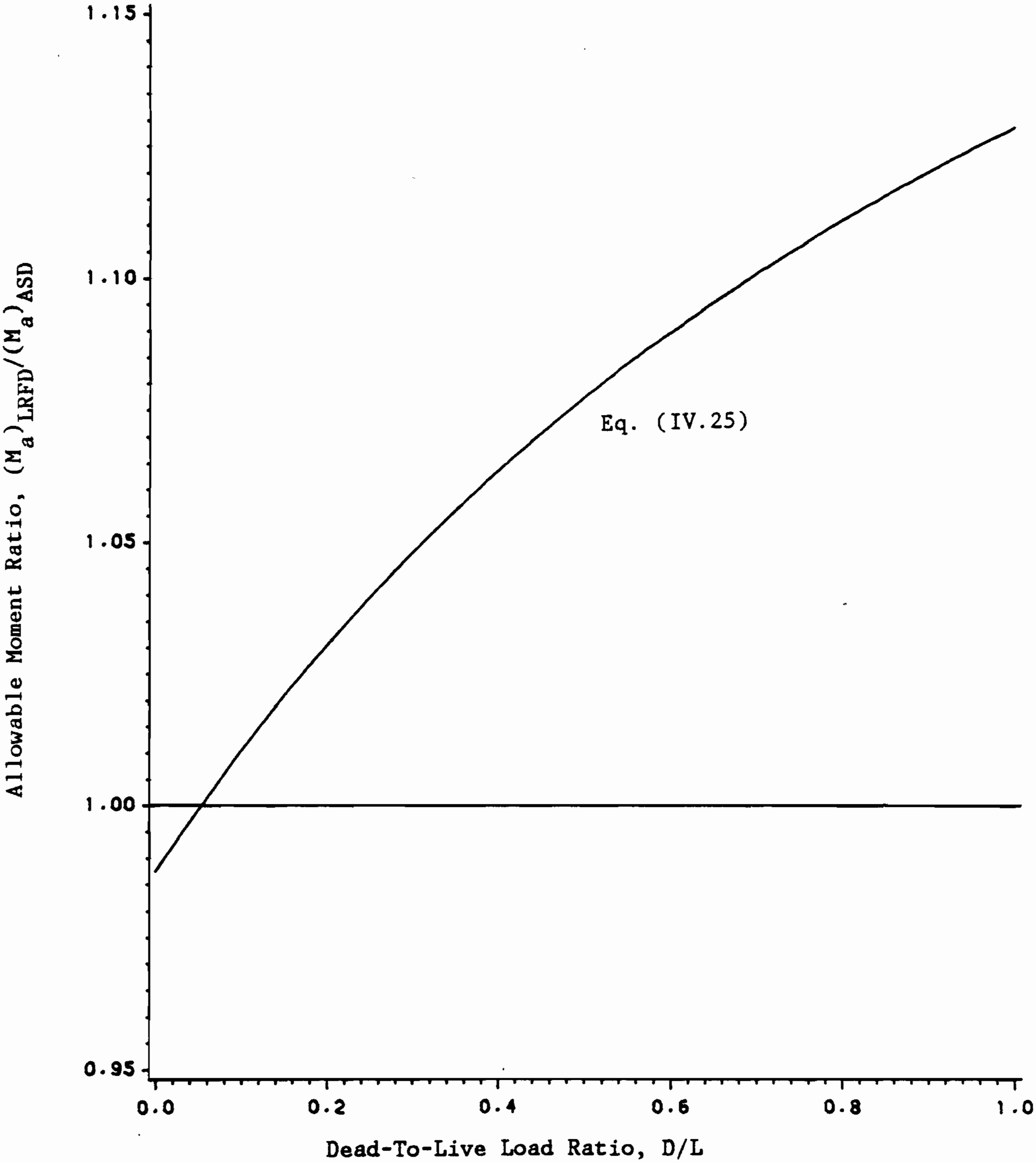

Figure 3. Allowable Moment Ratio vs. D/L Ratio for Bending Strength of Beams With Stiffened Compression Flanges

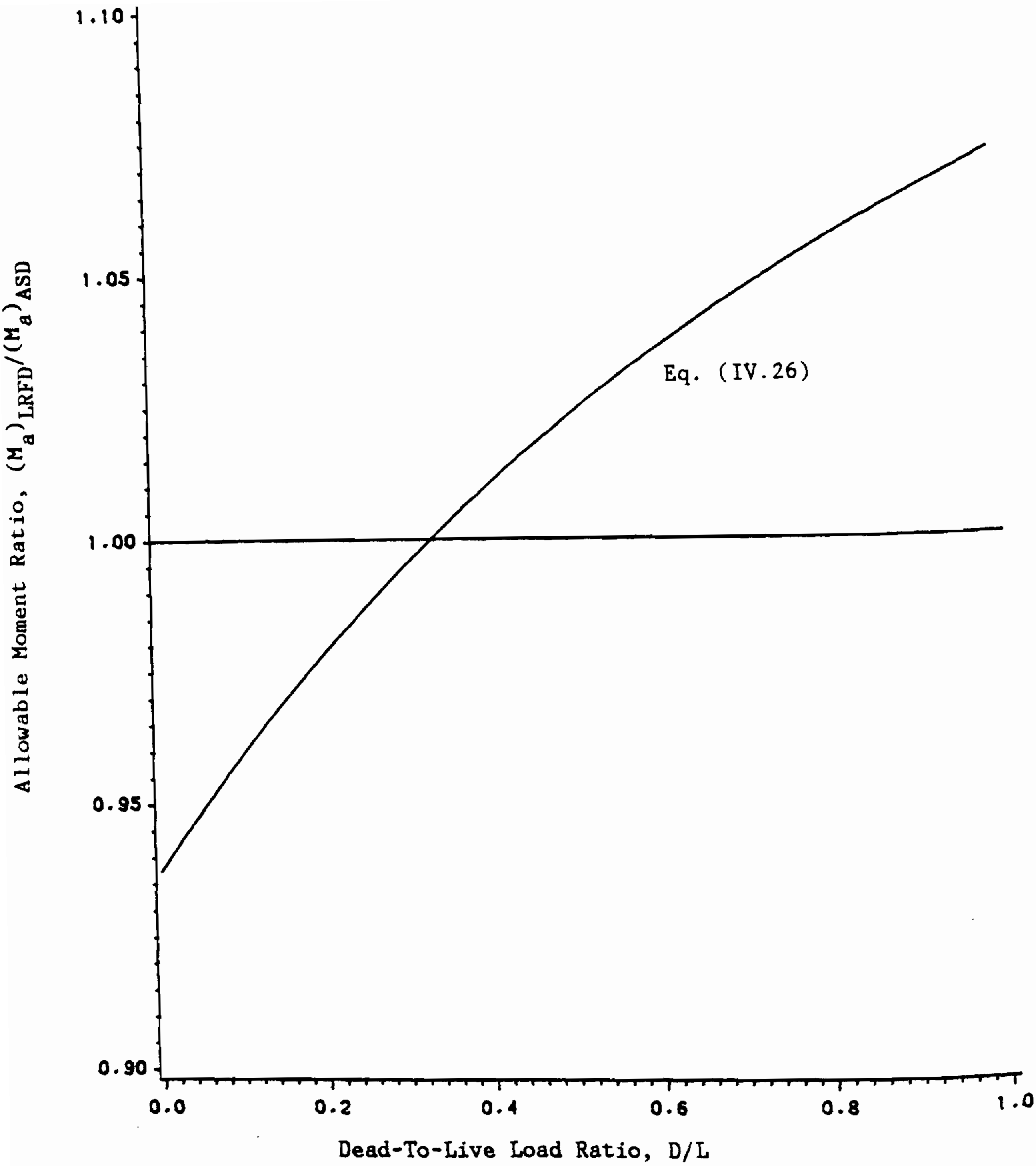

Figure 4. Allowable Moment Ratio vs. D/L Ratio for Section Strength of Beams With Unstiffened Compression Flanges and Lateral Buckling of Beams

1. For unreinforced webs, $k_v = 5.34$

2. For beam webs with transverse stiffeners satisfying the requirements of Section B6

when $a/h \le 1.0$

$$k_v = 4.00+5.34/(a/h)^2 \qquad \text{(IV.29)}$$

when $a/h > 1.0$

$$k_v = 5.34+4.00/(a/h)^2 \qquad \text{(IV.30)}$$

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 seperate element carrying its share of the shear force.

2. <u>LRFD Criteria.</u> According to Section C3.2 of the LRFD Specification, the factored nominal shear strength, $\phi_v V_n$, at any section shall be calculated as follows:

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

$\phi_v = 1.0$

$$V_n = 0.577F_y ht \qquad \text{(IV.31)}$$

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

$\phi_v = 0.90$

$$V_n = 0.64t^2\sqrt{k_v F_y E} \qquad \text{(IV.32)}$$

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

$\phi_v = 0.90$

$$V_n = 0.905Ek_v t^3/h \qquad \text{(IV.33)}$$

where

ϕ_v = Resistance factor for shear

V_n = Nominal shear strength of the beam

3. <u>Comparison.</u> The unfactored shear force can be calculated for both ASD and LRFD methods by using the following equation.

$$V_T = V_{DL}+V_{LL} \tag{IV.34}$$

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

$$(V_a)_{ASD} = V_n/\Omega \tag{IV.35}$$

For LRFD, the allowable shear load equation was developed from Eq. (II.6) and is

$$(V_a)_{LRFD} = \phi_v V_n(D/L+1)/(1.2D/L+1.6) \tag{IV.36}$$

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. For example, 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.

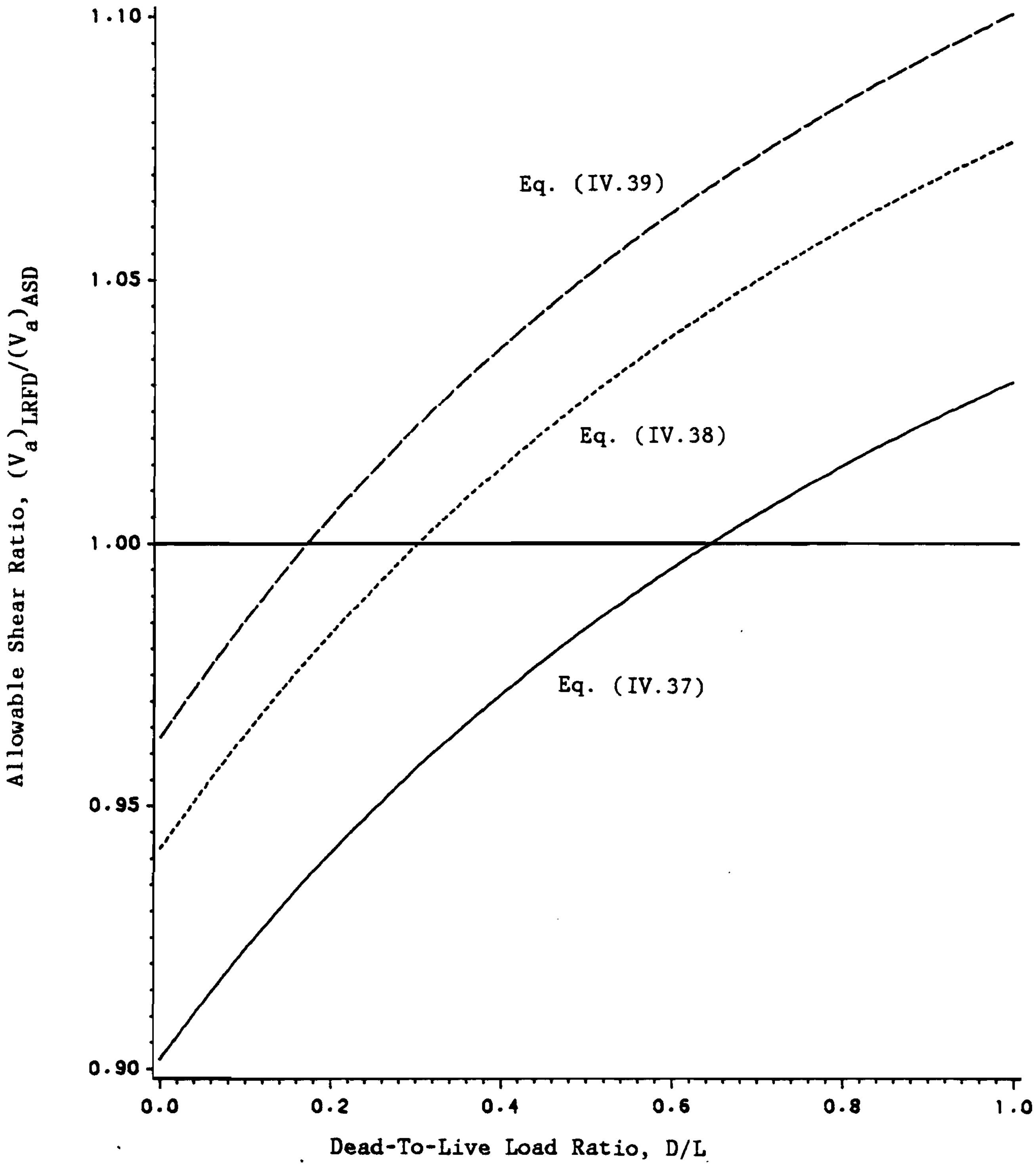

Figure 5. Allowable Shear Ratio vs. D/L Ratio for Shear Strength of Beams

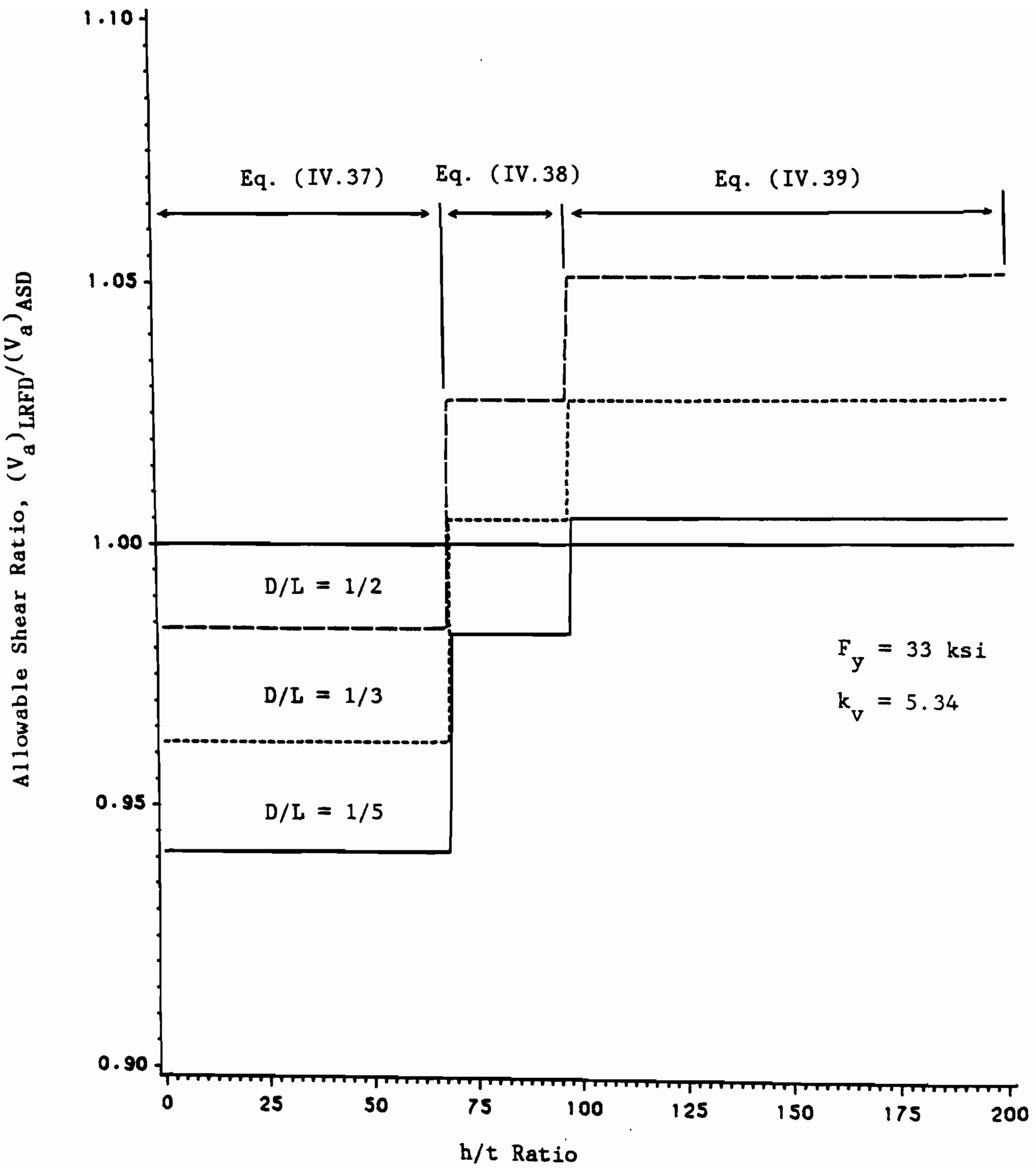

Figure 6. Allowable Shear Ratio vs. h/t Ratio for Shear Strength of Beams

The allowable shear ratios are:

For $h/t \le \sqrt{Ek_v/F_y}$ and $\phi_v = 1.0$,

$$\frac{(V_a)_{LRFD}}{(V_a)_{ASD}} = 1.443\phi_v \frac{D/L+1}{1.2D/L+1.6} = 1.443 \frac{D/L+1}{1.2D/L+1.6} \qquad \text{(IV.37)}$$

For $\sqrt{Ek_v/F_y} < h/t \le 1.38\sqrt{Ek_v/F_y}$ and $\phi_v = 0.90$

$$\frac{(V_a)_{LRFD}}{(V_a)_{ASD}} = 1.674\phi_v \frac{D/L+1}{1.2D/L+1.6} = 1.507 \frac{D/L+1}{1.2D/L+1.6} \qquad \text{(IV.38)}$$

For $h/t > 1.415\sqrt{Ek_v/F_y}$ and $\phi_v = 0.90$

$$\frac{(V_a)_{LRFD}}{(V_a)_{ASD}} = 1.712\phi_v \frac{D/L+1}{1.2D/L+1.6} = 1.541 \frac{D/L+1}{1.2D/L+1.6} \qquad \text{(IV.39)}$$

Figure 5 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 6, 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.

D. STRENGTH FOR COMBINED BENDING AND SHEAR

For continuous beams and cantilevers, maximum bending stress and shear stress act simultaneously at supports. The beams will fail at a lower stress than if only one stress were present. The interaction between bending and shear must be checked.

1. Allowable Stress Design. For beams subjected to both bending and shear, the following equations should be satisfied in accordance with Section C3.3 of the AISI Specification.

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

$$(M/M_a)^2+(V/V_a)^2 \leq 1.0 \qquad \text{(IV.40)}$$

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

$$0.6(M/M_a)+(V/V_a) \leq 1.3 \qquad \text{(IV.41)}$$

In the above equations:

M_a = Allowable moment determined according to Section C3.1.1 when bending alone exists excluding lateral buckling

V_a = Allowable shear force when shear alone exists

2. LRFD Criteria. Section C3.3 of the LRFD Specification specifies that for beams with unreinforced webs, the bending moment, M_D, and the shear force, V_D, computed on the basis of the factored loads shall satisfy the following interaction equation:

$$(M_D/\Phi M_n)^2+(V_D/\Phi_v V_n)^2 \leq 1.0 \qquad \text{(IV.42)}$$

For beams with transverse web stiffeners, the bending moment, M_D, and the shear force, V_D, computed on the basis of the factored loads shall not exceed ϕM_n and $\phi_v V_n$, respectively. When $M_D/(\phi M_n) > 0.5$ and $V_D/(\phi_v V_n) > 0.7$, then M_D and V_D shall satisfy the following interaction equation:

$$0.6(M_D/\phi M_n)+(V_D/\phi_v V_n) \leq 1.3 \qquad \text{(IV.43)}$$

In the above equations:

ϕ = Resistance factor for bending

ϕ_v = Resistance factor for shear

M_n = Nominal ultimate bending moment determined according to Section C3.1.1 when bending alone exists

V_n = Nominal ultimate shear strength when shear alone exists

3. <u>Comparison.</u> 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.

$$M_{TL} = M_{DL}+M_{LL} = c_m w_T L^2 \qquad \text{(IV.44)}$$

$$V_T = V_{DL}+V_{LL} = c_v w_T L \qquad \text{(IV.45)}$$

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:

$$\frac{M}{M_a} = \frac{M_{TL}}{0.6M_n} = \frac{1.667 c_m w_T L^2}{M_n} \qquad \text{(IV.46)}$$

For h/t $\le 1.38\sqrt{Ek_v/F_y}$,

$$\frac{V}{V_a} = \frac{V_T}{V_n/1.674} = \frac{1.674c_v w_T L}{V_n} \tag{IV.47}$$

By substituting Eqs. (IV.46) and (IV.47) into Eq. (IV.40),

$$\left(\frac{M}{M_a}\right)^2 + \left(\frac{V}{V_a}\right)^2 = w_T^2\left[\left(\frac{1.667c_m L^2}{M_n}\right)^2 + \left(\frac{1.674c_v L}{V_n}\right)^2\right] = 1$$

Therefore,

$$(w_T)_{ASD} = \frac{1}{L\sqrt{\left(\frac{1.667c_m L}{M_n}\right)^2 + \left(\frac{1.674c_v}{V_n}\right)^2}} \tag{IV.48}$$

For h/t $> 1.415\sqrt{Ek_v/F_y}$,

$$\frac{V}{V_a} = \frac{V_T}{V_n/1.712} = \frac{1.712c_v w_T L}{V_n} \tag{IV.49}$$

By substituting Eqs. (IV.46) and (IV.49) into Eq. (IV.40),

$$\left(\frac{M}{M_a}\right)^2 + \left(\frac{V}{V_a}\right)^2 = w_T^2\left[\left(\frac{1.667c_m L^2}{M_n}\right)^2 + \left(\frac{1.712c_v L}{V_n}\right)^2\right] = 1$$

Therefore,

$$(w_T)_{ASD} = \frac{1}{L\sqrt{\left(\frac{1.667c_m L}{M_n}\right)^2 + \left(\frac{1.712c_v}{V_n}\right)^2}} \tag{IV.50}$$

The allowable uniform load based on LRFD was calculated as follows:

$$\frac{M_D}{\phi M_n} = \frac{1.2D/L+1.6}{D/L+1}\left[\frac{M_{TL}}{\phi M_n}\right] = \frac{1.2D/L+1.6}{D/L+1}\left[\frac{c_m w_T L^2}{\phi M_n}\right] \quad (IV.51)$$

$$\frac{V_D}{\phi_v V_n} = \frac{1.2D/L+1.6}{D/L+1}\left[\frac{V_T}{\phi_v V_n}\right] = \frac{1.2D/L+1.6}{D/L+1}\left[\frac{c_v w_T L}{\phi_v V_n}\right] \quad (IV.52)$$

By substituting Eqs. (IV.51) and (IV.52) into Eq. (IV.42),

$$\left(\frac{M_D}{\phi M_n}\right)^2 + \left(\frac{V_D}{\phi_v V_n}\right)^2 = w_T^2\left(\frac{1.2D/L+1.6}{D/L+1}\right)^2\left[\left(\frac{c_m L^2}{\phi M_n}\right)^2 + \left(\frac{c_v L}{\phi_v V_n}\right)^2\right] = 1$$

Therefore,

$$(w_T)_{LRFD} = \frac{D/L+1}{1.2D/L+1.6}\,\frac{1}{L\sqrt{\left(\frac{c_m L}{\phi M_n}\right)^2 + \left(\frac{c_v}{\phi_v V_n}\right)^2}} \quad (IV.53)$$

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 = 0.95 and 0.90 for sections with stiffened compression flanges and unstiffened compression flanges, respectively, for nominal section strength and ϕ_v = 0.90, the allowable uniform load ratios are as follows:

For $h/t \le 1.38\sqrt{Ek_v/F_y}$,

$$\frac{(w_T)_{LRFD}}{(w_T)_{ASD}} = \frac{D/L+1}{1.2D/L+1.6}\sqrt{\frac{2.803+0.07716(V_nL/M_n)^2}{1.235+0.02778(V_nL/\phi M_n)^2}} \tag{IV.54}$$

For $h/t > 1.415\sqrt{Ek_v/F_y}$,

$$\frac{(w_T)_{LRFD}}{(w_T)_{ASD}} = \frac{D/L+1}{1.2D/L+1.6}\sqrt{\frac{2.929+0.07716(V_nL/M_n)^2}{1.235+0.02778(V_nL/\phi M_n)^2}} \tag{IV.55}$$

Equations (IV.54) and (IV.55) can be expressed in the following form:

$$\frac{(w_T)_{LRFD}}{(w_T)_{ASD}} = \frac{D/L+1}{1.2D/L+1.6}(K_w) \tag{IV.56}$$

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. (IV.56) as given above. It is not only a function of dead-to-live load ratio but is also a function of h/t, 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 7 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[44]. Different curves represent the relationships for different thicknesses by using the same span length and material. Table IV.1 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. (IV.55) because of the higher h/t ratio.

In Figure 8, 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 IV.2. 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 9 shows the allowable uniform load ratio versus h/t ratio for the 5 in. - deep channels used in Figure 7 and Table IV.1 for a dead-to-live load ratio of 1/5 and a span length of 5 ft. Table IV.3 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 10 shows the relationship of allowable load ratio and dead-to-live load ratio for channels with stiffened flanges. Sectional properties and other related data are included in Table IV.4. Deeper sections with larger h/t ratios give smaller values of the allowable load ratio as indicated in Figure 10.

Table IV.1
Channels With Stiffened Flanges, 5 in. Depths - Case A.

Section	h/t	V_n (Kips)	M_n (K-in.)	ΦM_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.

Channels with unstiffened flanges were also studied. Figure 11 shows the allowable load ratio versus dead-to-live load ratio for 6 in. x 1.5 in. standard channel sections with unstiffened flanges which are listed in Table 2 of Part V of the AISI Design Manual. Different curves represent the relationships for different thicknesses by using the same span length and material. Table IV.5 shows the sectional properties and calculated values used to obtain the curves which indicate that thinner members result in slightly higher values for the allowable load ratio.

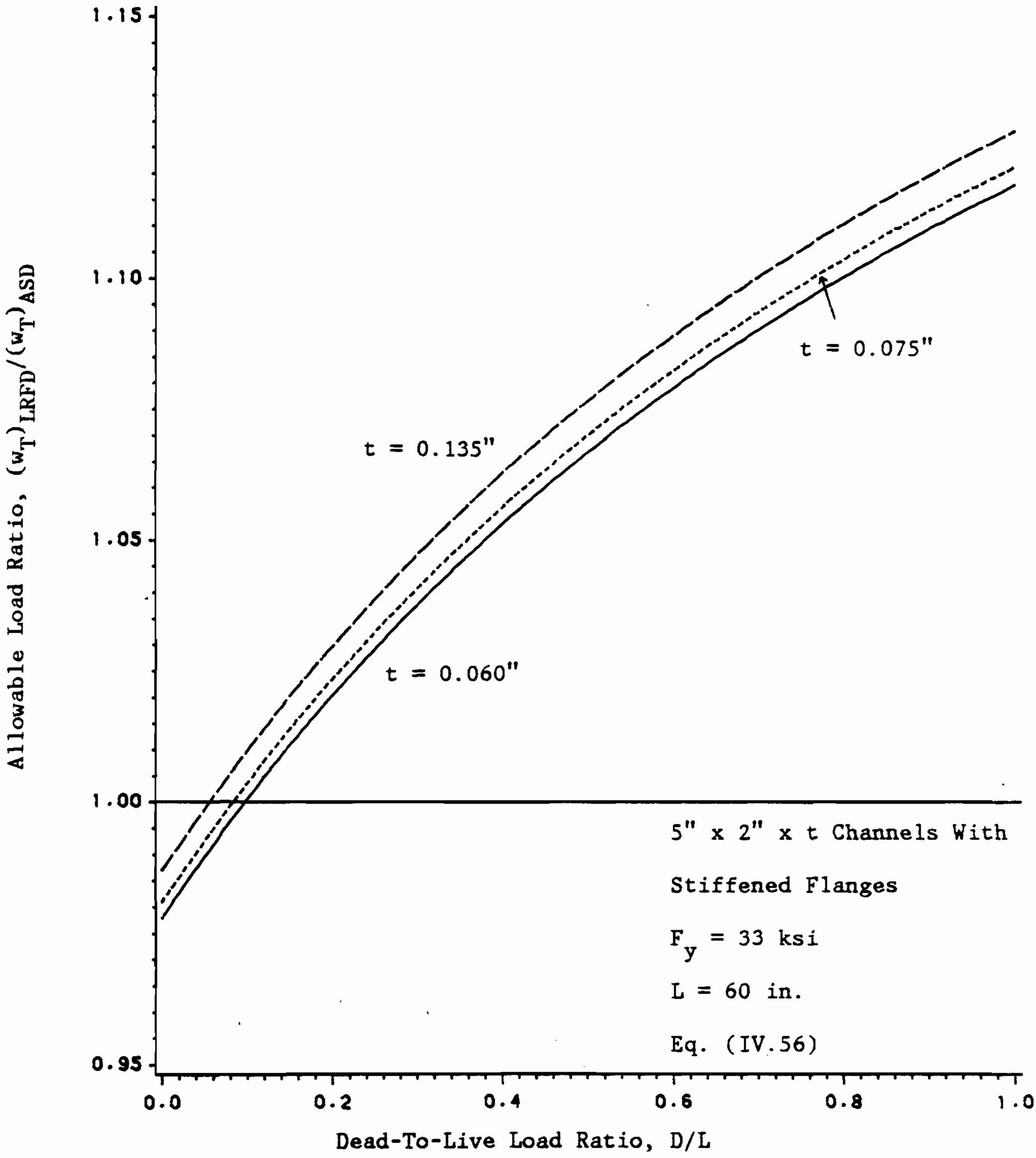

Figure 7. Allowable Load Ratio vs. D/L Ratio for Combined Bending and Shear in Beams - Case A

Table IV.2
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.)	ϕM_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

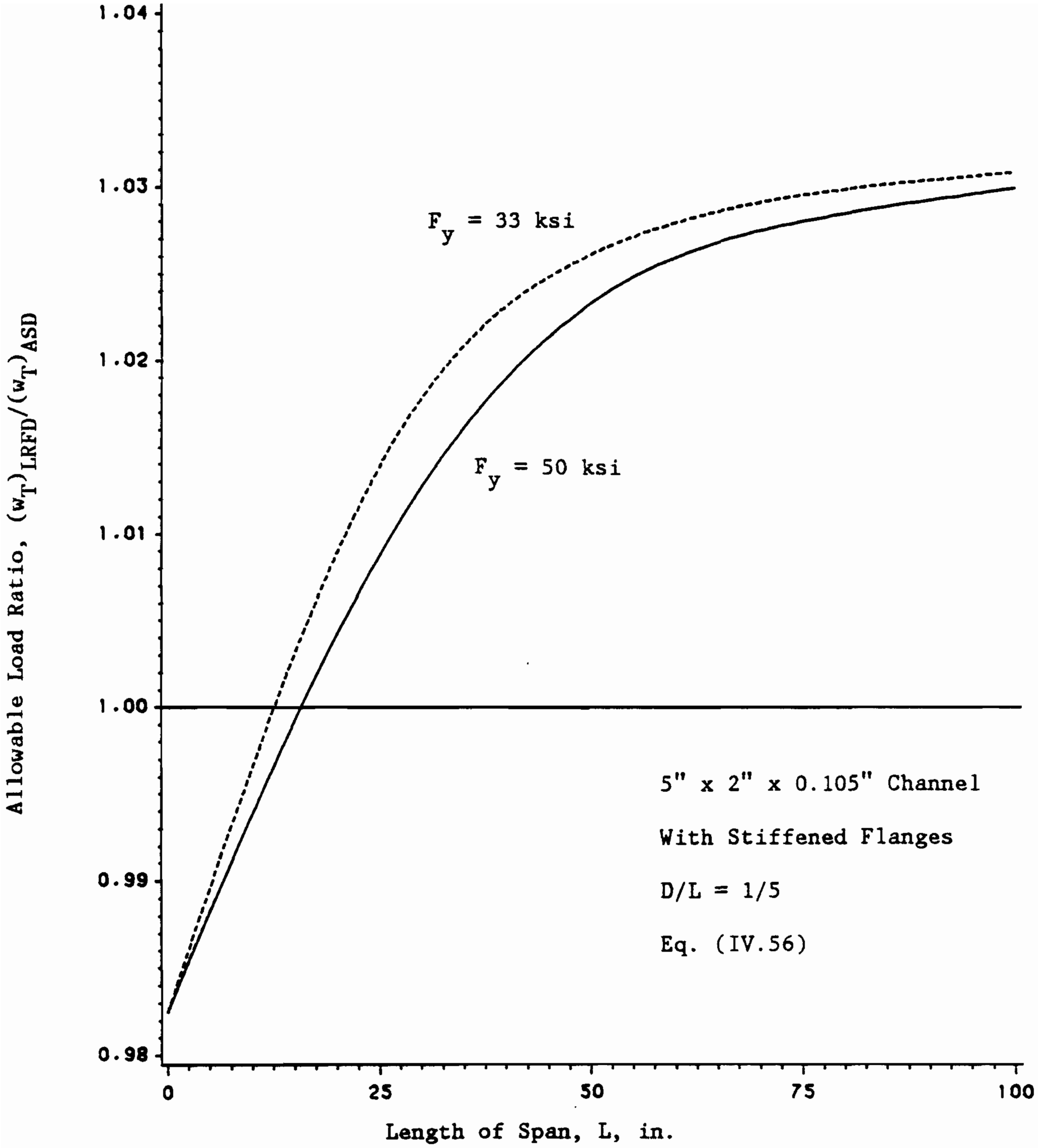

Figure 8. Allowable Load Ratio vs. Span Length - Case A

Table IV.3
5 in. x 2 in. Channels With Stiffened Flanges for F_y = 50 ksi

Section	h/t	V_n (Kips)	M_n (K-in.)	ϕM_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 IV.4
Channels With Stiffened Flanges - Case B.

Section	h/t	V_n (Kips)	M_n (K-in.)	ϕM_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.

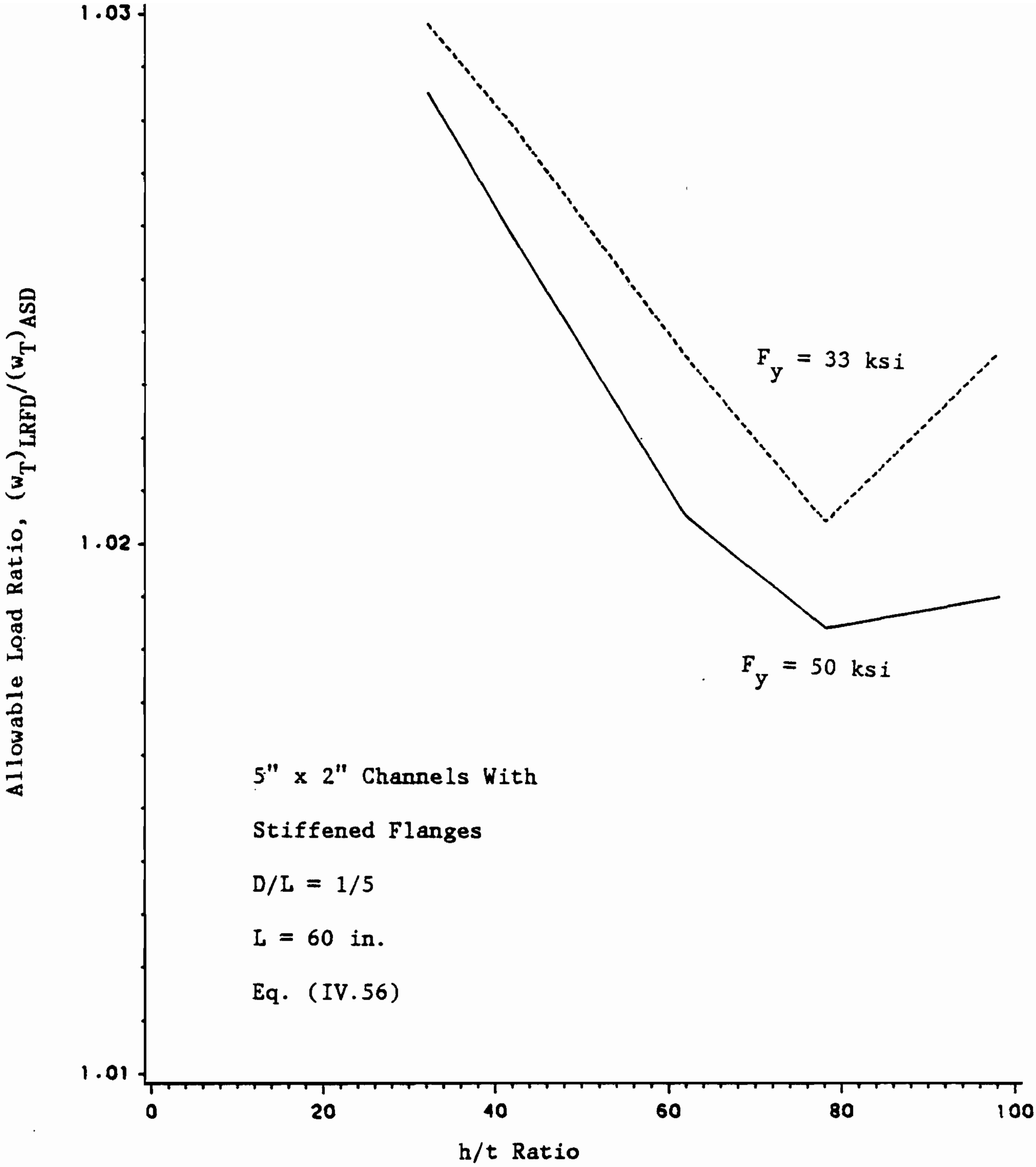

Figure 9. Allowable Load Ratio vs. h/t Ratio for Combined Bending and Shear in Beams - Case A

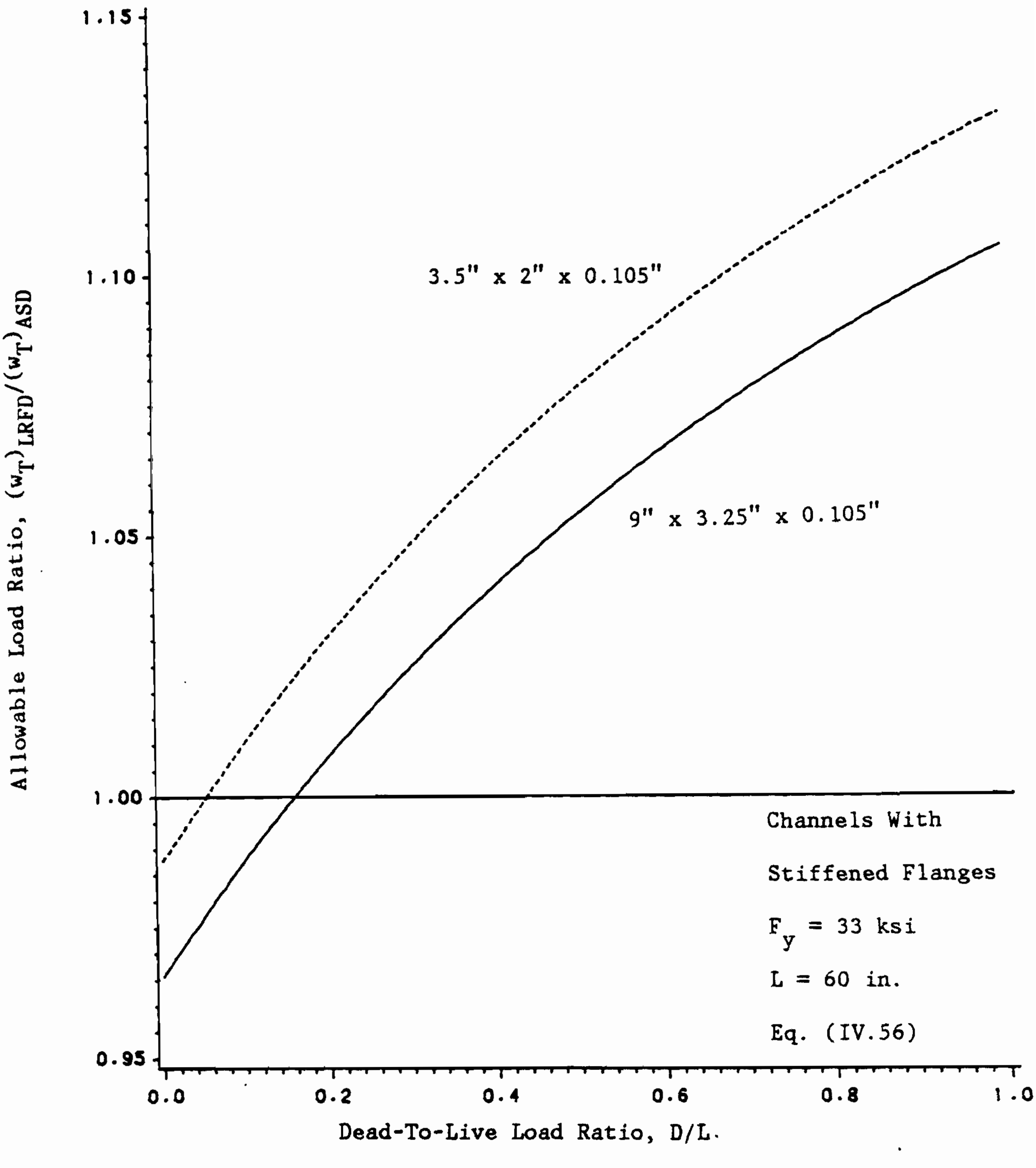

Figure 10. Allowable Load Ratio vs. D/L Ratio for Combined Bending and Shear in Beams - Case B

In Figure 12, the span length was varied for a 6 in. x 1.5 in. x 0.105 in. channel with unstiffened flanges for D/L = 1/5 and F_y = 33 to 50 ksi. It can be seen that the material strength has small effect on the allowable load ratio. This figure also shows that for the channel section used in this comparison, the allowable load permitted by LRFD is about 2% less than that determined by ASD for various span lengths.

Figure 13 shows the allowable load ratio versus h/t ratio for the 6 in.-deep channels used in Figure 11 and Table IV.5 for a dead-to-live load ratio of 1/5 and a span length of 5 ft for F_y = 33 and 50 ksi. This figure shows that higher h/t ratios give slightly larger values of allowable load ratio.

Table IV.5
Channels With Unstiffened Flanges, 6 in. Depths

Section	h/t	V_n (Kips)	M_n (K-in.)	ϕM_n (K-in.)	K_w
6x1.5x0.135	39.67	26.594	58.667	52.800	1.5002
0.105	51.57	16.088	46.637	41.973	1.5004
0.075	75.50	8.208	31.788	28.609	1.5008
0.060	94.88	5.253	24.129	21.716	1.5011
0.048	119.09	2.758	18.314	16.483	1.5123

* F_y = 33 ksi, L = 60 in.

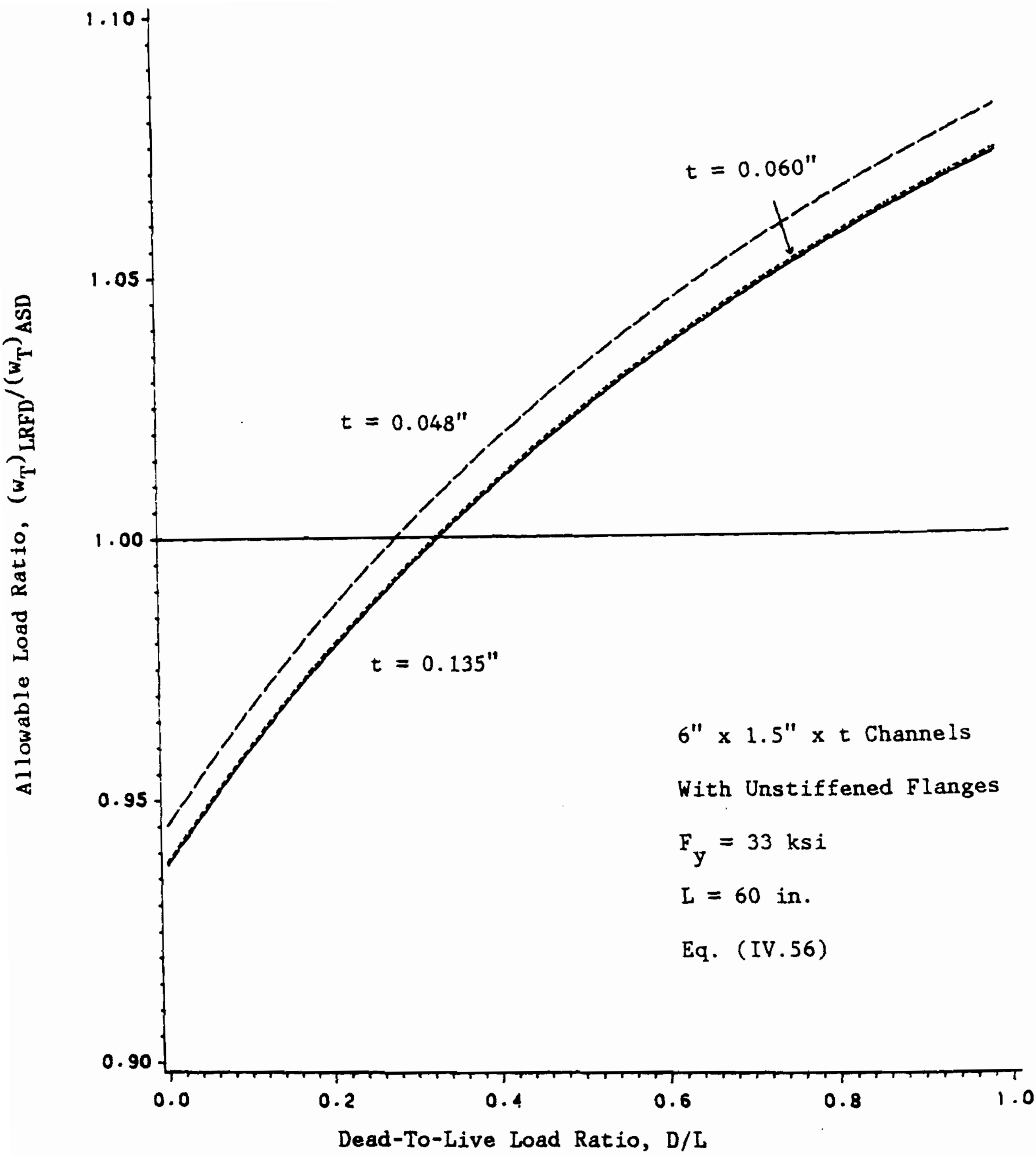

Figure 11. Allowable Load Ratio vs. D/L Ratio for Combined Bending and Shear in Beams - Case C

Table IV.6
6 in. x 1.5 in. x 0.105 in. Channels With Unstiffened Flanges
for Various Lengths and Yield Points

F_y (ksi)	L (in.)	V_n (Kips)	M_n (K-in.)	ΦM_n (K-in.)	K_w
33	0	16.088	46.637	41.973	1.5065
	25	16.088	46.637	41.973	1.5021
	50	16.088	46.637	41.973	1.5006
	75	16.088	46.637	41.973	1.5003
	100	16.088	46.637	41.973	1.5001
50	0	19.803	68.840	61.956	1.5065
	25	19.803	68.840	61.956	1.5026
	50	19.803	68.840	61.956	1.5009
	75	19.803	68.840	61.956	1.5004
	100	19.803	68.840	61.956	1.5002

Table IV.7
6 in. x 1.5 in. Channels With Unstiffened Flanges, for F_y = 50 ksi

Section	h/t	V_n (Kips)	M_n (K-in.)	ΦM_n (K-in.)	K_w
6x1.5x0.135	39.67	32.735	88.890	80.001	1.5004
0.105	51.57	19.803	68.840	61.956	1.5006
0.075	75.50	10.103	45.584	41.026	1.5011
0.060	94.88	5.410	34.647	31.183	1.5117
0.048	119.09	2.758	26.283	23.655	1.5191

* L = 60 in.

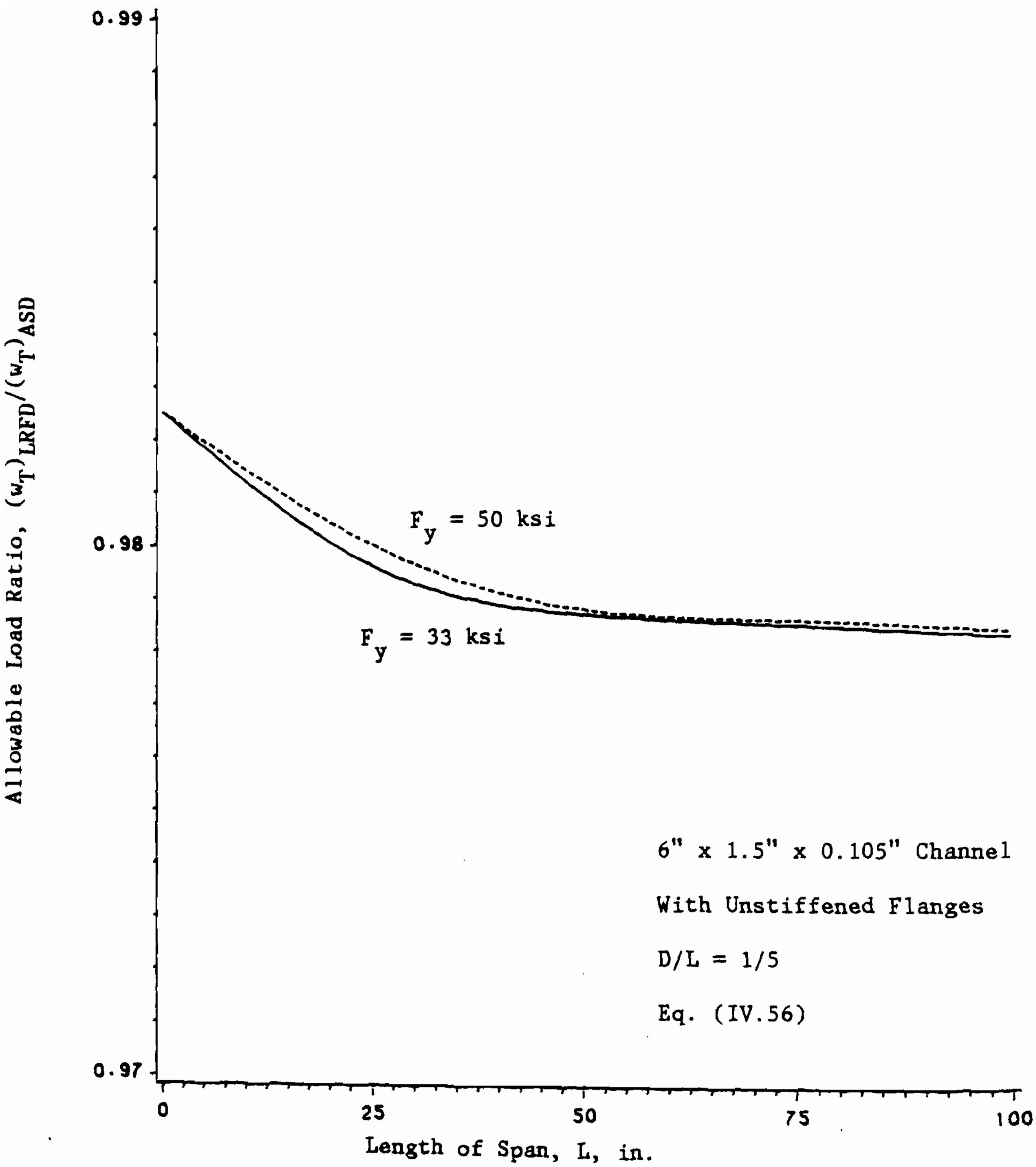

Figure 12. Allowable Load Ratio vs. Span Length - Case C

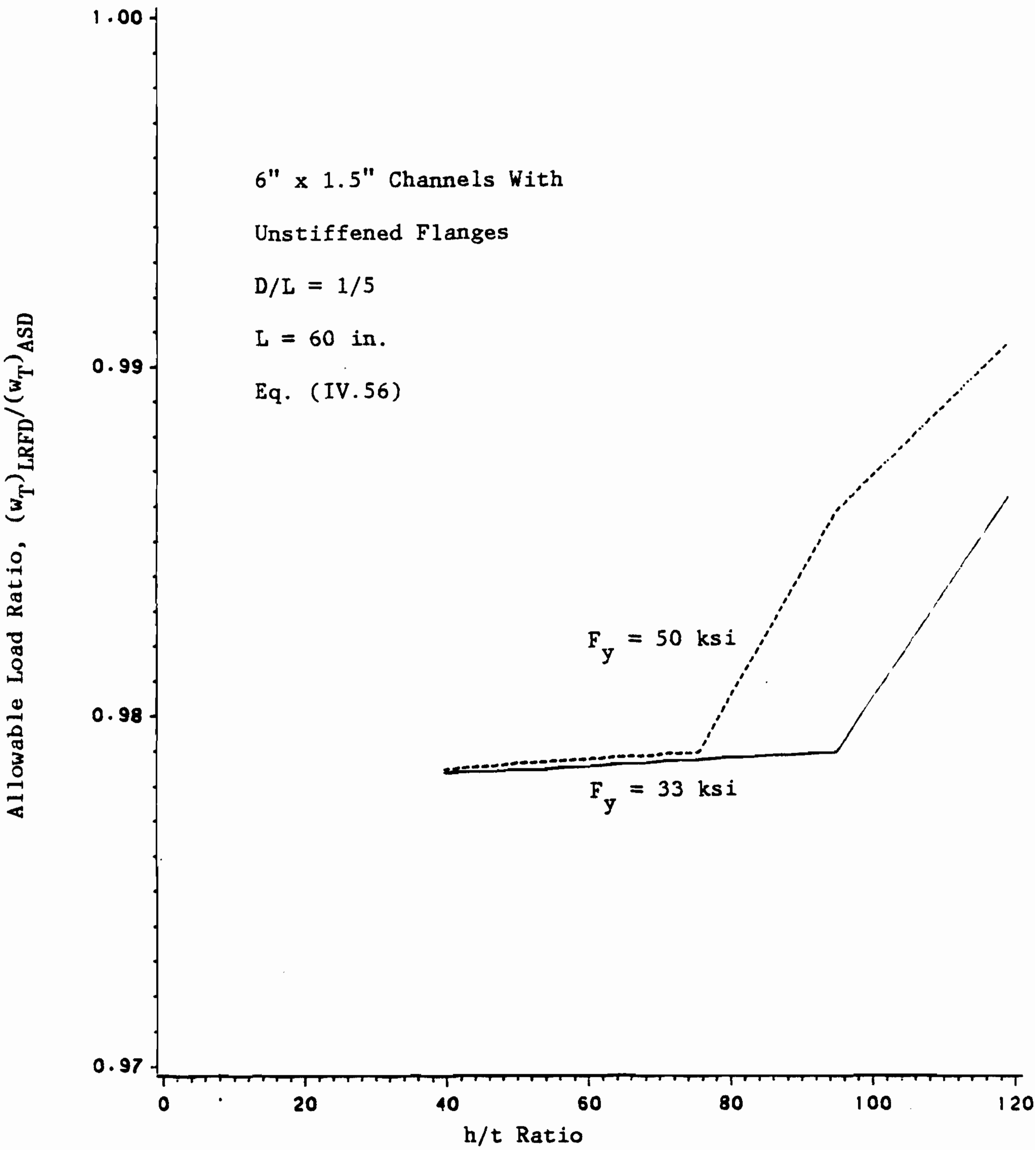

Figure 13. Allowable Load Ratio vs. h/t Ratio for Combined Bending and Shear in Beams - Case C

For hat sections, one web was assumed to carry one-half of the load and, therefore, only half-sectional properties were used. Dimensions and sectional properties of numerous hat sections are given in Table 9 of Part V of the AISI Design Manual and Table IV.8 lists sectional properties and calculated member strengths used in this comparison. Figure 14 shows the relationships between allowable uniform load ratio and dead-to-live load ratio for three hat sections with a yield point of 33 ksi and a span length of 5 ft. All 4 in. deep hat sections resulted in the same curve regardless of h/t ratio. Hat sections with larger depths or larger h/t ratios resulted in smaller values of allowable load ratio.

I-sections made of two channels back-to-back would result in the same comparison and conclusions as the single channel sections.

From Figures 7 through 14, 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 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 conservative than LRFD. Yield point of steel has little effect on the allowable load ratio. The lower the yield point, the larger the difference. Span length has little effect on the allowable uniform load ratio as shown in Figs. 8 and 12. 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. For hat sections, smaller depths result in a larger difference between the allowable loads.

Table IV.8
Hat Sections (Positive Bending)

Section	h/t	V_n (Kips)	M_n (K-in.)	ϕM_n (K-in.)	K_w
4x2x0.075	48.83	8.208	15.086	14.331	1.5805
4x4x0.105	32.52	16.088	29.374	27.906	1.5806
4x4x0.075	48.83	8.208	17.237	16.375	1.5798
4x6x0.135	24.85	26.594	43.978	41.779	1.5810
4x6x0.105	32.52	16.088	30.756	29.218	1.5803
6x9x0.105	51.57	16.088	55.454	52.681	1.5745
10x5x0.075	128.83	6.225	72.671	69.037	1.5573

* F_y = 33 ksi, L = 60 in.

E. <u>WEB CRIPPLING STRENGTH</u>

Beam webs should be checked for web crippling at locations of high intensity loads. This would occur under concentrated loads or support reactions.

1. <u>Allowable Stress Design.</u> To avoid crippling of unreinforced flat webs of flexural members having a flat width ratio, h/t, equal to or less than 200, concentrated loads and reactions shall not exceed the value of P_a given in Table IV.9 according to Section C3.4 of the AISI Specification. 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.

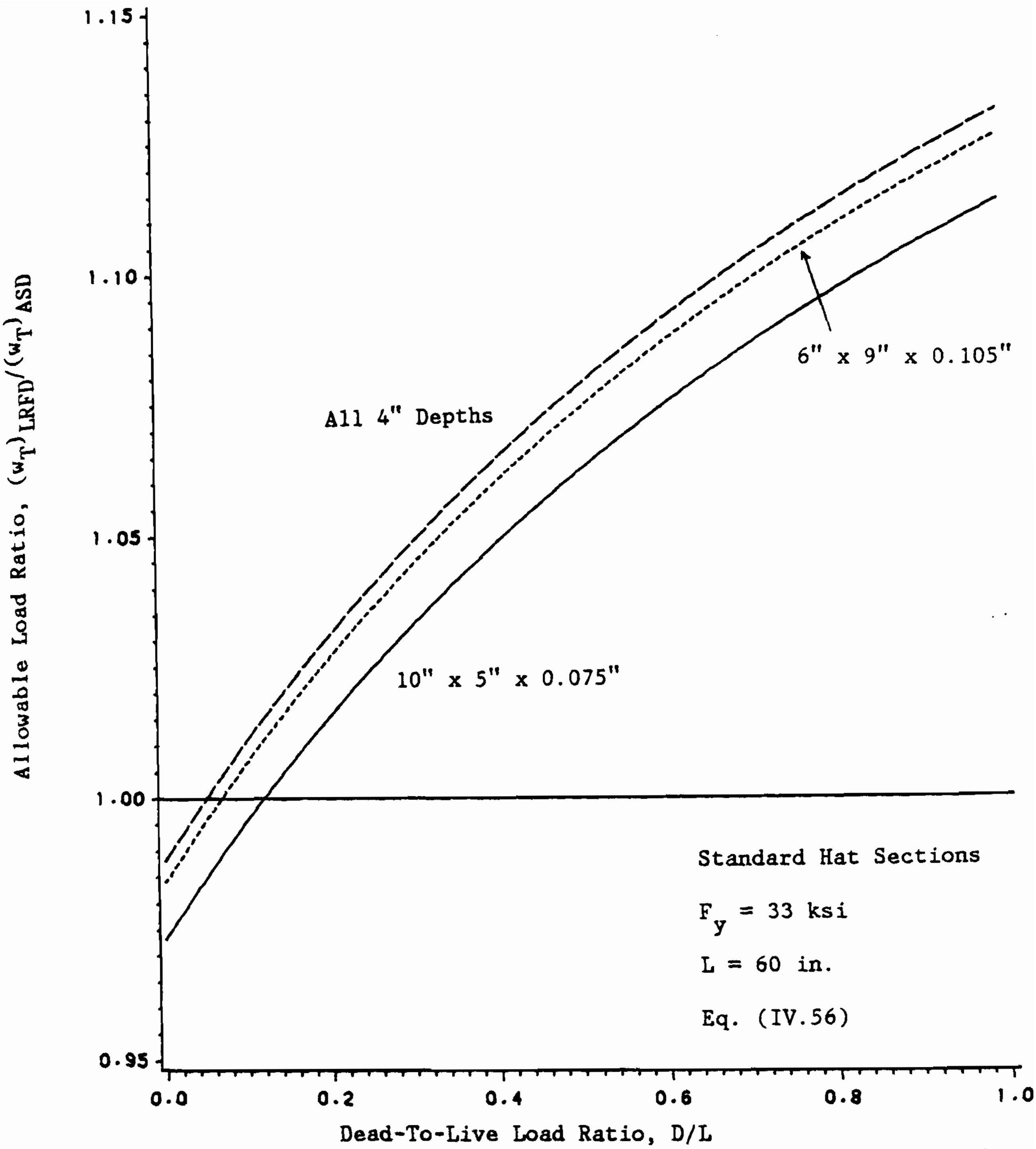

Figure 14. Allowable Load Ratio vs. D/L Ratio for Combined Bending and Shear in Beams - Case D

The formulas in Table IV.9 apply to beams when $R/t \leq 6$ and to deck when $R/t \leq 7$, $N/t \leq 210$ and $N/h \leq 3.5$.

Table IV.9
Allowable Load for Web Crippling, P_a

		Shapes Having Single Webs		Shapes Having Multiple Webs (1)
		Stiffened Flanges	Unstiffened Flanges	Stiffened and Unstiffened Flanges
Opposing Loads Spaced >1.5h (2)	End Reaction (3)	Eq. IV.57	Eq. IV.58	Eq. IV.59
	Interior Reaction (4)	Eq. IV.60	Eq. IV.60	Eq. IV.61
Opposing Loads Spaced ≤1.5h (5)	End Reaction (3)	Eq. IV.62	Eq. IV.62	Eq. IV.63
	Interior Reaction (4)	Eq. IV.64	Eq. IV.64	Eq. IV.65

Footnotes and Equation References to Table IV.9:

(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.

Equations for Table IV.9:

$$t^2kC_3C_4C_\theta[179-0.33(h/t)][1+0.01(N/t)] \quad (IV.57)$$

$$t^2kC_3C_4C_\theta[117-0.15(h/t)][1+0.01(N/t)] \quad (IV.58)$$

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

$$t^2F_yC_6(5.0+0.63\sqrt{N/t}) \quad (IV.59)$$

$$t^2kC_1C_2C_\theta[291-0.40(h/t)][1+0.007(N/t)] \quad (IV.60)$$

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

$$t^2F_yC_5(0.88+0.12m)(7.50+1.63\sqrt{N/t}) \quad (IV.61)$$

$$t^2kC_3C_4C_\theta[132-0.31(h/t)][1+0.01(N/t)] \quad (IV.62)$$

$$t^2F_yC_8(0.64+0.31m)(5.0+0.63\sqrt{N/t}) \quad (IV.63)$$

$$t^2kC_1C_2C_\theta[417-1.22(h/t)][1+0.0013(N/t)] \quad (IV.64)$$

$$t^2F_yC_7(0.82+0.15m)(7.50+1.63\sqrt{N/t}) \quad (IV.65)$$

In the above referenced formulas,

P_a = Allowable concentrated load or reaction per web, kips

$C_1 = (1.22-0.22k)$ (IV.66)

$C_2 = (1.06-0.06R/t) \leq 1.0$ (IV.67)

$C_3 = (1.33-0.33k)$ (IV.68)

$C_4 = (1.15-0.15R/t) \leq 1.0$ but not less than 0.50 (IV.69)

$C_5 = (1.49-0.53k) \geq 0.6$ (IV.70)

$C_6 = 1+(h/t)/750$, when $h/t \leq 150$ (IV.71)

$= 1.20$, when $h/t>150$ (IV.72)

$C_7 = 1/k$, when $h/t \leq 66.5$ (IV.73)

$= [1.10-(h/t)/665]/k$, when $h/t>66.5$ (IV.74)

$C_8 = [0.98-(h/t)/865]/k$ (IV.75)

$C_\theta = 0.7+0.3(\theta/90)^2$ (IV.76)

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

$k = F_y/33$ (IV.77)

$m = t/0.075$ (IV.78)

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 $\geq 45^\circ$, but not more than 90°

2. LRFD Criteria. Section C3.4 of the LRFD Specification specifies that to avoid crippling of unreinforced flat webs of flexural

members having a flat width ratio, h/t, equal to or less than 200, concentrated loads and reactions determined according to the factored design loads shall not exceed the values of $\phi_w P_n$, with ϕ_w = 0.75 and 0.80 for single unreinforced webs and I-sections, respectively, and P_n given in Table C3.4-1 of the LRFD Specification which is obtained from the above listed equations by using a factor of safety of 1.85 for single unreinforced webs and 2.0 for multiple webs. For 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.

3. <u>Comparison.</u> The unfactored concentrated load or reaction can be calculated for both methods by using Eq. (IV.79):

$$P_T = P_{DL} + P_{LL} \tag{IV.79}$$

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. (II.6) and is as follows:

$$(P_a)_{LRFD} = \phi_w P_n(D/L+1)/(1.2D/L+1.6) \tag{IV.80}$$

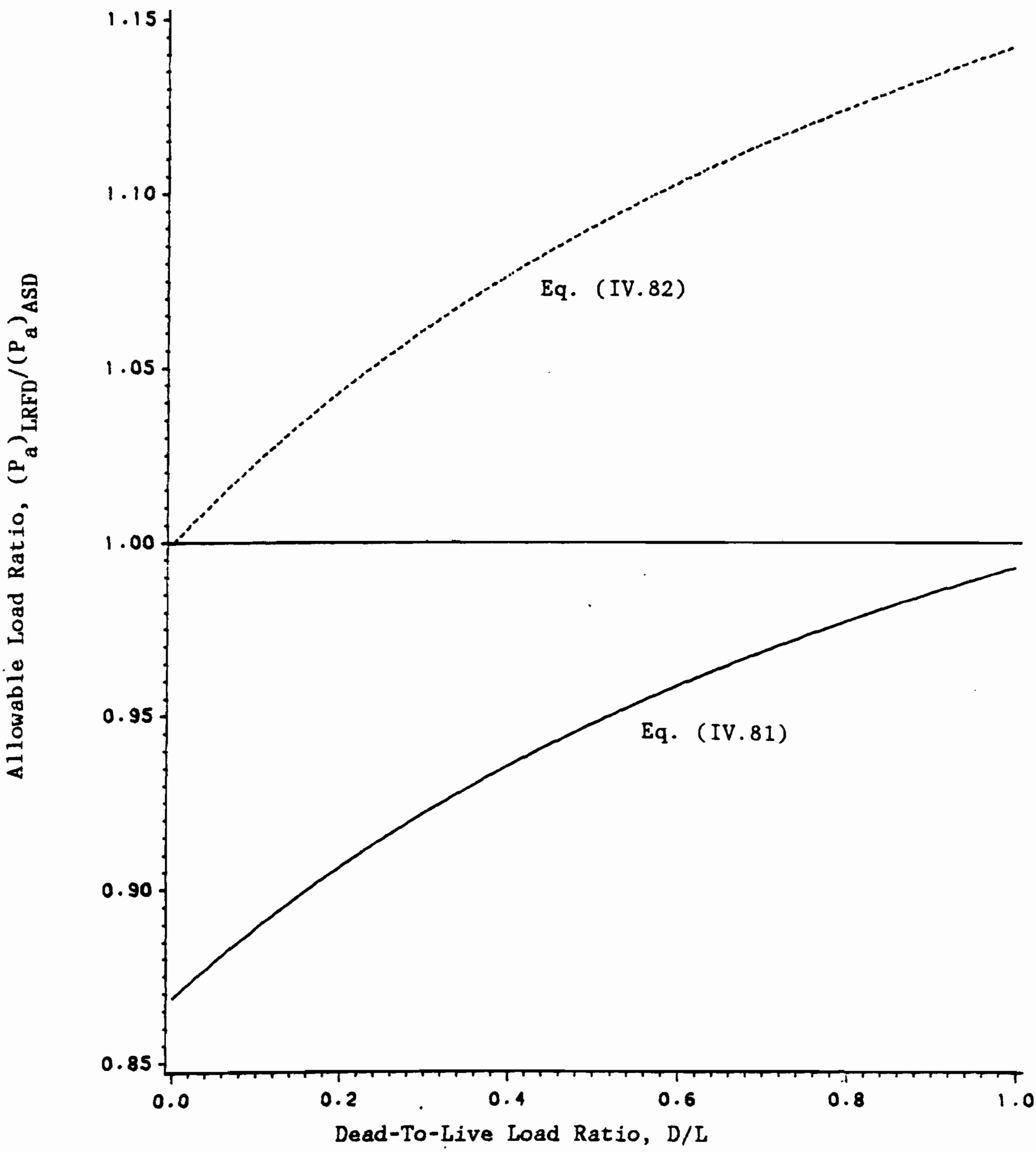

Figure 15. Allowable Load Ratio vs. D/L Ratio for Web Crippling

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 $\phi_w = 0.75$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.85\phi_w \frac{D/L+1}{1.2D/L+1.6} = 1.39 \frac{D/L+1}{1.2D/L+1.6} \qquad \text{(IV.81)}$$

For I-sections or similar shapes and $\phi_w = 0.80$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.00\phi_w \frac{D/L+1}{1.2D/L+1.6} = 1.60 \frac{D/L+1}{1.2D/L+1.6} \qquad \text{(IV.82)}$$

Figure 15 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 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.

F. <u>COMBINED BENDING AND WEB CRIPPLING STRENGTH.</u> The interaction between bending and web crippling is similar to that of combined bending and shear and exists when a large bending moment is applied close to concentrated loads or support reactions. The web crippling capacity may be reduced according to the following interaction equations provided in the specifications.

1. <u>Allowable Stress Design.</u> According to Section C3.5 of the AISI Specification, 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:

$$1.2(P/P_a)+(M/M_a) \leq 1.5 \tag{IV.83}$$

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);

$$1.1(P/P_a)+(M/M_a) \leq 1.5 \tag{IV.84}$$

Exception: When $h/t \leq 2.33/\sqrt{(F_y/E)}$ and $\lambda \leq 0.673$, the allowable concentrated load or reaction may be determined by Section C3.4 of the AISI Specification.

In the above formulas,

P = Concentrated load or reaction in the presence of bending moment

P_a = Allowable concentrated load or reaction in the absence of bending moment determined in accordance with Section C3.4 of the AISI Specification

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

M_a = Allowable bending moment determined according to Section C3.1.1 if bending alone exists excluding lateral buckling

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

t = Thickness of the web or flange

λ = Slenderness factor

2. LRFD Criteria. Section C3.5 of the LRFD Specification specifies that 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:

$$1.07(P_D/\Phi_w P_n)+(M_D/\Phi M_n) \leq 1.42 \qquad \text{(IV.85)}$$

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);

$$0.82(P_D/\phi_w P_n)+(M_D/\phi M_n) \leq 1.32 \qquad \text{(IV.86)}$$

Exception: When $h/t \leq 2.33/\sqrt{(F_y/E)}$ and $\lambda \leq 0.673$, the nominal ultimate concentrated load or reaction may be determined by Section C3.4 of the LRFD Specification.

In the above formulas,

ϕ = Resistance factor for bending

ϕ_w = Resistance factor for web crippling

P_D = Concentrated load or reaction computed on the basis of factored loads in the presence of bending moment.

P_n = Nominal ultimate concentrated load or reaction in the absence of bending moment determined in accordance with Section C3.4 of the LRFD Specification.

M_D = Applied bending moment at, or immediately adjacent to, the point of application of the concentrated load or reaction P_D, computed on the basis of factored loads

M_n = Nominal ultimate bending moment determined according to Section C3.1.1 of the LRFD Specification if bending alone exists

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

t = Thickness of the web or flange

λ = Slenderness factor

3. Comparison. 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 loads, P_T, were calculated for both design methods. Since each design procedure utilizes separate design variable, the allowable loads were determined using nominal resistances.

The allowable load based on allowable stress design was calculated as follows:

$$\frac{M}{M_a} = \frac{M_{TL}}{0.6M_n} = \frac{P_T L/4}{0.6M_n} = \frac{0.4167P_T L}{M_n} \qquad \text{(IV.87)}$$

For beams with single webs,

$$\frac{P}{P_a} = \frac{P_T}{P_n/1.85} = \frac{1.85P_T}{P_n} \qquad \text{(IV.88)}$$

By substituting Eqs. (IV.87) and (IV.88) into Eq. (IV.83),

$$1.2\frac{P}{P_a} + \frac{M}{M_a} = \frac{2.22P_T}{P_n} + \frac{0.4167P_T L}{M_n} = 1.5$$

Therefore,

$$(P_T)_{ASD} = \frac{3.6P_n}{5.328 + (P_n L/M_n)} \qquad \text{(IV.89)}$$

For I-sections,

$$\frac{P}{P_a} = \frac{P_T}{P_n/2.00} = \frac{2.00P_T}{P_n} \tag{IV.90}$$

By substituting Eqs. (IV.87) and (IV.90) into Eq. (IV.84),

$$1.1\frac{P}{P_a} + \frac{M}{M_a} = \frac{2.20P_T}{P_n} + \frac{0.4167P_TL}{M_n} = 1.5$$

Therefore,

$$(P_T)_{ASD} = \frac{3.6P_n}{5.280 + (P_nL/M_n)} \tag{IV.91}$$

The allowable load based on LRFD criteria was calculated as follows:

$$\frac{M_D}{\phi M_n} = \frac{1.2D/L+1.6}{D/L+1}\left[\frac{M_{TL}}{\phi M_n}\right] = \frac{1.2D/L+1.6}{D/L+1}\left[\frac{P_TL/4}{\phi M_n}\right] \tag{IV.92}$$

$$\frac{P_D}{\phi_w P_n} = \frac{1.2D/L+1.6}{D/L+1}\left[\frac{P_T}{\phi_w P_n}\right] \tag{IV.93}$$

For beams with single webs, Eqs. (IV.92) and (IV.93) were substituted into Eq. (IV.85) to obtain the following expression:

$$1.07\frac{P_D}{\phi_w P_n} + \frac{M_D}{\phi M_n} = \frac{1.2D/L+1.6}{D/L+1}(P_T)\left[\frac{1.07}{\phi_w P_n} + \frac{0.25L}{\phi M_n}\right] = 1.42$$

Therefore,

$$(P_T)_{LRFD} = \frac{D/L+1}{1.2D/L+1.6}\left[\frac{5.680\phi_w P_n}{4.280+(\phi_w P_n L/\phi M_n)}\right] \tag{IV.94}$$

For I-sections, Eqs. (IV.92) and (IV.93) were substituted into Eq. (IV.86) to obtain the following expression:

$$0.82\,\frac{P_D}{\phi_w P_n} + \frac{M_D}{\phi M_n} = \frac{1.2D/L+1.6}{D/L+1}\,(P_T)\left[\frac{0.82}{\phi_w P_n} + \frac{0.25L}{\phi M_n}\right] = 1.32$$

Therefore,

$$(P_T)_{LRFD} = \frac{D/L+1}{1.2D/L+1.6}\left[\frac{5.280\phi_w P_n}{3.280+(\phi_w P_n L/\phi M_n)}\right] \tag{IV.95}$$

The allowable load ratios based on the design examples for combined bending and web crippling are given in Eqs. (IV.96) and (IV.97) for ϕ = 0.95 and 0.90 for nominal section strength of sections with stiffened compression flanges and unstiffened compression flanges, respectively.

For beams with single webs (ϕ_w = 0.75),

$$\frac{(P_T)_{LRFD}}{(P_T)_{ASD}} = \frac{D/L+1}{1.2D/L+1.6}\left[\frac{6.305+1.183(P_n L/M_n)}{4.280+(0.75/\phi)(P_n L/M_n)}\right] \tag{IV.96}$$

For I-sections ($\phi_w = 0.80$),

$$\frac{(P_T)_{LRFD}}{(P_T)_{ASD}} = \frac{D/L+1}{1.2D/L+1.6}\left[\frac{6.195+1.173(P_nL/M_n)}{3.280+(0.80/\phi)(P_nL/M_n)}\right] \tag{IV.97}$$

Eqs. (IV.96) and (IV.97) can be expressed in the following form:

$$\frac{(P_T)_{LRFD}}{(P_T)_{ASD}} = \frac{D/L+1}{1.2D/L+1.6}(K_w) \tag{IV.98}$$

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, sectional geometry, and material strength. Several individual beam sections with different conditions were studied due to the complexity involved in the comparison.

Figures 16 and 18 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 IV.10 and IV.12 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 17 shows how the span length and yield point of steel affect the allowable load ratio for channels with stiffened flanges. Table IV.11 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 17, it can be seen that yield point of steel has a negligible effect on the allowable load ratio.

Figure 19 shows the relationship between allowable load ratio and dead-to-live load ratio for 8 in. x 2 in. x 0.105 in. channel with unstiffened flanges with L = 5 ft and F_y = 33 ksi. Table IV.13 presents section properties and calculated member strengths for this channel section. For D/L = 0.5, the allowable web crippling load determined by LRFD is 1.4% lower than that permitted by allowable stress design.

Figure 20 shows how the span length and yield point of steel affect the allowable load ratio for channels with unstiffened flanges. Table IV.14 presents calculated member strengths for different span lengths and yield points. As shown in this figure, larger span lengths will result in slightly lower values of the allowable load ratio. Also from Figure 17, it can be seen that yield point of steel has a negligible effect on the allowable load ratio.

For I-sections made from two channels back-to-back, Figure 21 shows the relationship between allowable load ratio and dead-to-live load ratio. Table IV.15 presents sectional properties and calculated values for the cold-formed I-section with F_y = 33 ksi and L = 5 ft. For the I-section with stiffened flanges shown in Figure 21, LRFD would result in an allowable load about 7.1% higher than the load computed from allowable stress design for D/L = 0.5.

Figure 22 shows how the span length and yield point of steel affect the allowable load ratio. Table IV.16 presents calculated member strengths for different span lengths and yield points. A higher yield point of steel results in a larger value of the allowable load ratio. As shown in Figure 22, span length has a greater effect on the allowable load ratio for I-sections than it does on channel sections which are shown in Figures 17 and 20. In general, large span lengths result in lower values of the allowable load ratio.

Table IV.10
Channels With Stiffened Flanges

Section	h/t	P_n (Kips)	M_n (K-in.)	ΦM_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.

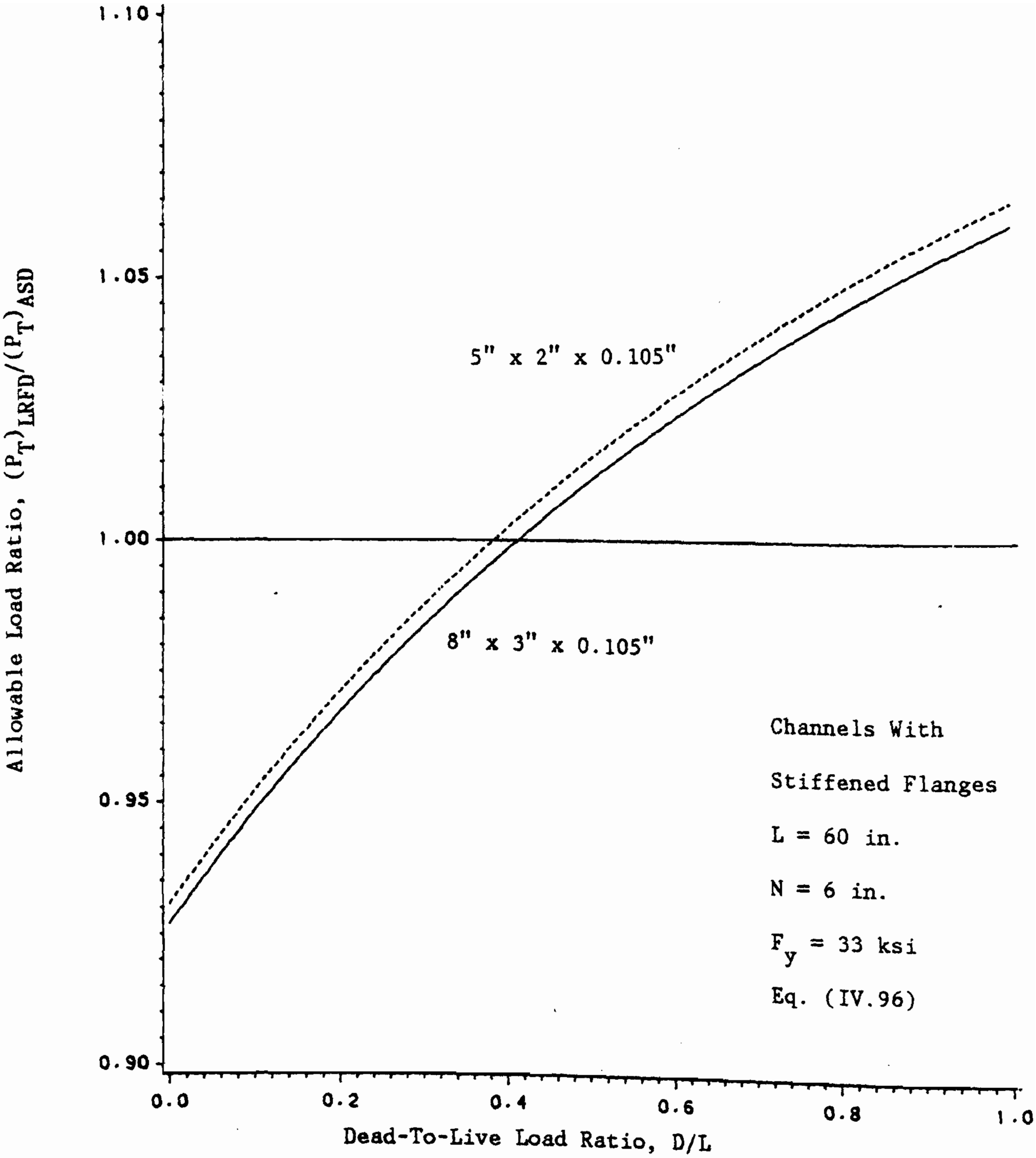

Figure 16. Allowable Load Ratio vs. D/L Ratio for Combined Bending and Web Crippling - Case 1

Table IV.11
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.)	ϕM_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

Table IV.12
Channels With Stiffened Flanges, 5 in. Depths

Section	h/t	P_n (Kips)	M_n (K-in.)	ϕM_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.

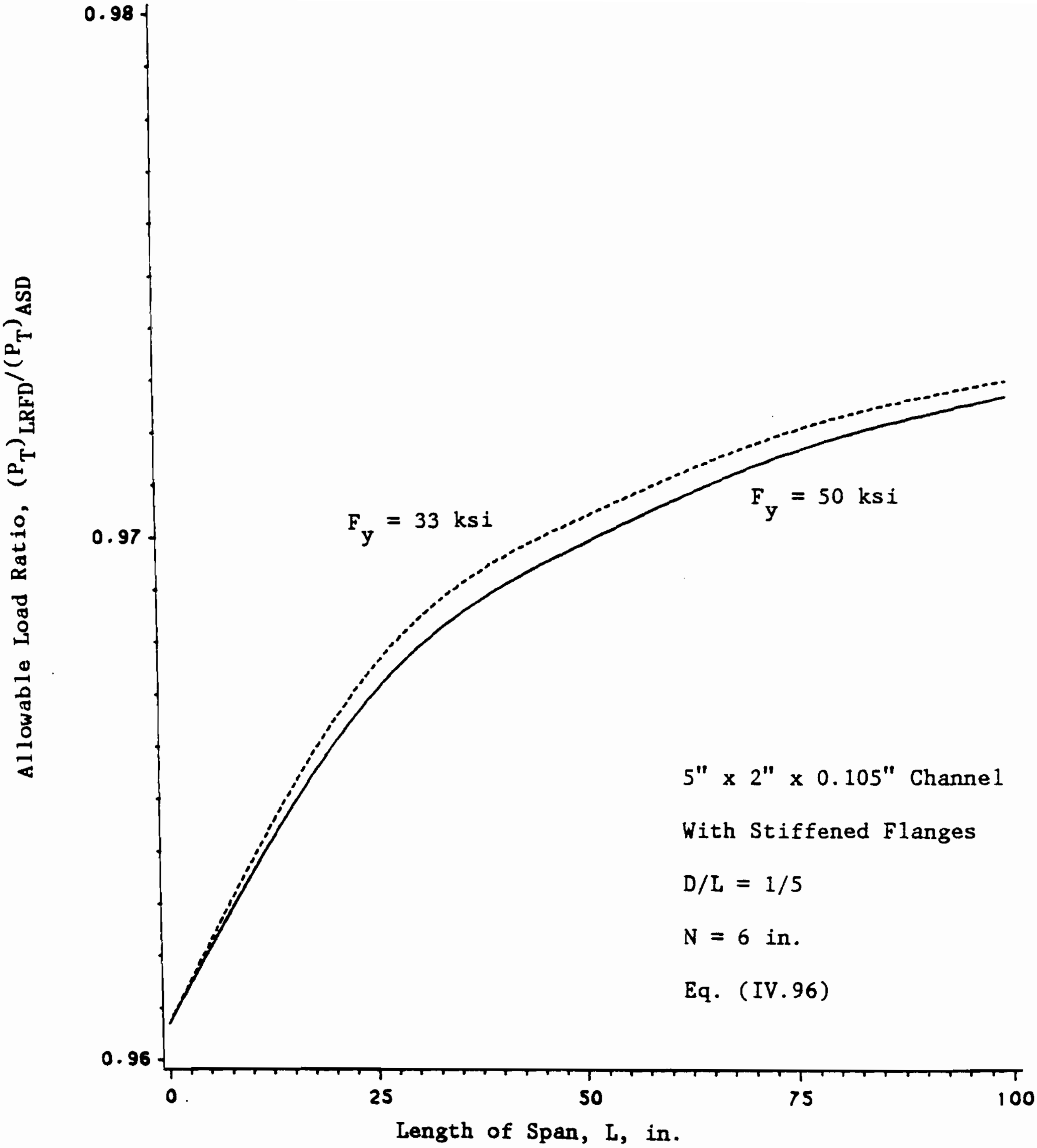

Figure 17. Allowable Load Ratio vs. Span Length for Combined Bending and Web Crippling - Case 2

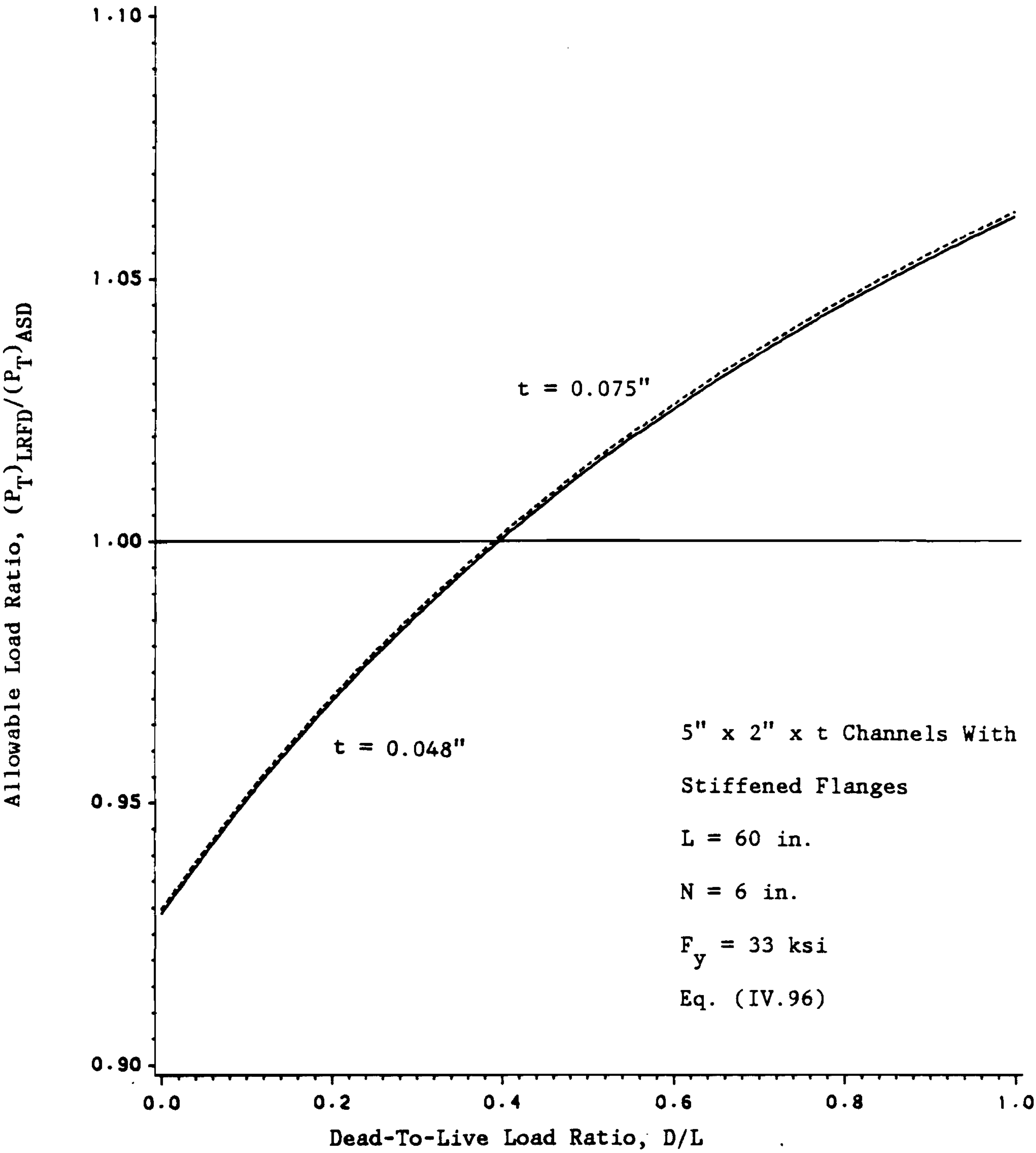

Figure 18. Allowable Load Ratio vs. D/L Ratio for Combined Bending and Web Crippling - Case 2

Table IV.13
Channel With Unstiffened Flanges

Section	h/t	P_n (Kips)	M_n (K-in.)	ϕM_n (K-in.)	K_w
8x2x0.105	70.62	7.144	80.189	72.170	1.4458

* F_y = 33 ksi, L = 60 in., N = 6 in.

Table IV.14
8 in. x 2 in. x 0.105 in. Channels With Unstiffened Flanges
for Various Lengths and Yield Points

F_y (ksi)	L (in.)	P_n (Kips)	M_n (K-in.)	ϕM_n (K-in.)	K_w
33	0	7.144	80.189	72.170	1.4731
	25	7.144	80.189	72.170	1.4569
	50	7.144	80.189	72.170	1.4483
	75	7.144	80.189	72.170	1.4429
	100	7.144	80.189	72.170	1.4392
50	0	9.597	115.222	103.700	1.4731
	25	9.597	115.222	103.700	1.4577
	50	9.597	115.222	103.700	1.4492
	75	9.597	115.222	103.700	1.4438
	100	9.597	115.222	103.700	1.4400

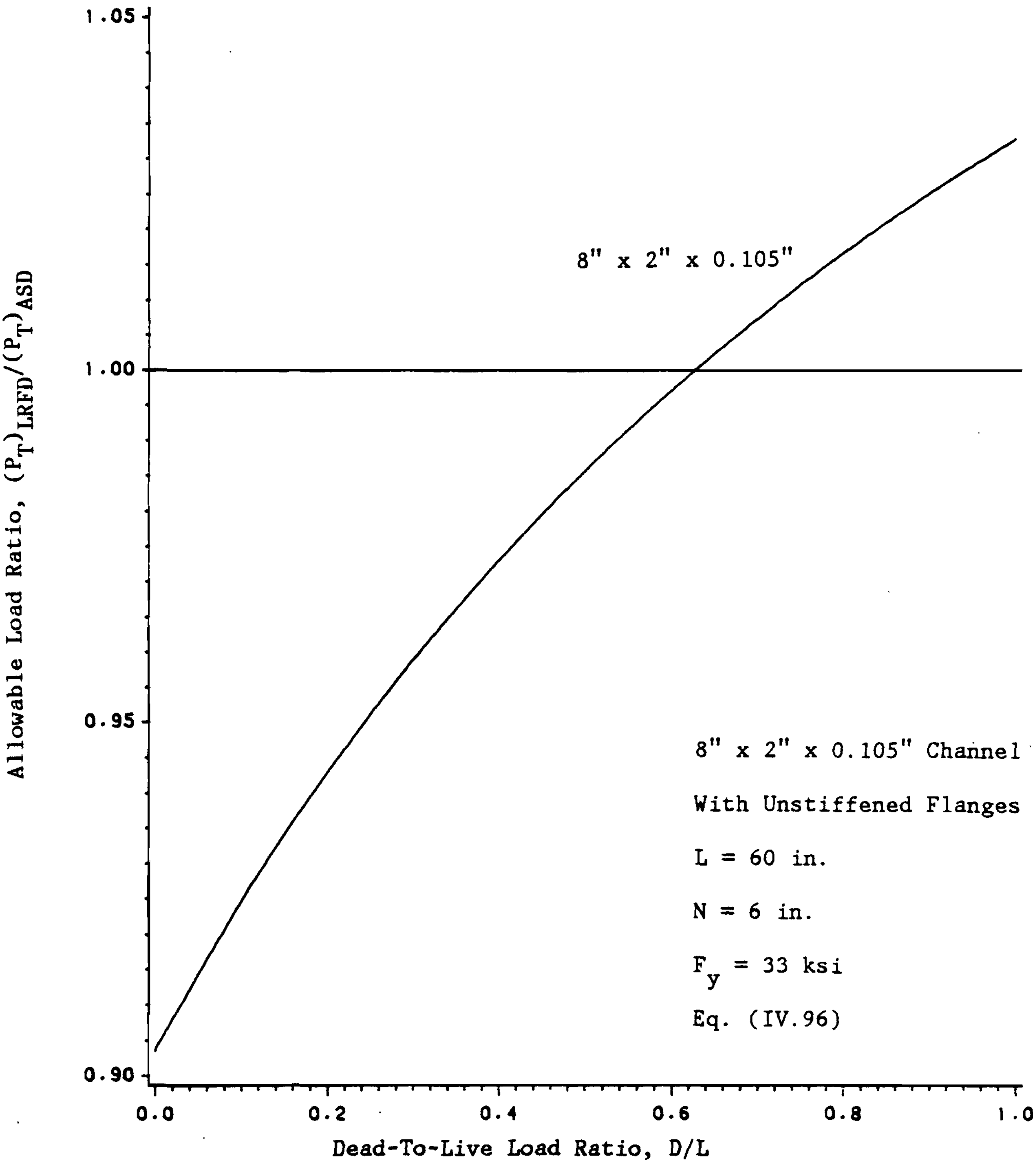

Figure 19. Allowable Load Ratio vs. D/L Ratio for Combined Bending and Web Crippling - Case 3

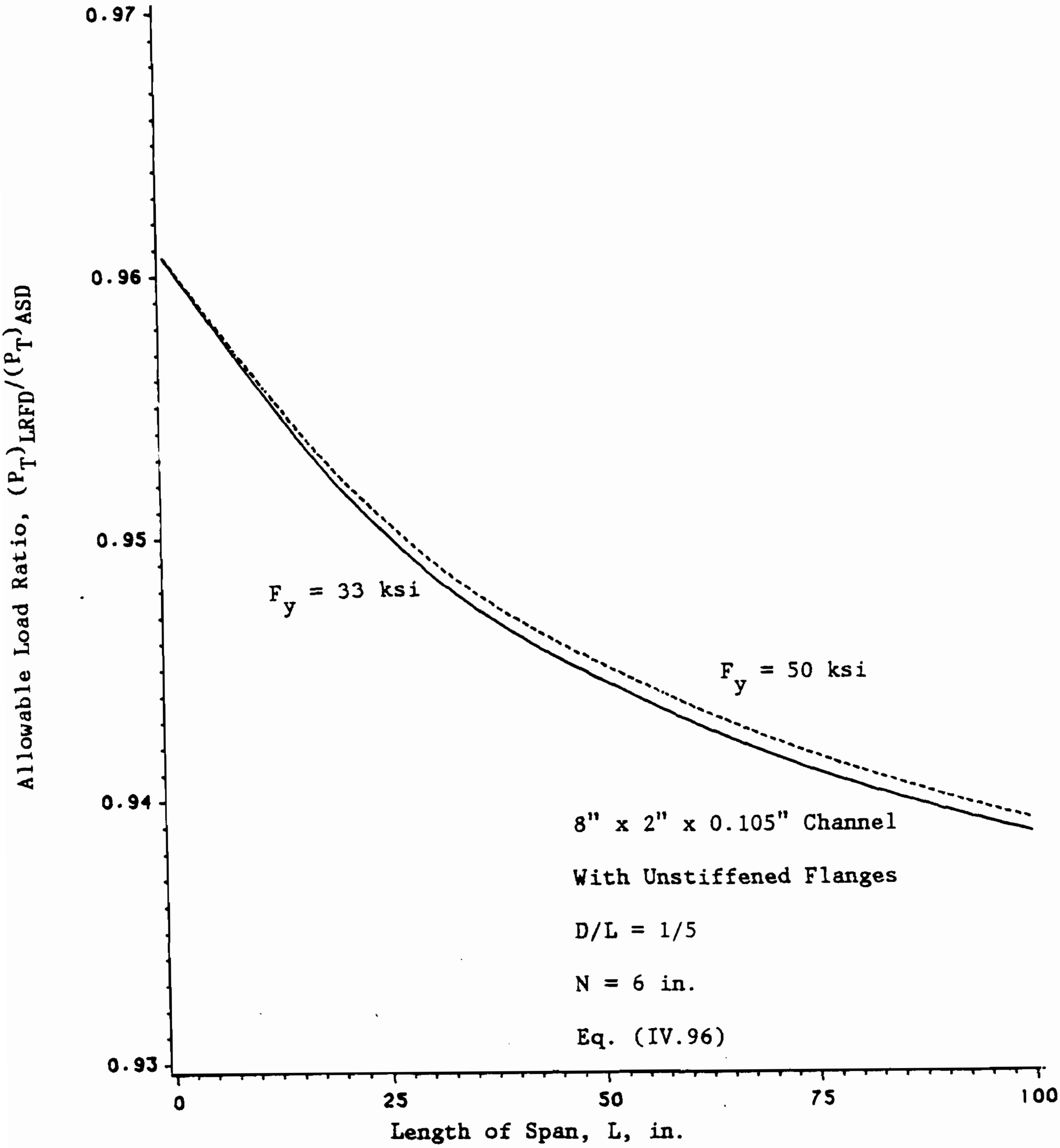

Figure 20. Allowable Load Ratio vs. Span Length for Combined Bending and Web Crippling - Case 3

Table IV.15
I-Section With Stiffened Flanges

Section	h/t	P_n (Kips)	M_n (K-in.)	ϕM_n (K-in.)	K_w
8x6x0.105	70.62	28.966	249.538	237.061	1.5708

* F_y = 33 ksi, L = 60 in., N = 6 in.

Table IV.16
8 in. x 6 in. x 0.105 in. I-Sections With Stiffened Flanges
for Various Lengths and Yield Points

F_y (ksi)	L (in.)	P_n (Kips)	M_n (K-in.)	ϕM_n (K-in.)	K_w
33	0	28.966	249.538	237.061	1.8887
	25	28.966	249.538	237.061	1.6770
	50	28.966	249.538	237.061	1.5920
	75	28.966	249.538	237.061	1.5462
	100	28.966	249.538	237.061	1.5175
50	0	31.406	361.223	343.162	1.8887
	25	31.406	361.223	343.162	1.7111
	50	31.406	361.223	343.162	1.6272
	75	31.406	361.223	343.162	1.5873
	100	31.406	361.223	343.162	1.5463

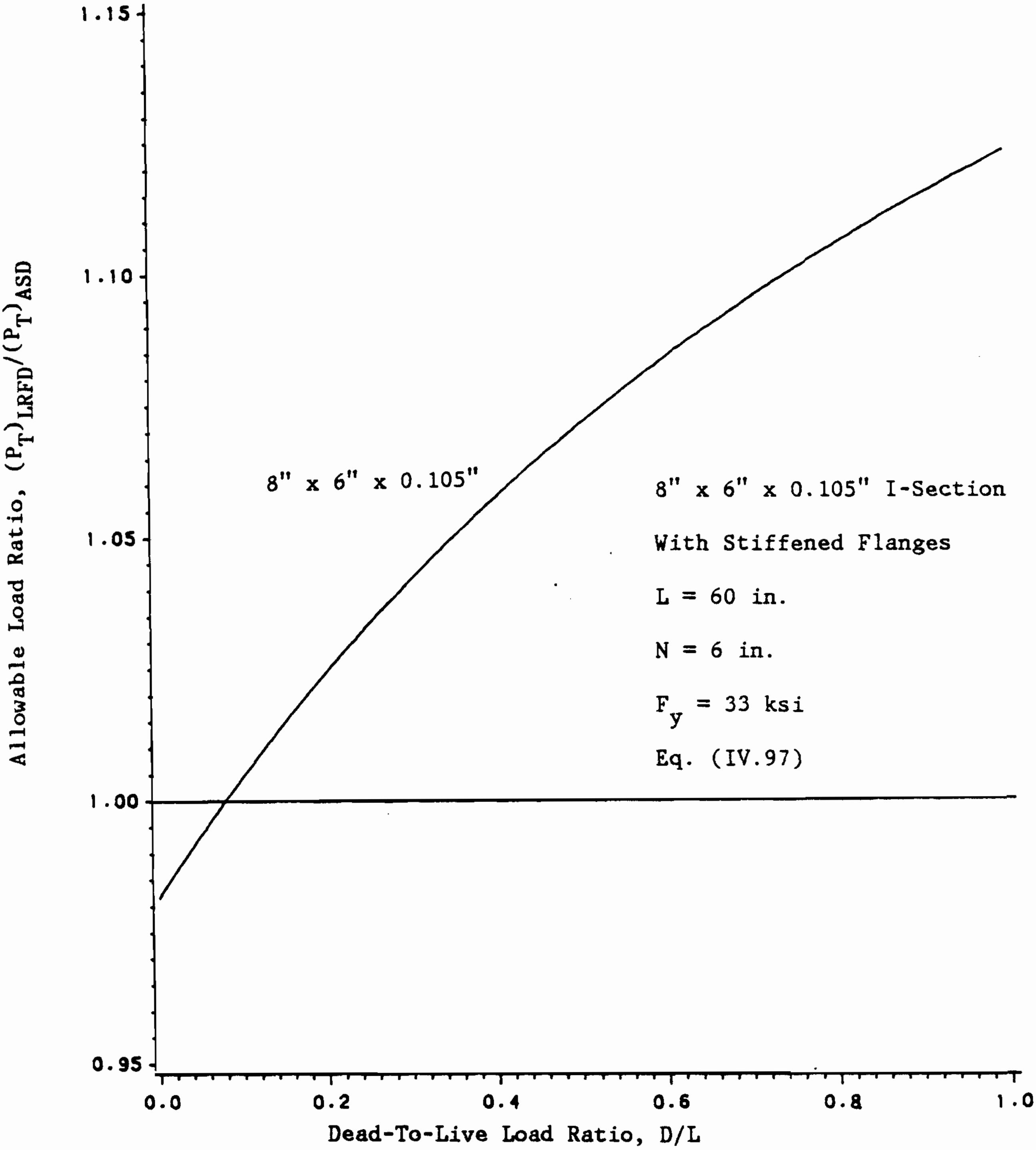

Figure 21. Allowable Load Ratio vs. D/L Ratio for Combined Bending and Web Crippling - Case 4

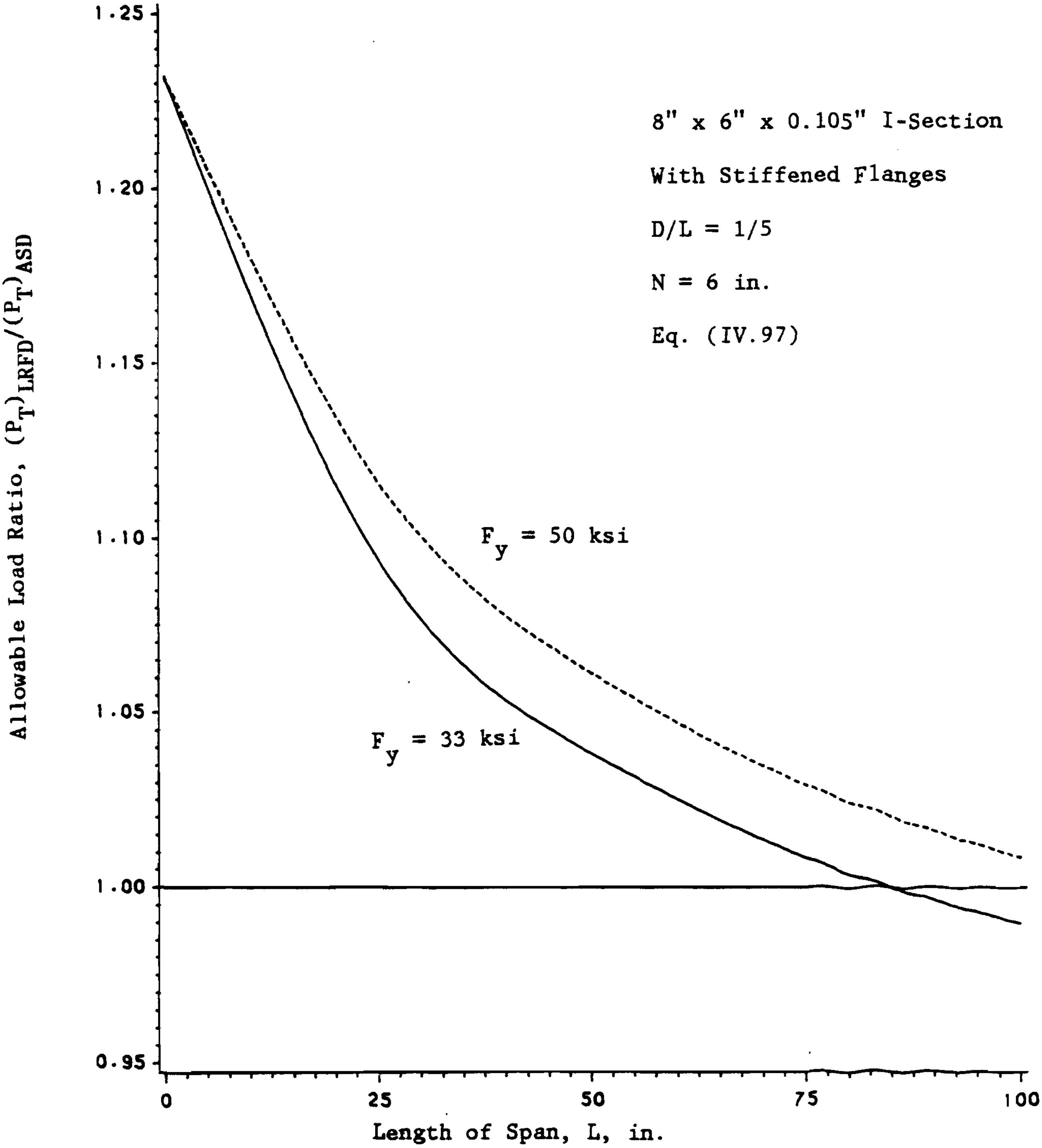

Figure 22. Allowable Load Ratio vs. Span Length for Combined Bending and Web Crippling - Case 4

V. CONCENTRICALLY LOADED COMPRESSION MEMBERS

A. GENERAL

Cold-formed steel concentrically loaded compression members have three possible modes of failure. Short and compact columns will fail by yielding. Local buckling of an individual element could occur if the flat-width to thickness ratio is large. Overall column buckling of intermediate and long columns could occur in one of three buckling modes: flexural buckling, torsional buckling, and torsional-flexural buckling.

B. ALLOWABLE STRESS DESIGN

Section C4 of the 1986 AISI Specification contains the following requirements for compression 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, defined in that section.

(a) The axial load shall not exceed P_a calculated as follows:

$$P_a = P_n/\Omega_c \tag{V.1}$$

where

$$P_n = A_eF_n \tag{V.2}$$

A_e = Effective area at the stress F_n.

F_n is determined as follows:

For $F_e > F_y/2$ $\qquad F_n = F_y(1-F_y/4F_e)$ (V.3)

For $F_e \le F_y/2$ $\qquad F_n = F_e$ (V.4)

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

Ω_c = Factor of safety for axial compression

= 1.92, except when F_e is determined according to Section C4.1 of the AISI Specification for fully effective sections having wall thickness greater than 0.09 inches and $F_e > F_y/2$. In this case,

$\Omega_c = 5/3+(3/8)R-(1/8)R^3$

where

$R = \sqrt{(F_y/2F_e)}$

(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:

$$P_n = A\pi^2E/[25.7(w/t)^2] \qquad (V.5)$$

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:

$$F_e = \pi^2 E/(KL/r)^2 \qquad (V.6)$$

where

E = Modulus of elasticity

K = Effective length factor

L = Unbraced length of member

r = Radius of gyration of the full, unreduced 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:

$$F_e = \left[(\sigma_{ex}+\sigma_t) - \sqrt{(\sigma_{ex}+\sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t}\right]/(2\beta) \qquad (V.7)$$

where

σ_t and σ_{ex} are as defined in Article IV of this report

$$\beta = 1-(x_0/r_0)^2 \qquad (V.8)$$

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

$$F_e = \sigma_t\sigma_{ex}/(\sigma_t+\sigma_{ex}) \qquad (V.9)$$

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.

C. LRFD CRITERIA

Based on Section C4 of the LRFD Specification, 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, the factored axial strength, $\phi_c P_n$, shall be determined with $\phi_c = 0.85$ and the nominal axial strength, P_n, calculated exactly the same as that specified in Section C4 of the AISI Specification.

D. COMPARISON

The unfactored load applied to the member can be computed for both design methods by using the following formula:

$$P_T = P_{DL}+P_{LL} \qquad (V.10)$$

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

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

$$(P_a)_{ASD} = P_n/\Omega_c \qquad (V.11)$$

For LRFD, the allowable axial load can be computed by using the following equation developed from Eq. (II.6):

$$(P_a)_{LRFD} = \phi_c P_n(D/L+1)/(1.2D/L+1.6) \qquad (V.12)$$

Then, the allowable load ratio can be determined as follow:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = \frac{\phi_c P_n}{P_n/\Omega_c}\left[\frac{D/L+1}{1.2D/L+1.6}\right] = 0.85\Omega_c \frac{D/L+1}{1.2D/L+1.6} \qquad (V.13)$$

For fully effective sections having wall thickness greater than 0.09 in. and $F_e > F_y/2$,

$$\Omega_c = 5/3+(3/8)R-(1/8)R^3$$

Therefore, the allowable load ratio is

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 0.85\left(\frac{5}{3} + \frac{3}{8}R - \frac{1}{8}R^3\right)\frac{D/L+1}{1.2D/L+1.6} \qquad (V.14)$$

For all other cases, $\Omega_c = 1.92 = 23/12$, therefore the allowable load ratio is

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 0.85(23/12)\frac{D/L+1}{1.2D/L+1.6} = 1.629\frac{D/L+1}{1.2D/L+1.6} \qquad (V.15)$$

Figure 23 shows the allowable load ratio versus dead-to-live load ratio for the columns used to develop Eq. (V.15). 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.

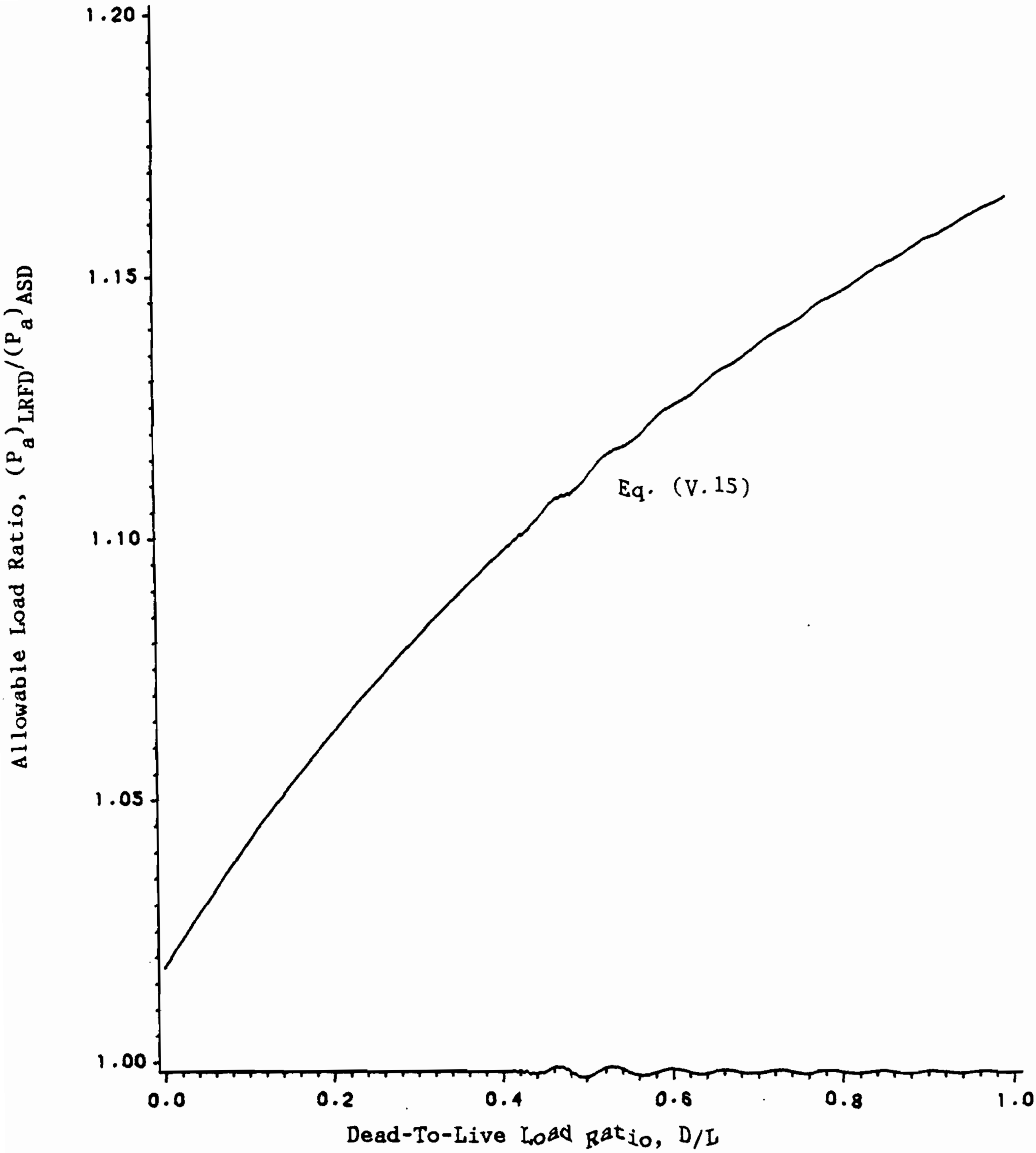

Figure 23. Allowable Load Ratio vs. D/L Ratio for Column Buckling

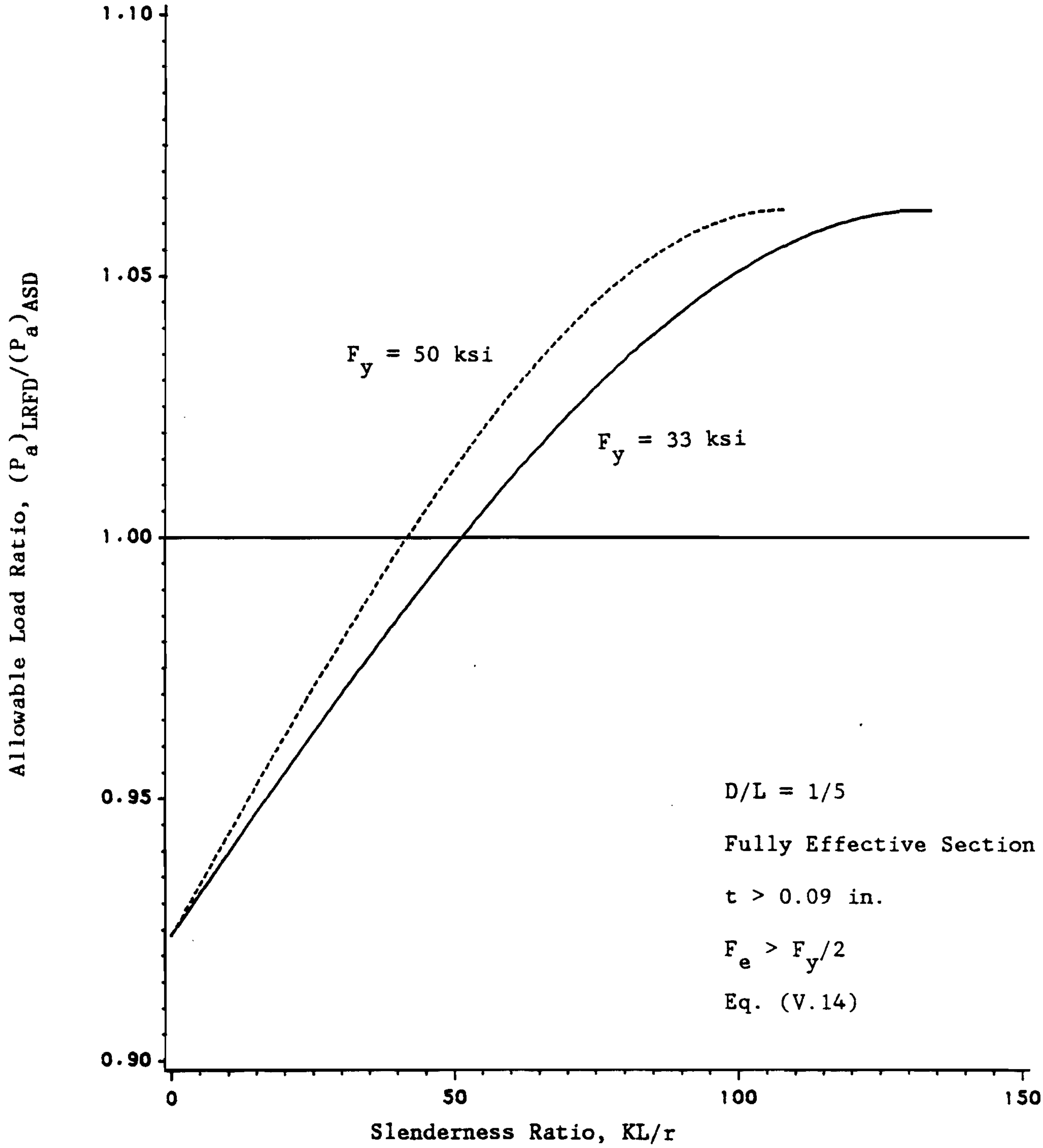

Figure 24. Allowable Load Ratio vs. Slenderness Ratio for Flexural Buckling of Columns

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 24. For this case, the LRFD criteria were found to be conservative for short columns as compared with allowable stress design. As shown in Figure 24, higher yield point materials give slightly higher values of the allowable load ratio.

VI. BEAM-COLUMNS

A. GENERAL

Beam-columns are structural members subjected to combined axial load and bending. The structural behavior of beam-columns depends on the shape and dimensions of the cross section, the location of the applied eccentric load, column length, and condition of bracing[45]. Interaction formulas are used to analyze beam-columns for flexural and torsional-flexural buckling.

B. ALLOWABLE STRESS DESIGN

In the 1986 edition of AISI Specification, the design criteria for combined axial load and bending are stated in Section C5 as follows:

The axial force and bending moments shall satisfy the following interaction equations:

$$P/P_a + C_{mx}M_x/(M_{ax}\alpha_x) + C_{my}M_y/(M_{ay}\alpha_y) \le 1.0 \tag{VI.1}$$

$$P/P_{ao} + M_x/M_{axo} + M_y/M_{ayo} \le 1.0 \tag{VI.2}$$

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

$$P/P_a + M_x/M_{ax} + M_y/M_{ay} \le 1.0 \tag{VI.3}$$

where

P = Applied axial load

M_x and M_y = Applied moments with respect to 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 results in a lower value of P_a

P_a = Allowable axial load

P_{ao} = Allowable axial load determined with $F_n = F_y$

M_{ax} and M_{ay} = Allowable moments about the centroidal axes

M_{axo} and M_{ayo} = Allowable moments about the centroidal axes excluding lateral buckling

$1/\alpha_x$, $1/\alpha_y$ = Magnification factors

$$= 1/[1-(\Omega_c P/P_{cr})] \qquad \text{(VI.4)}$$

Ω_c = Factor of safety used in determining P_a

$$P_{cr} = \pi^2 EI_b/(K_b L_b)^2 \qquad \text{(VI.5)}$$

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

$C_m = 0.6 - 0.4(M_1/M_2)$ (VI.6)

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,
   $C_m = 1.0$.

C. LRFD CRITERIA

According to Scetion C5 of the LRFD Specification, the design values P_D, M_{DX} and M_{DY} computed on the basis of factored loads, shall satisfy the following interaction equations:

$$P_D/\Phi_c P_n + C_{mx} M_{DX}/\Phi M_{nx} \alpha_{nx} + C_{my} M_{DY}/\Phi M_{ny} \alpha_{ny} \leq 1.0 \quad \text{(VI.7)}$$

$$P_D/\Phi_c P_{no} + M_{DX}/\Phi M_{nxo} + M_{DY}/\Phi M_{nyo} \leq 1.0 \quad \text{(VI.8)}$$

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

$$P_D/\Phi_c P_n + M_{DX}/\Phi M_{nx} + M_{DY}/\Phi M_{ny} \leq 1.0 \quad \text{(VI.9)}$$

where

P_D = Factored design axial load

M_{DX} and M_{DY} = Factored design moments with respect to the centroidal axes of the effective section determined for the factored design axial load alone. For angle sections, M_{DY} shall be taken either as the factored moment or the factored moment plus $P_D L/1000$, 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 beam strengths about the centroidal axes

M_{nxo} and M_{nyo} = Nominal beam strengths about the centroidal axes, excluding lateral buckling

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

$= 1/(1-P_D/\Phi_c P_E)$ (VI.10)

Φ = 0.95 or 0.90 for bending strength (Section C3.1.1) or 0.90 for laterally unbraced beam (Section C3.1.2)

ϕ_c = 0.85

P_E = $\pi^2 EI_b/(K_b L_b)^2$ (VI.11)

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

   $C_m = 0.6-0.4(M_1/M_2)$ (VI.12)

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,

$C_m = 1.0$.

D. COMPARISON

Because of the complexity of the interaction formulas, the 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 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:

$$\frac{P}{P_a} = \frac{P_T}{P_n/\Omega_c} = \frac{\Omega_c P_T}{P_n} \qquad \text{(VI.13)}$$

$$\frac{M}{M_a} = \frac{M_T}{0.6M_n} = \frac{(M_T/M_{no})(M_{no}/M_n)}{0.6} \qquad \text{(VI.14)}$$

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 which is defined in Article V

Substitution of Eqs. (VI.13) and (VI.14) into Eq. (VI.1) results in the following expression :

$$\frac{\Omega_c P_T}{P_n} + \frac{C_m(M_T/M_{no})(M_{no}/M_n)}{0.6(1-\Omega_c P_T/P_{cr})} = 1.0 \qquad \text{(VI.15)}$$

By solving for P_T in the first term of Eq. (VI.15), the following equation for allowable load is obtained :

$$(P_T)_{ASD1} = \left[1 - \frac{C_m(M_T/M_{no})(M_{no}/M_n)}{0.6(1-\Omega_c P_T/P_{cr})}\right]\frac{P_n}{\Omega_c} \qquad \text{(VI.16)}$$

Equation (VI.16) is based on Eq. (VI.1) for failure at the midlength of the beam-column and requires a solution by iterations.

The following expressions were used to solve for the allowable load based on Eq. (VI.2) :

$$\frac{P}{P_{ao}} = \frac{P_T}{P_{no}/\Omega_c} = \frac{\Omega_c P_T}{P_{no}} \tag{VI.17}$$

$$\frac{M}{M_{ao}} = \frac{M_T}{0.6M_{no}} = \frac{(M_T/M_{no})}{0.6} \tag{VI.18}$$

Substitution of Eqs. (VI.17) and (VI.18) into Eq. (VI.2) results in the following expression :

$$\frac{\Omega_c P_T}{P_{no}} + \frac{(M_T/M_{no})}{0.6} = 1.0 \tag{VI.19}$$

By solving for P_T in Eq. (VI.19), the following equation for allowable load is obtained :

$$(P_T)_{ASD2} = \left[1 - \frac{(M_T/M_{no})}{0.6}\right]\frac{P_{no}}{\Omega_c} \tag{VI.20}$$

Equation (VI.20) is based on Eq. (VI.2) for failure at the braced points.

When $P/P_a \leq 0.15$, Eq. (VI.3) can be used in lieu of Eqs. (VI.1) and (VI.2). Equation (VI.3) can be written in the following form by using Eqs. (VI.13) and (VI.14) :

$$\frac{\Omega_c P_T}{P_n} + \frac{(M_T/M_{no})(M_{no}/M_n)}{0.6} = 1.0 \tag{VI.21}$$

By solving for P_T in Eq. (VI.21), the following equation for allowable load is obtained :

$$(P_T)_{ASD3} = \left[1 - \frac{(M_T/M_{no})(M_{no}/M_n)}{0.6}\right] \frac{P_n}{\Omega_c} \tag{VI.22}$$

Equation (VI.22) is based on Eq. (VI.3) for flexural failure when the effect of the secondary moment is neglected.

For LRFD, the allowable axial loads were computed in accordance with Eq. (II.6) as follows :

$$\frac{P_D}{\phi_c P_n} = \frac{1.2D/L+1.6}{D/L+1}\left[\frac{P_T}{\phi_c P_n}\right] \tag{VI.23}$$

$$\frac{M_D}{\phi M_n} = \frac{1.2D/L+1.6}{D/L+1}\left[\frac{(M_T/M_{no})(M_{no}/M_n)}{\phi}\right] \tag{VI.24}$$

$$\frac{P_D}{\phi_c P_E} = \frac{1.2D/L+1.6}{D/L+1}\left[\frac{P_T}{\phi_c P_E}\right] \tag{VI.25}$$

Substitution of Eqs. (VI.23), (VI.24), and (VI.25) into Eq. (VI.7) results in the following expression :

$$\frac{1.2D/L+1.6}{D/L+1}\left\{\frac{P_T}{\phi_c P_n} + \frac{C_m(M_T/M_{no})(M_{no}/M_n)}{\phi\left[1-(1.2D/L+1.6)P_T/(D/L+1)\phi_c P_E\right]}\right\} = 1.0 \tag{VI.26}$$

By solving for P_T in the first term of Eq. (VI.26), the following equation for allowable load is obtained :

$$(P_T)_{LRFD1} = \left\{ \frac{D/L+1}{1.2D/L+1.6} - \frac{C_m(M_T/M_{no})(M_{no}/M_n)}{\phi\left[1-(1.2D/L+1.6)P_T/(D/L+1)\phi_c P_E\right]} \right\} \phi_c P_n \quad (VI.27)$$

Equation (VI.27) is based on Eq. (VI.7) for flexural failure at the midlength of the beam-column and requires a solution by iterations.

The following expressions were used to solve for the allowable load based on Eq. (VI.8) :

$$\frac{P_D}{\phi_c P_{no}} = \frac{1.2D/L+1.6}{D/L+1} \left[\frac{P_T}{\phi_c P_{no}} \right] \quad (VI.28)$$

$$\frac{M_D}{\phi M_{no}} = \frac{1.2D/L+1.6}{D/L+1} \left[\frac{(M_T/M_{no})}{\phi} \right] \quad (VI.29)$$

Substitution of Eqs. (VI.28) and (VI.29) into Eq. (VI.8) results in the following expression :

$$\frac{1.2D/L+1.6}{D/L+1} \left[\frac{P_T}{\phi_c P_{no}} + \frac{(M_T/M_{no})}{\phi} \right] = 1.0 \quad (VI.30)$$

By solving for P_T in Eq. (VI.30), the following equation for allowable load is obtained :

$$(P_T)_{LRFD2} = \left[\frac{D/L+1}{1.2D/L+1.6} - \frac{(M_T/M_{no})}{\phi} \right] \phi_c P_{no} \quad (VI.31)$$

Equation (VI.31) is based on Eq. (VI.8) for failure at the braced points.

When $P_D/(\phi_c P_n) \leq 0.15$, Eq. (VI.9) can be used in lieu of Eqs. (VI.7) and (VI.8). Equation (VI.9) can be written in the following form by using Eqs. (VI.23) and (VI.24) :

$$\frac{1.2D/L+1.6}{D/L+1}\left[\frac{P_T}{\phi_c P_n} + \frac{(M_T/M_{no})(M_{no}/M_n)}{\phi}\right] = 1.0 \qquad \text{(VI.32)}$$

By solving for P_T in Eq. (VI.32), the following equation for allowable load is obtained :

$$(P_T)_{LRFD3} = \left[\frac{D/L+1}{1.2D/L+1.6} - \frac{(M_T/M_{no})(M_{no}/M_n)}{\phi}\right]\phi_c P_n \qquad \text{(VI.33)}$$

Equation (VI.33) is based on (VI.9) for flexural failure when the effect of the secondary moment is neglected.

Equations (VI.16), (VI.20), and (VI.22) for determining the allowable axial load based on allowable stress design and Eqs. (VI.27), (VI.31), and (VI.33) 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 25 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. (VI.16) and (VI.27) 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 25. 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 26 shows the allowable load ratio based on Eqs. (VI.20) and (VI.31) versus dead-to-live load ratio for the same I-section used in Figure 25. Figure 26 is based on failure at the braced points which corresponds to Eqs. (VI.20) and (VI.31). For D/L = 0.05, the LRFD criteria give an allowable load about 3% more than the value computed from allowable stress design for all end moment ratios shown in the figure. It also can be seen from this figure that LRFD design will always give a larger allowable load than allowable stress design.

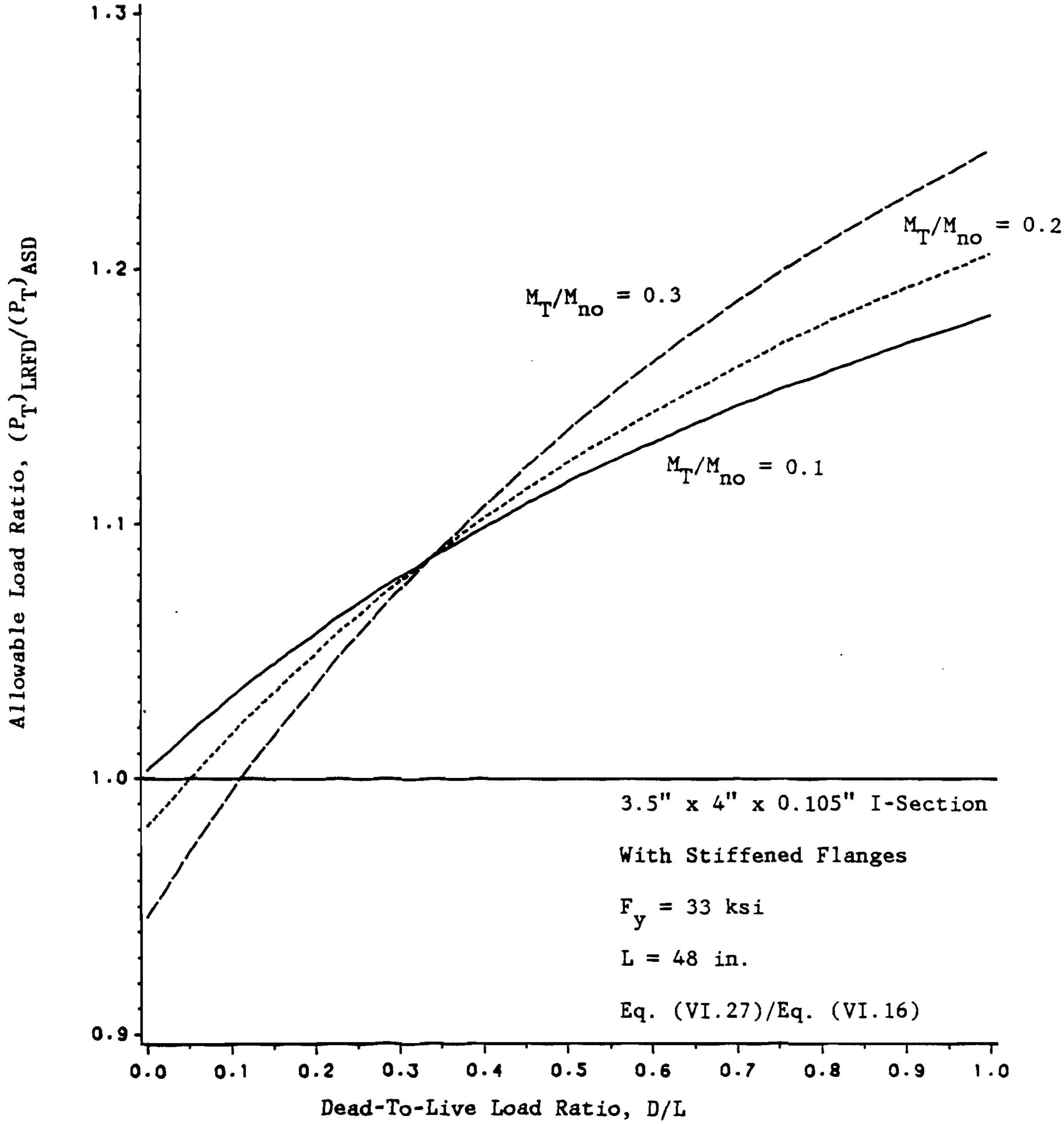

Figure 25. Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case A

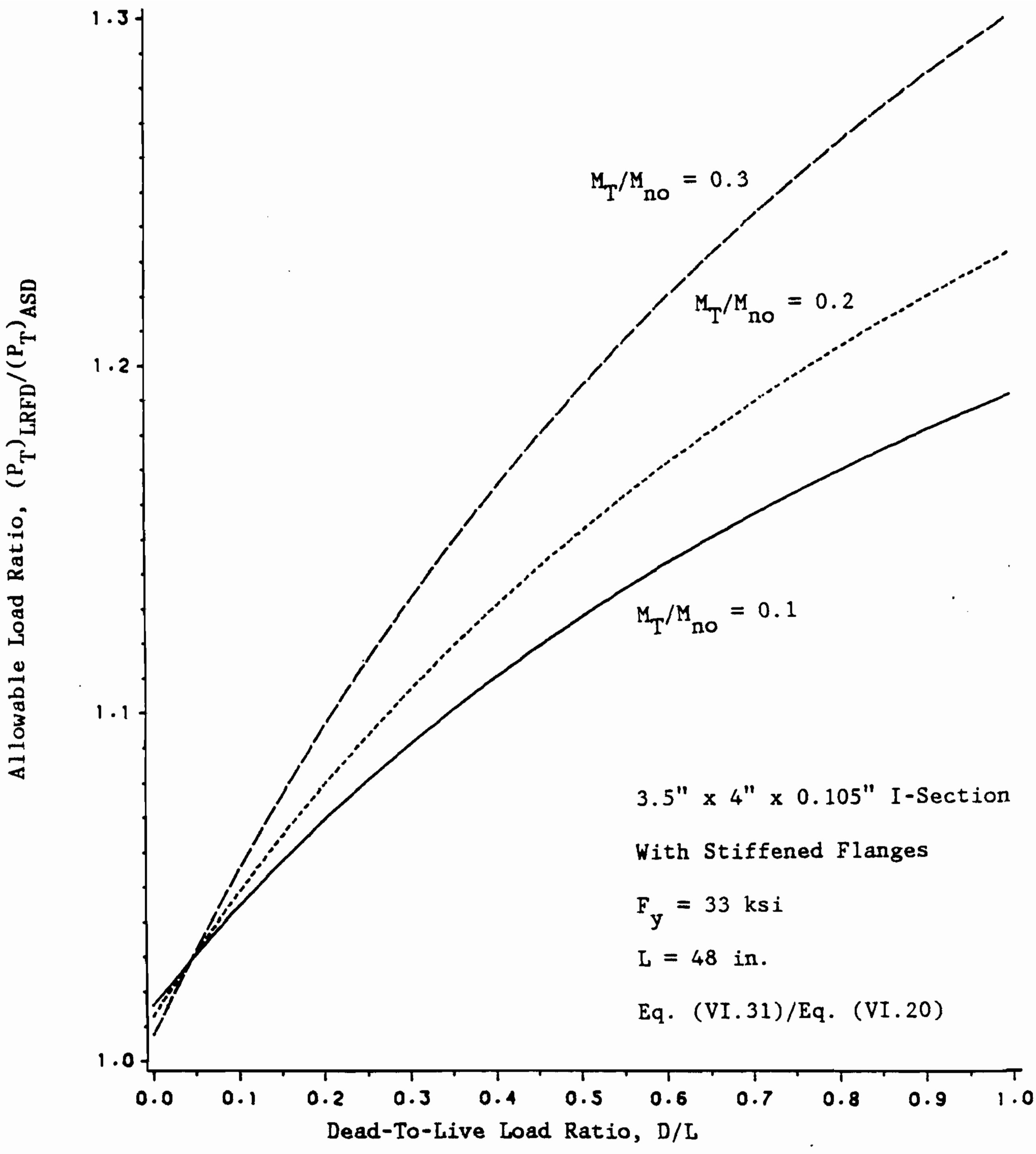

Figure 26. Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case B

Figures 27 and 28 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 ralationships between the allowable load ratio and column length are shown in Figures 27, and 28 for various D/L ratios. Figures 29 and 30 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 25 through 28. 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.

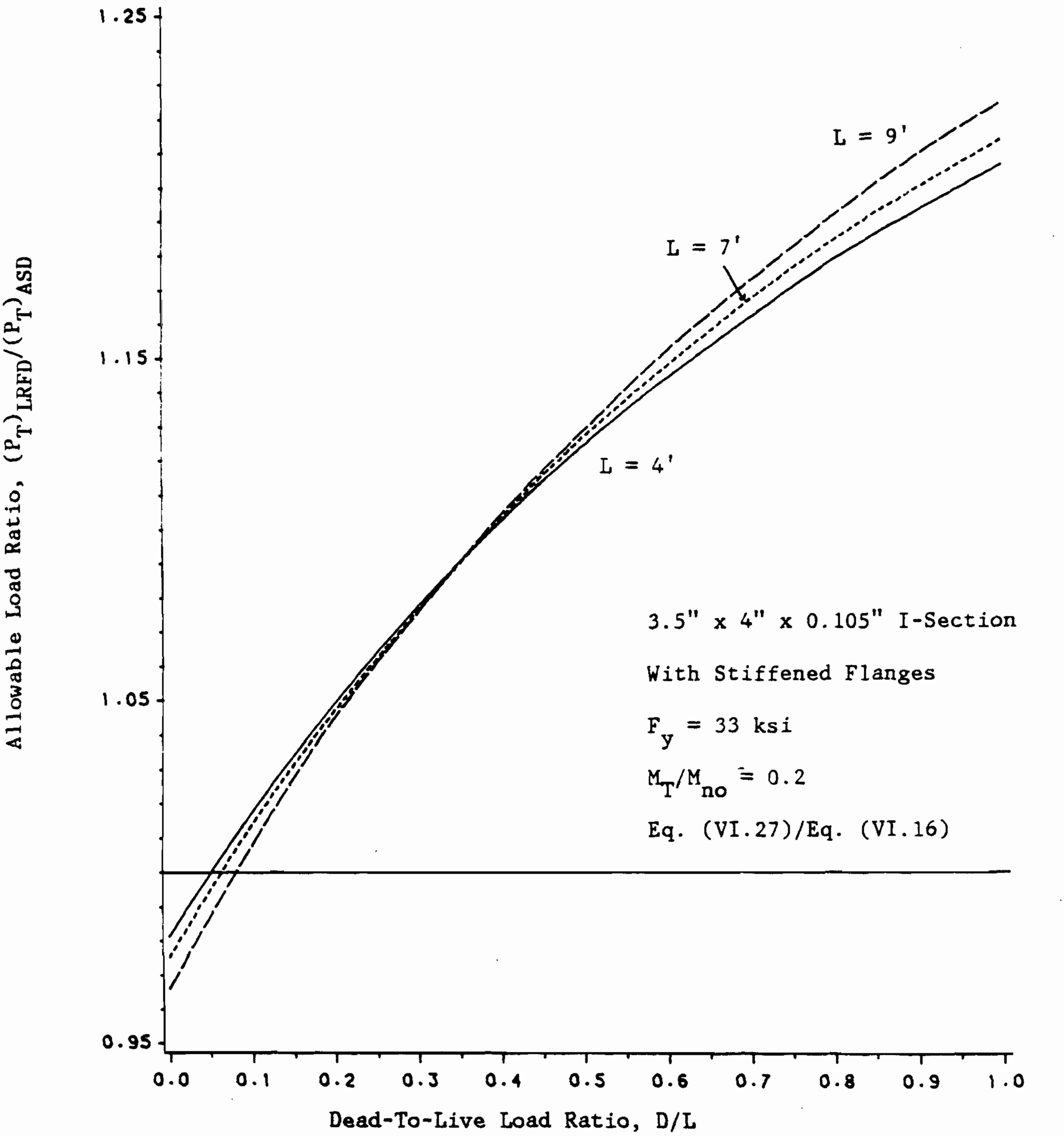

Figure 27. Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case C

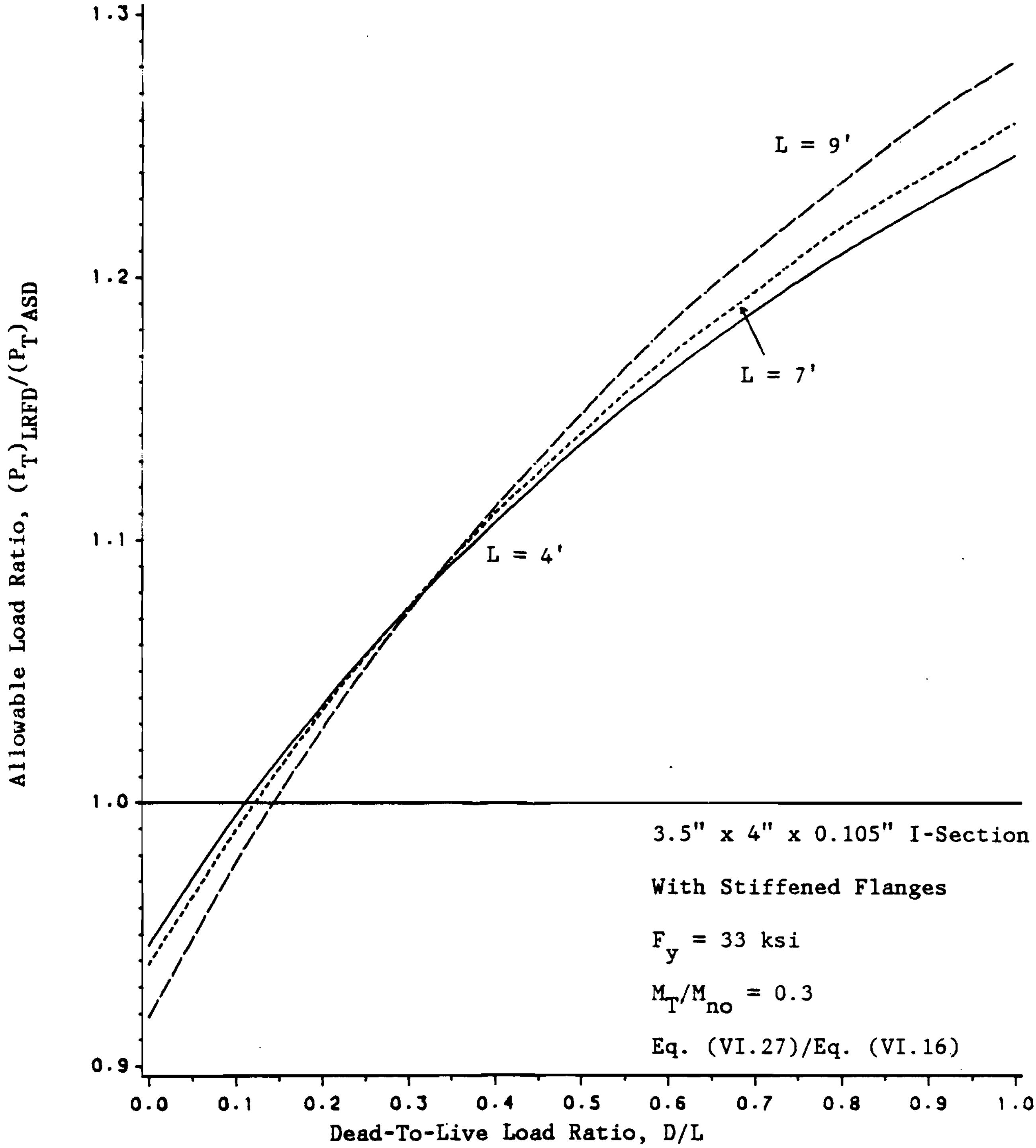

Figure 28. Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case D

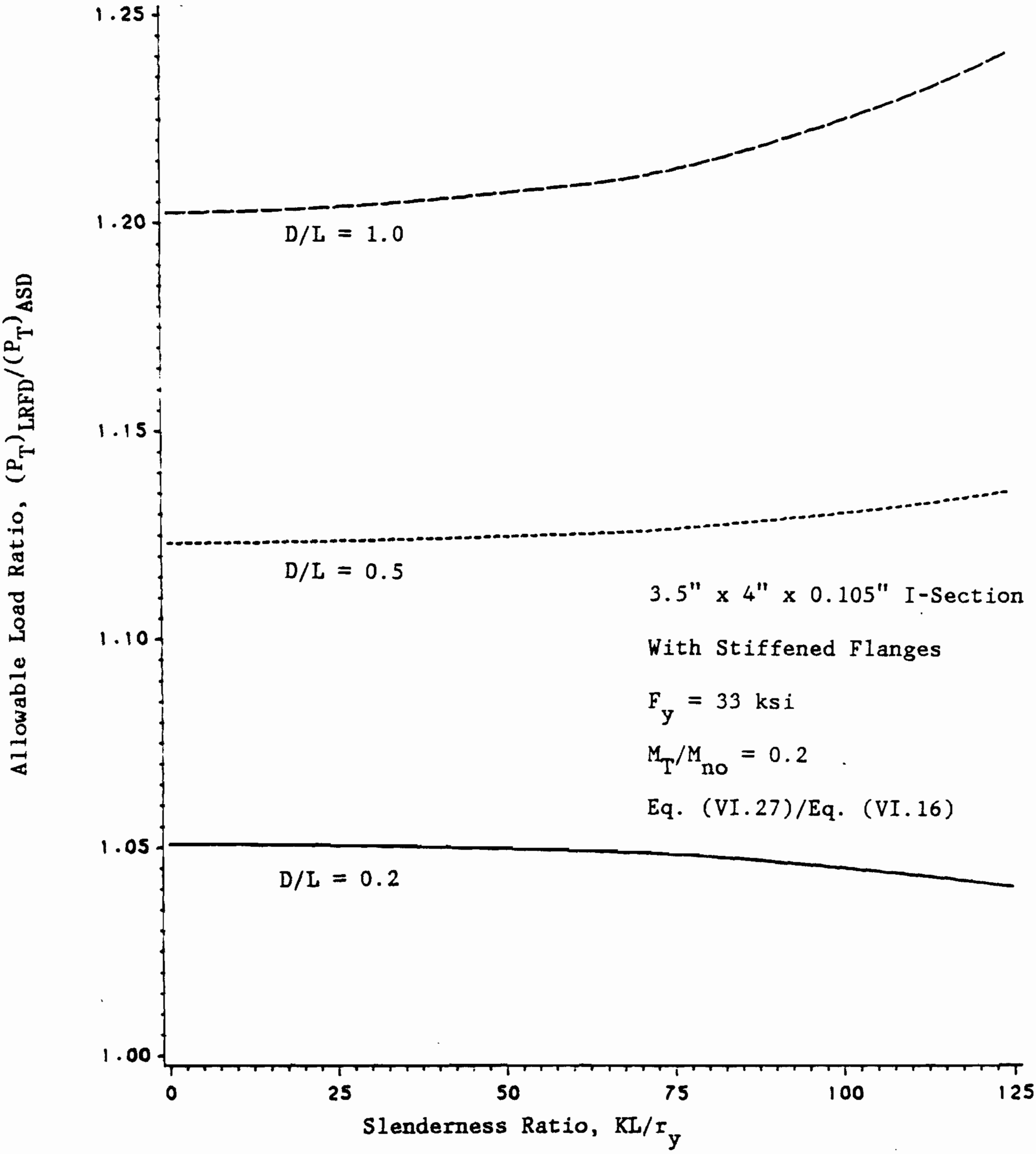

Figure 29. Allowable Load Ratio vs. Slenderness Ratio for Beam-Columns - Case C

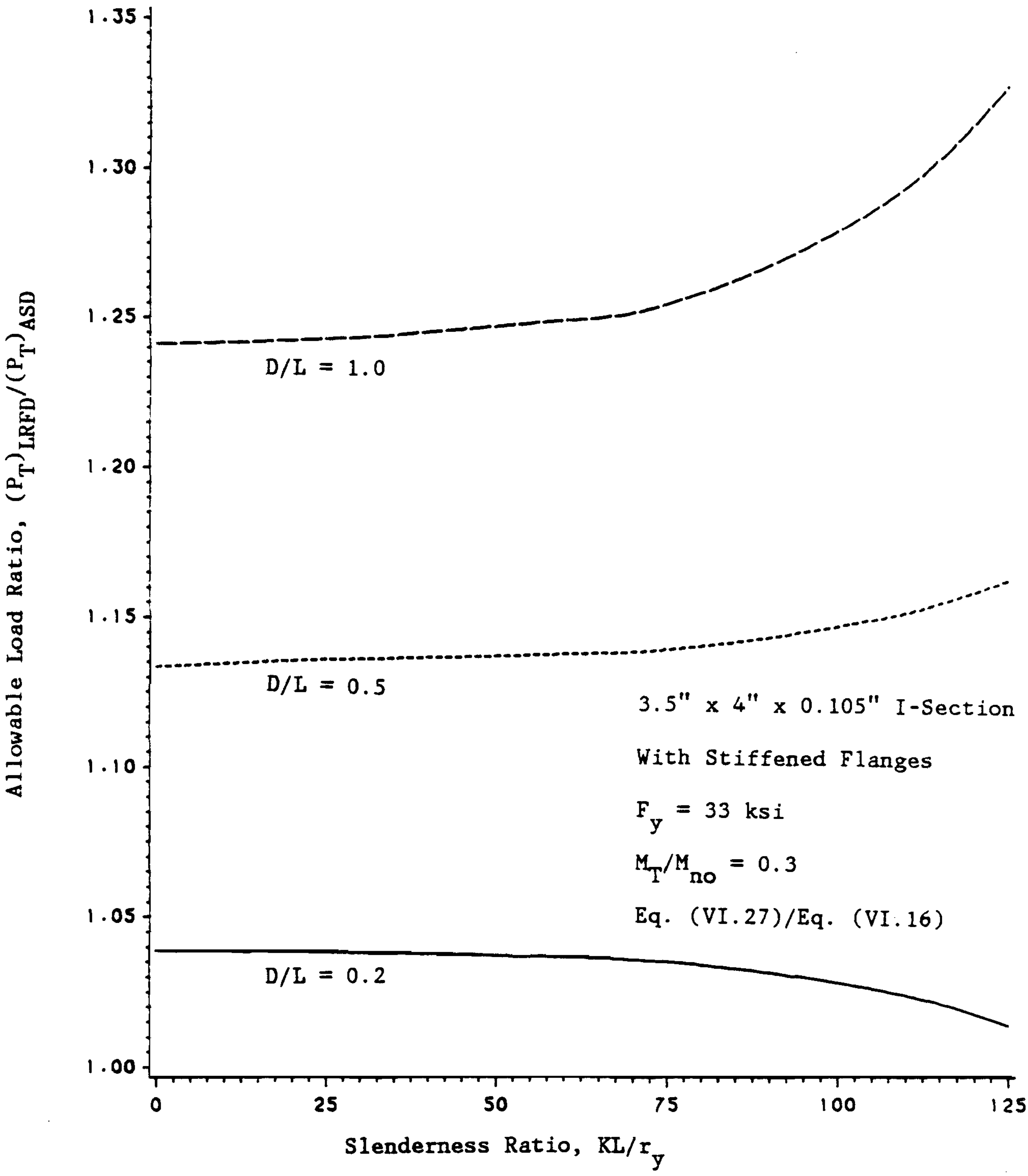

Figure 30. Allowable Load Ratio vs. Slenderness Ratio for Beam-Columns - Case D

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 31 shows the allowable load ratio based on Eqs. (VI.16) and (VI.27) 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 25 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 31 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 31. 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 32 shows the relationship between allowable load ratio and dead-to-live load ratio for the 6 in. deep I-section used in Figure 31 with a consideration of flexural failure at the braced points. This figure is the same as Figure 26 for the 4 in. deep I-section. The curves shown in Figure 32 are applicable for yield points ranging from 33 to 50 ksi and all values of C_m.

I-sections with unstiffened flanges were studied in a similar manner. Figure 33 shows the allowable load ratio versus dead-to-live load ratio for an I-section (4 in. x 2.25 in. x 0.105 in.) having unstiffened flanges with F_y = 33 ksi and an effective column length of 4 ft. This figure is based on flexural failure at the midlength of the beam-column which would govern the design in this case. The allowable load ratio was determined from Eqs. (VI.16) and (VI.27). Figure 33 is similar to Figure 25 prepared for an I-section with stiffened flanges. For D/L = 0.5 and M_T/M_{no} = 0.1, the allowable load obtained from LRFD is 11.7% larger than the value obtained from allowable stress design. For D/L = 0.5 and M_T/M_{no} = 0.3, LRFD would result in an allowable load 14.6% higher than the value determined from allowable stress design.

Figure 34 shows the relationship between allowable load ratio and dead-to-live load ratio for the same I-section used in Figure 33 by considering flexural failure at the braced points. Equations (VI.20) and (VI.31) are used for this type of failure. For D/L = 0.5, the allowable loads obtained from LRFD are from 11.6% to 13.6% greater than the allowable loads determined from allowable stress design for end moment ratios from 0.1 to 0.3.

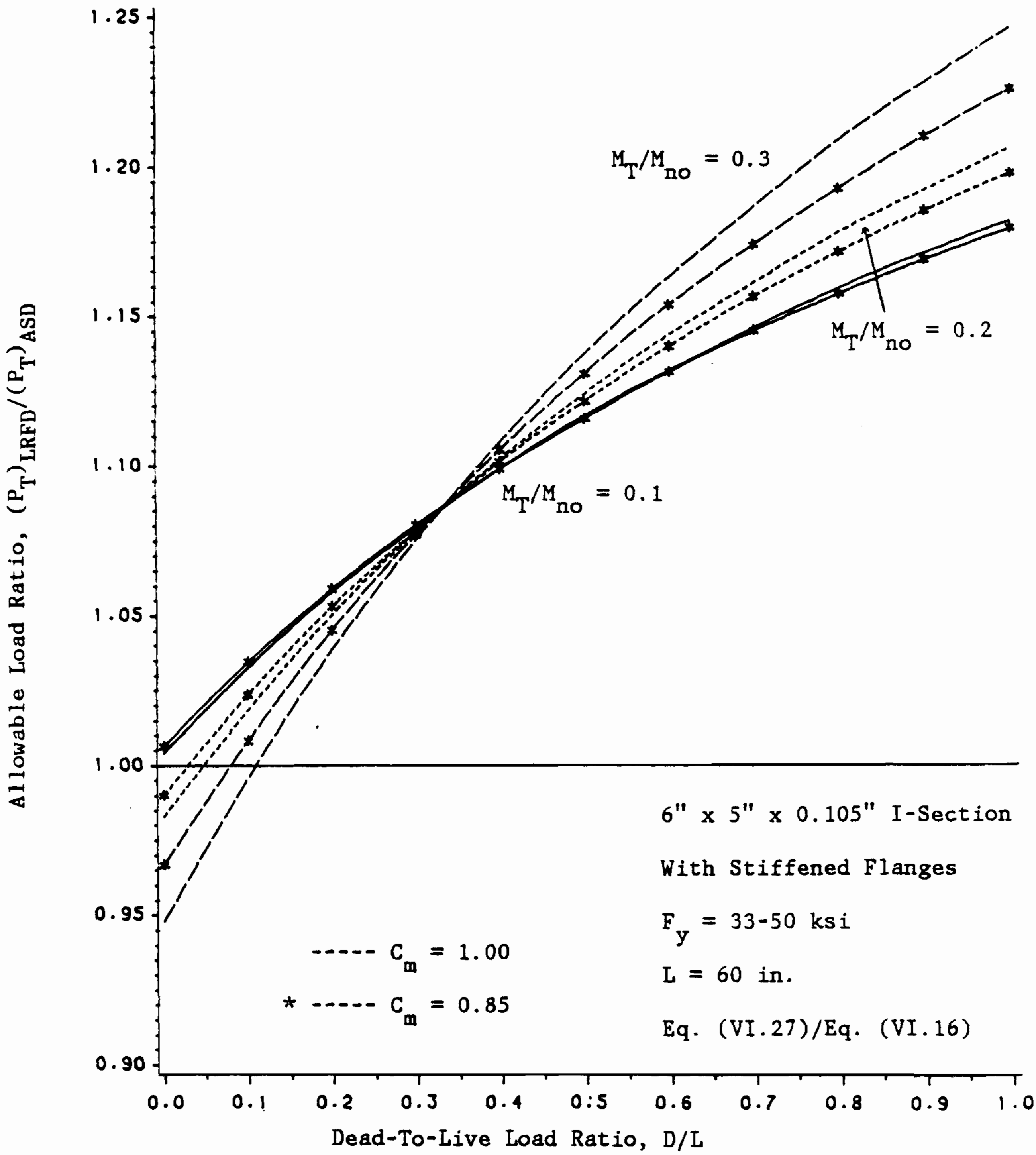

Figure 31. Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case E

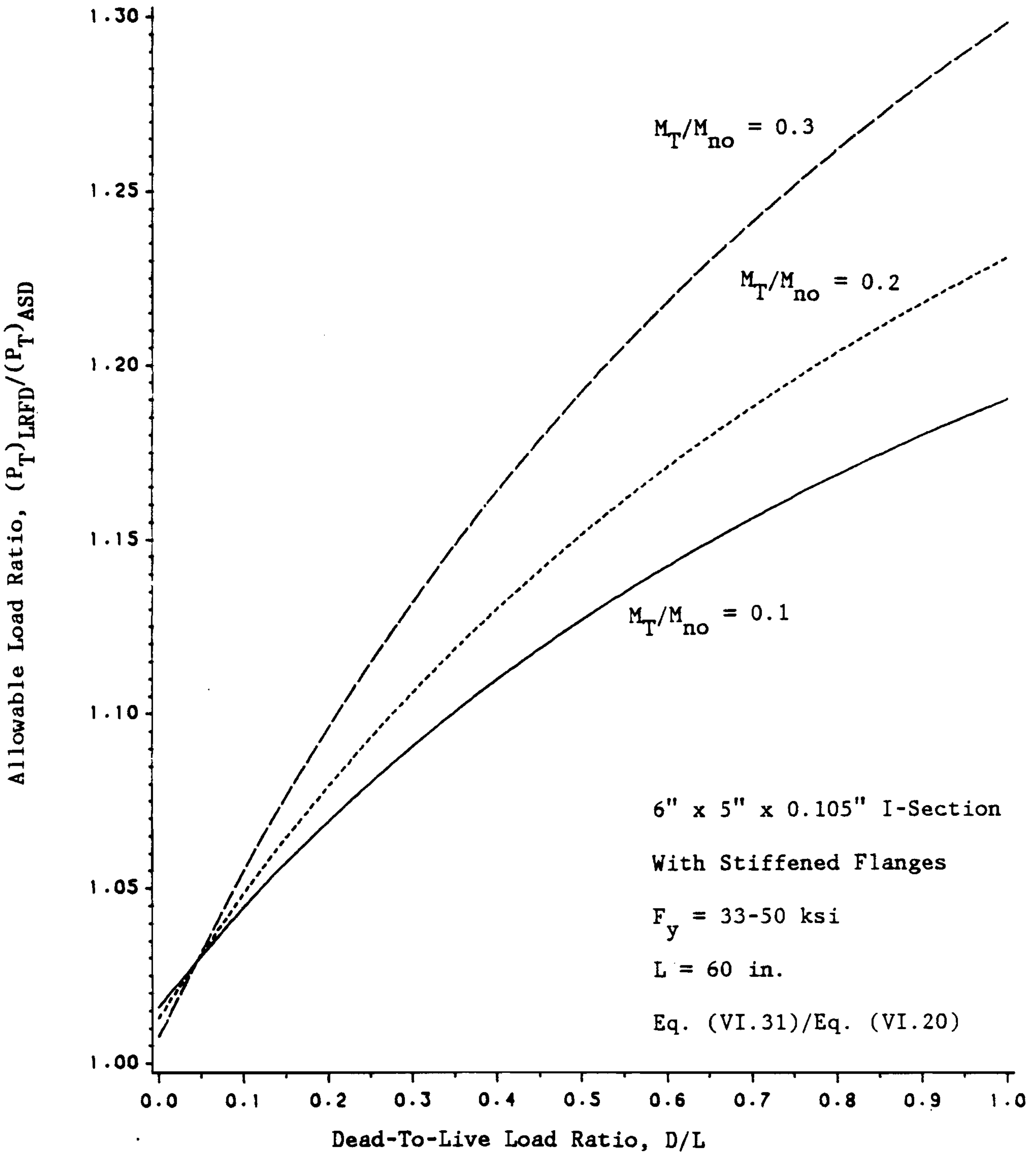

Figure 32. Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case F

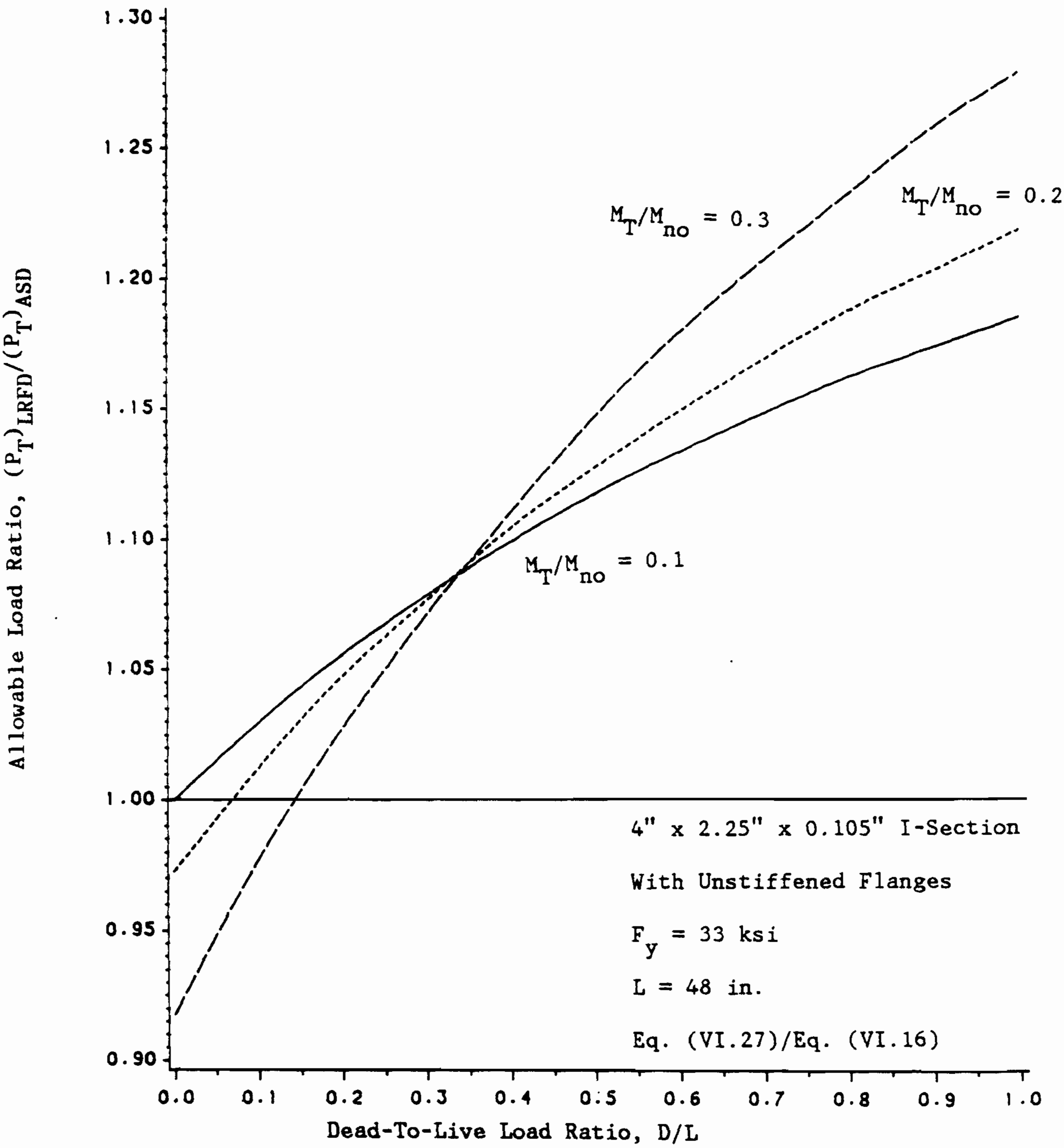

Figure 33. Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case G

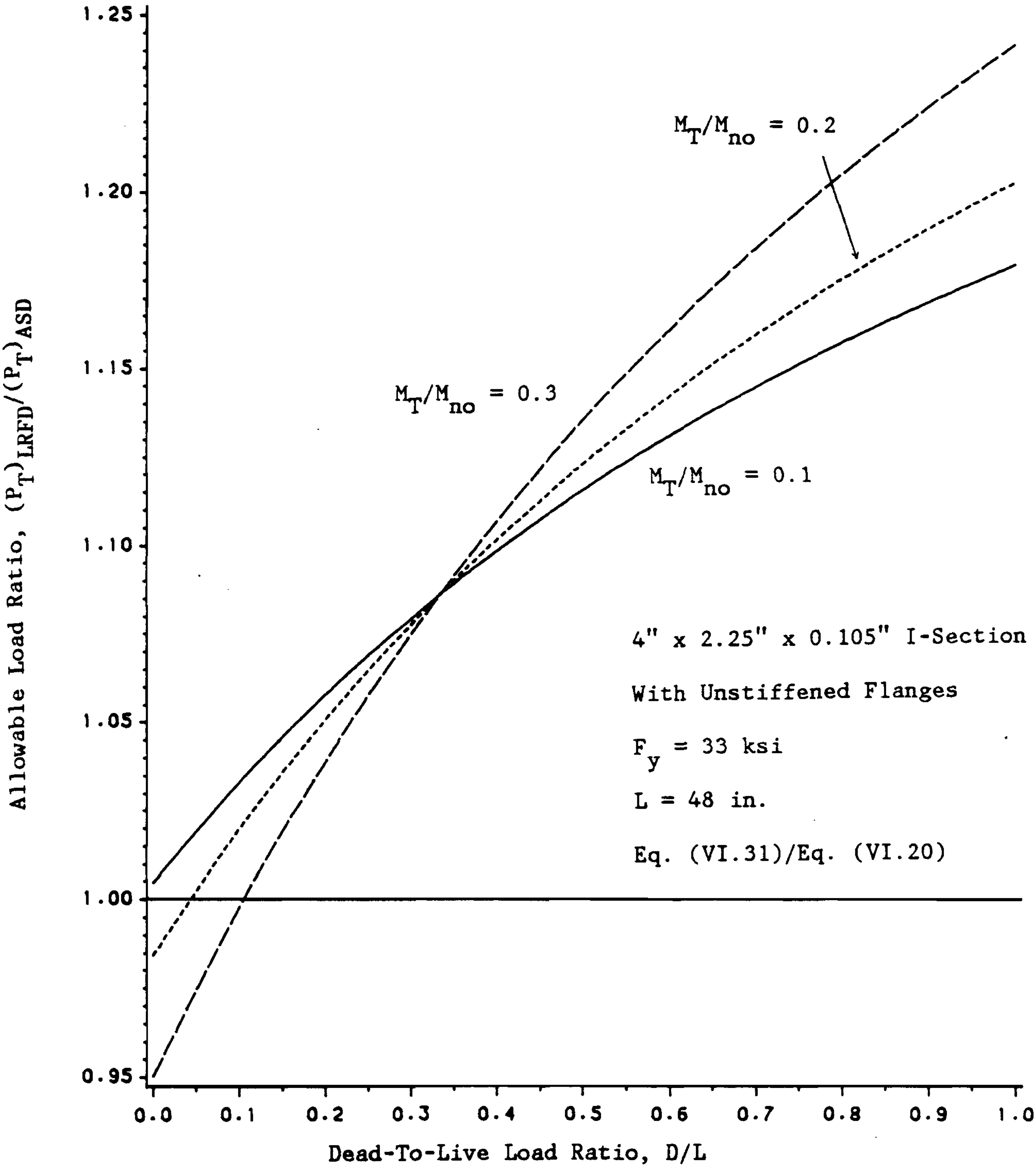

Figure 34. Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case H

Figures 35 and 36 show the allowable load ratio versus dead-to-live load ratio for end moment ratio of 0.1 and 0.2, respectively. Different curves represent different lengths of the I-section (4 in. x 2.25 in. x 0.105 in.) with F_y = 33 ksi. These two figures are similar to Figures 27 and 28 which were prepared for I-section with stiffened flanges. For the values of M_T/M_{no} between 0.1 and 0.2 and D/L = 0.5, the allowable load values obtained from LRFD vary from 11.7% to 15.3% larger than the values obtained from the allowable stress design method.

Figure 37 shows the relationship between allowable load ratio and slenderness ratio, KL/r_y, for the same I-section used in previous figures and for an end moment ratio of 0.1. Each curve in the figure represents a different D/L ratio. The relationship in Figure 37 is similar to the relationship indicated in Figures 29 and 30 which are used in the study of I-sections with stiffened flanges. As shown in this figure, the allowable load ratio increases with increasing slenderness ratio for large D/L ratios. For small D/L ratios, the slenderness ratio has small effect on the allowable load ratio. This figure also shows that for all three D/L ratios, the LRFD method would permit a larger load than the ASD method.

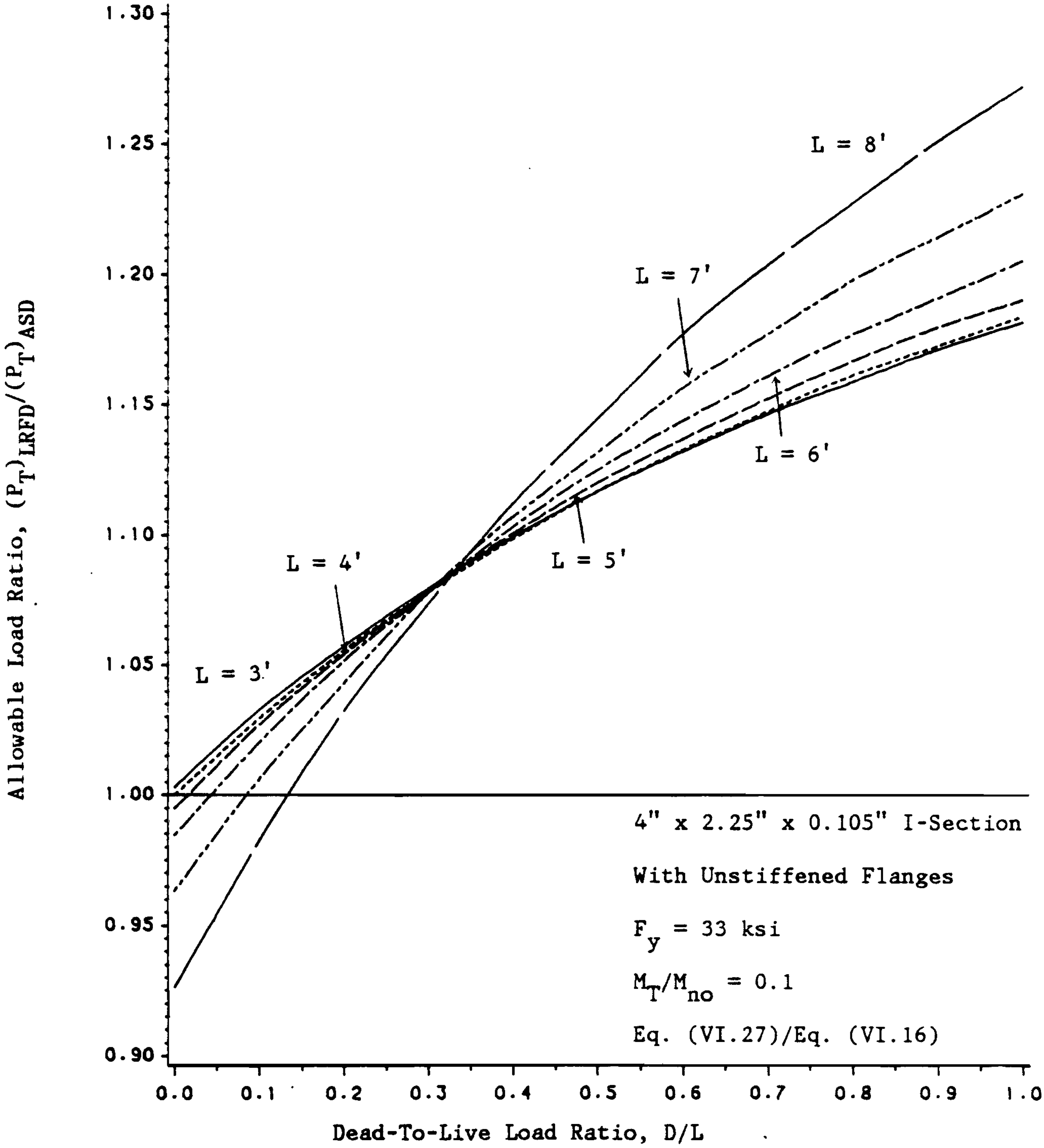

Figure 35. Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case I

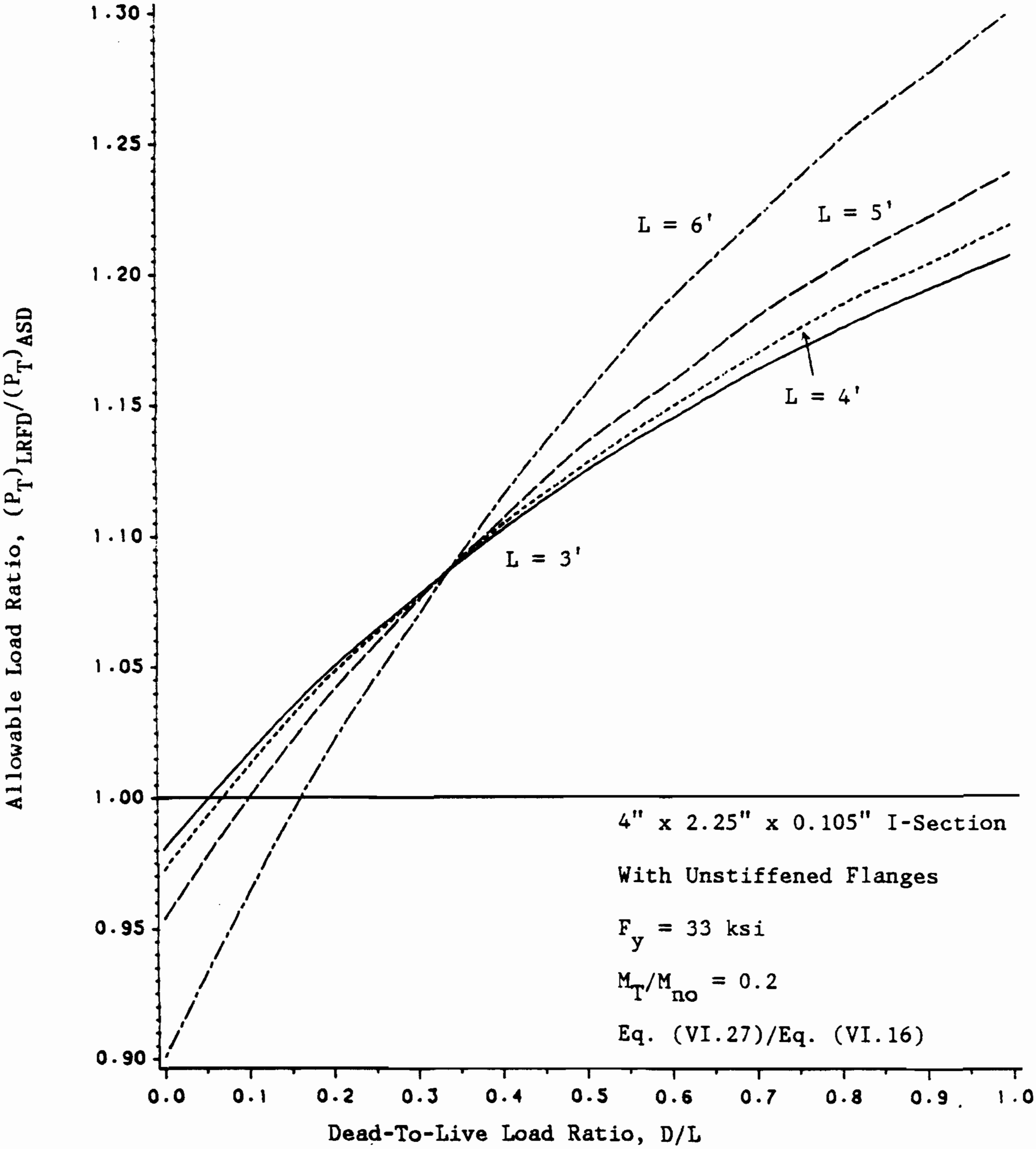

Figure 36. Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case J

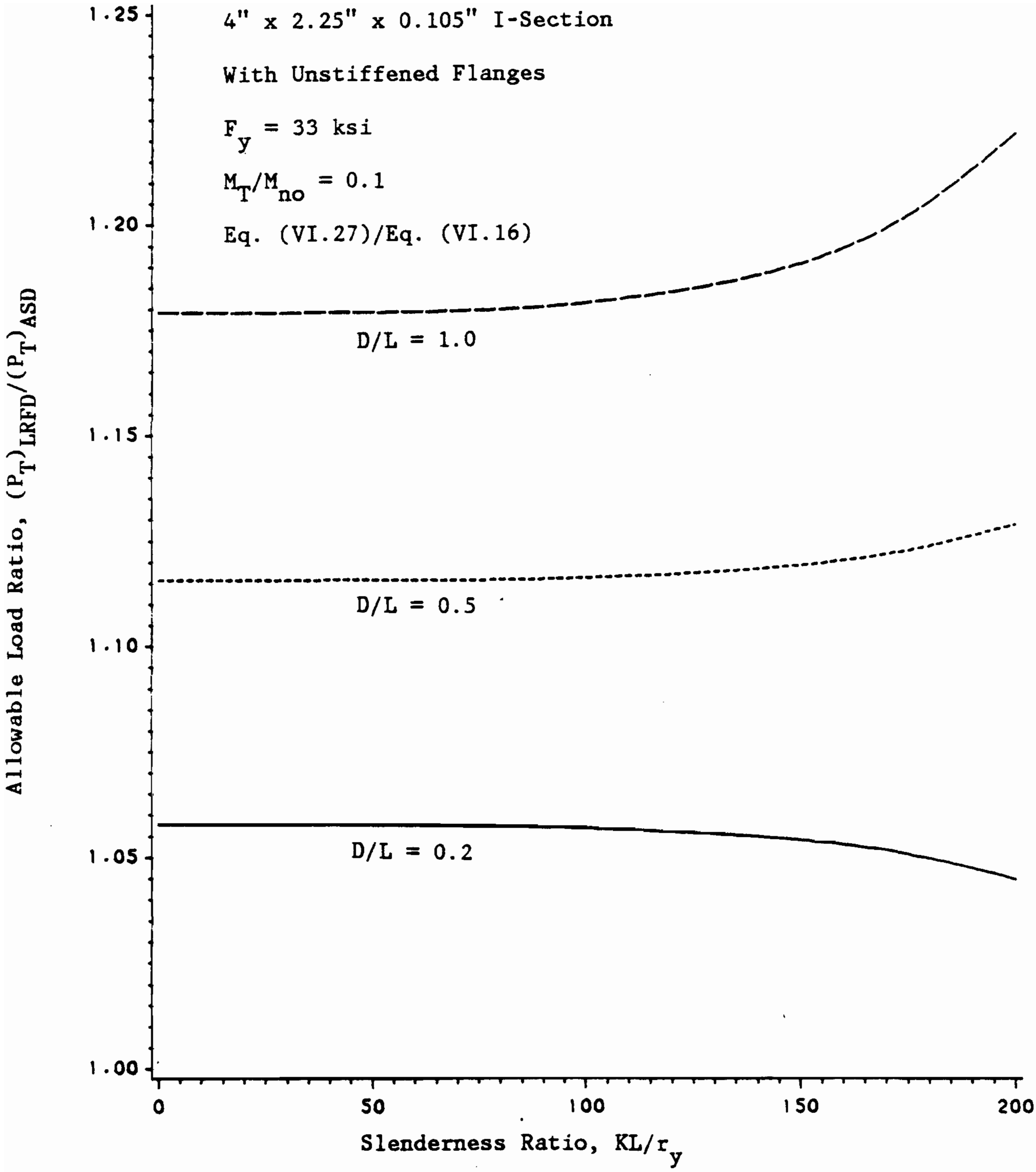

Figure 37. Allowable Load Ratio vs. Slenderness Ratio for Beam-columns - Case I

A deeper I-section (6 in. x 3 in. x 0.105 in.) with unstiffened flanges was also included in this study for a length of 5 ft. The relationship between allowable load ratio and dead-to-live load ratio for the I-section is shown in Figure 38 for various end moment ratios. This figure is based on flexural failure at the midlength of the member. The curves computed for F_y = 33 ksi are similar to the curves shown in Figure 31 obtained for an I-section with stiffened flanges. For D/L = 0.5, the allowable load ratios vary from 1.12 to 1.14 for M_T/M_{no} ratios ranging from 0.1 to 0.3.

The lines with star symbols in Figure 38 represent the allowable load ratios determined for the same I-section by using F_y = 50 ksi. It can be seen that the allowable load ratios computed for F_y = 50 ksi are lower than that computed for F_y = 33 ksi when D/L < 1/3. This effect would be negligible for beam-columns with small end moment ratios as shown in Figure 38. This comparison does not agree with the results of a study of I-sections with stiffened flanges, for which the yield point had no significant effect on the allowable load ratio for the I-section with stiffened flanges illustrated in Figure 31.

Figure 39 shows how the C_m coefficient affects the allowable load ratio for the I-section having unstiffened flanges. The curves without star symbols are plotted for C_m = 1.0. The lines with star symbols represent the allowable load ratios calculated by using C_m = 0.85. It should be noted that the relationship shown in Figure 39 is very similar to the relationship illustrated in Figure 31 obtained for an I-section with stiffened flanges. For D/L < 1/3, the allowable load ratios are larger for C_m = 0.85 as compared to the allowable load ratios computed with $C_m = 1.0$ In general, the effect of the C_m value on the allowable load ratio is more important for beam-columns with large end moment ratios.

Figure 40 shows the allowable load ratio versus dead-to-live load ratio for the same I-section used in Figures 38 and 39 but for flexural failure at the braced points. For D/L = 0.5, the LRFD criteria result in a larger allowable load than the value obtained from allowable stress design. For M_T/M_{no} ratios ranging from 0.1 to 0.3, the differences vary from 11.6% to 13.6%.

2. <u>Singly-Symmetric Sections.</u> 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:

$$M_T = e_T P_T \tag{VI.34}$$

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

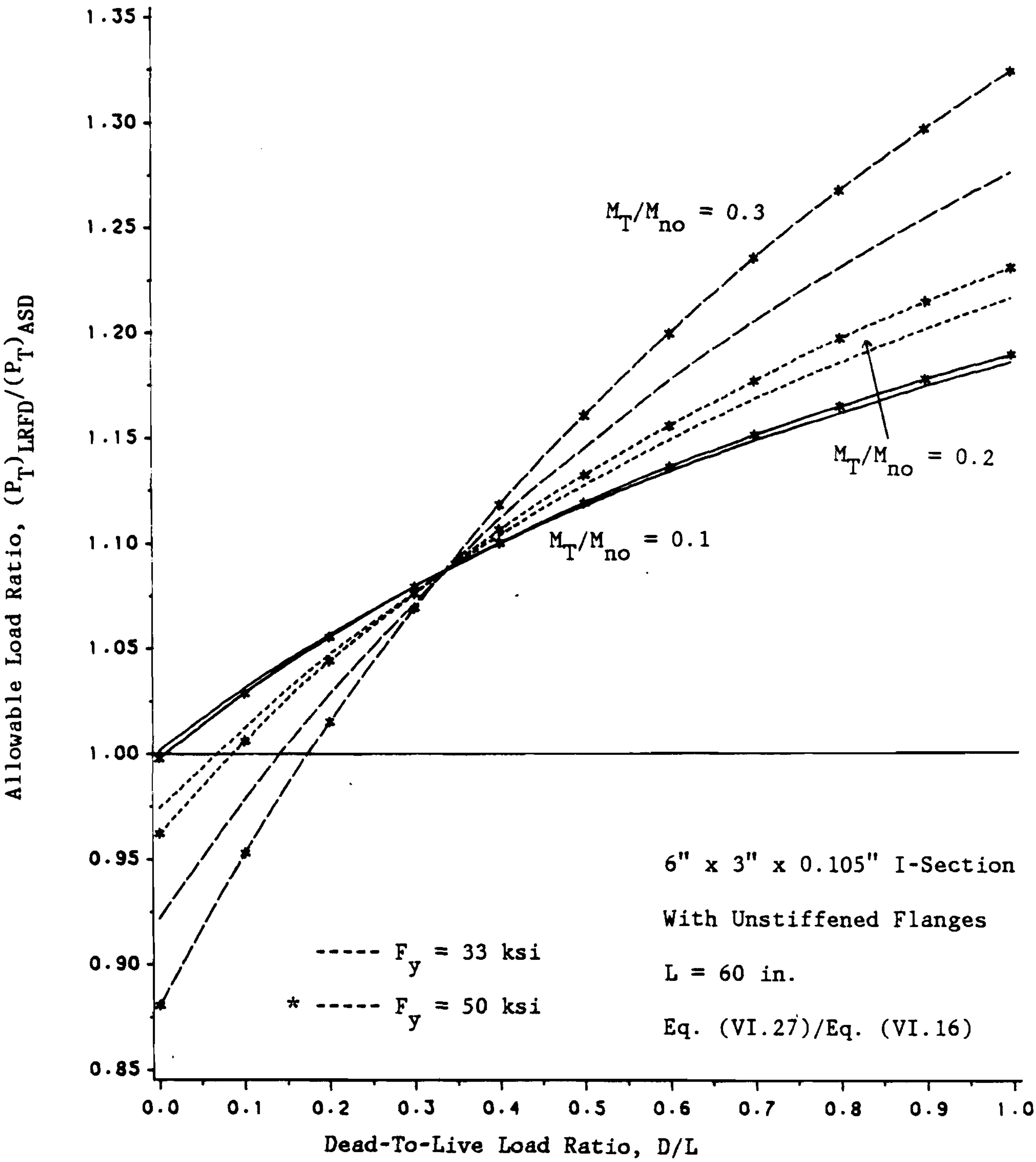

Figure 38. Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case K

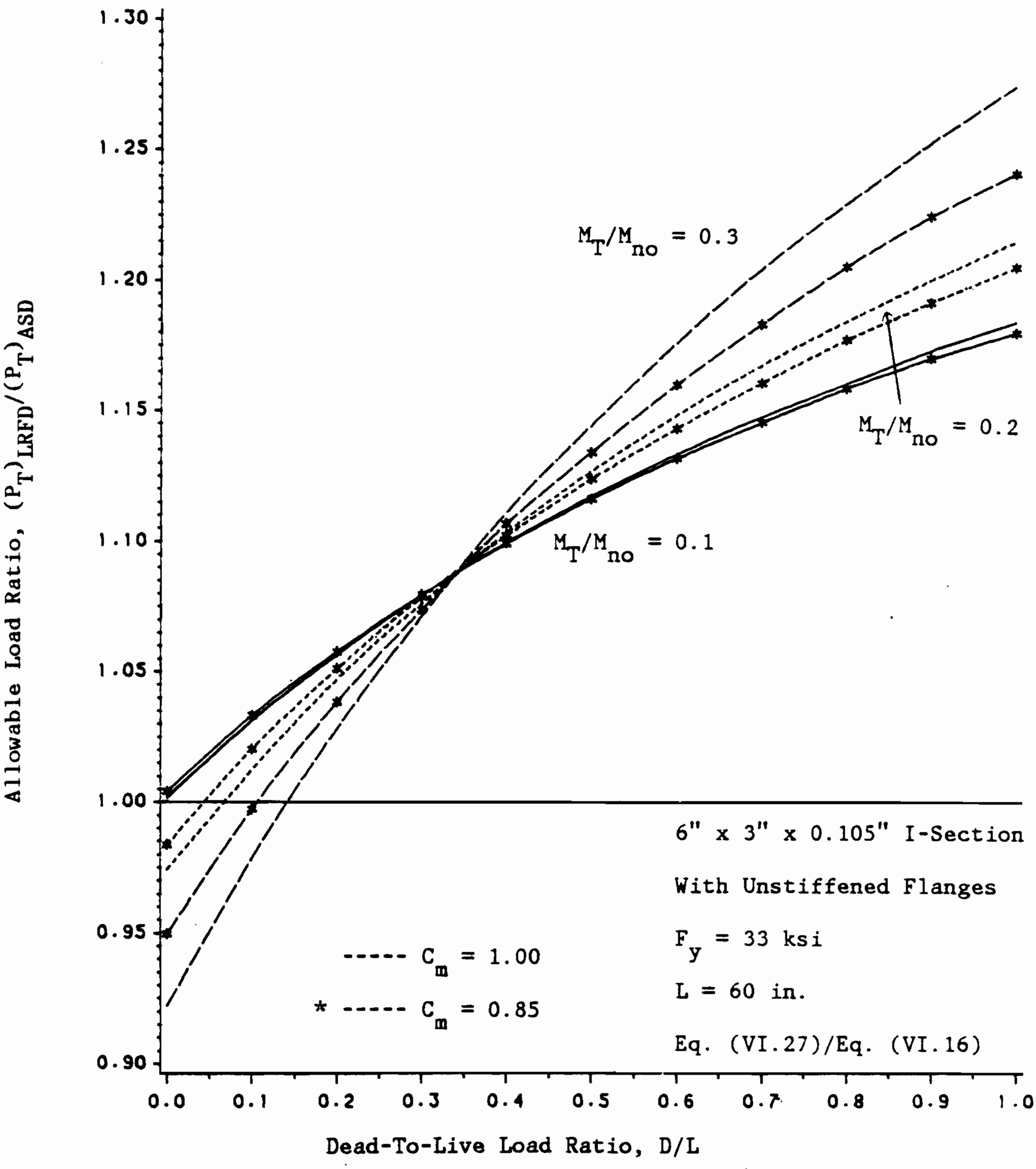

Figure 39. Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case L

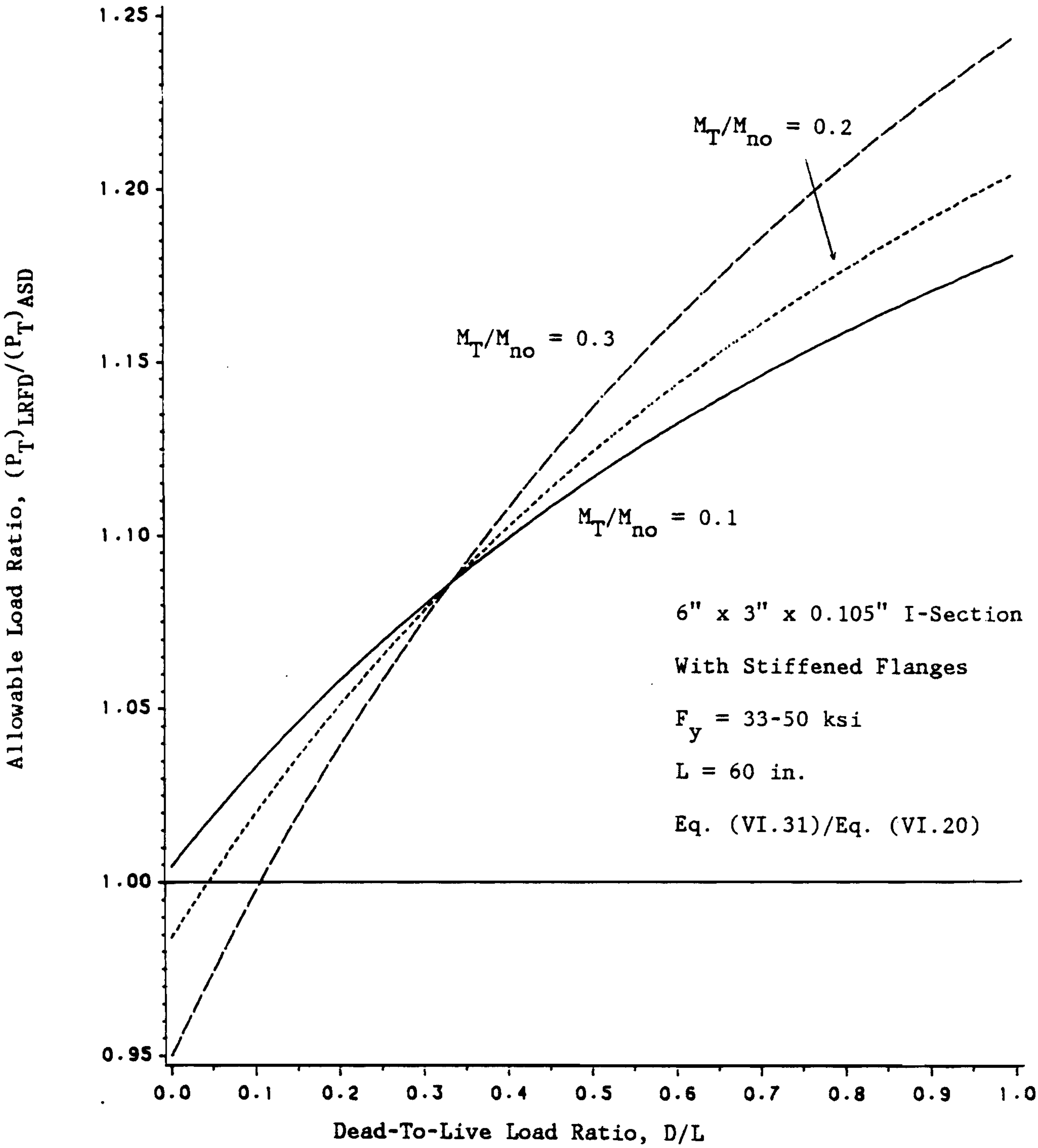

Figure 40. Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case M

Substitutions similar to the ones made to solve for the allowable loads of beam-columns with doubly-symmetric shapes in Part 1 of this section were used to solve for the allowable loads for members with singly-symmetric shapes.

Equation (VI.1) for allowable stress design is based on flexural failure at the midlength of the beam-column. Equations (VI.13) and (VI.34) were substituted into Eq. (VI.1) to obtain the following expression:

$$\frac{\Omega_c P_T}{P_n} + \frac{C_m e_T P_T}{0.6M_n(1-\Omega_c P_T/P_{cr})} = 1.0 \qquad \text{(VI.35)}$$

By solving for P_T in Eq. (VI.35), the following equation for allowable load is obtained :

$$(P_T)_{ASD1} = \frac{1.0}{\dfrac{\Omega_c}{P_n} + \dfrac{C_m e_T}{0.6M_n(1-\Omega_c P_T/P_{cr})}} \qquad \text{(VI.36)}$$

Equation (VI.36) requires a solution using iterations, since the allowable axial load is a function of the actual axial load , P_T.

Equation (VI.2) for allowable stress design is based on flexural failure at the braced points. Equations (VI.17), (VI.18), and (VI.34) were substituted into equation (VI.2) to obtain the following expression:

$$\frac{\Omega_c P_T}{P_{no}} + \frac{e_T P_T}{0.6M_{no}} = 1.0 \qquad \text{(VI.37)}$$

By solving for P_T in Eq. (VI.37), the following equation for allowable load is obtained :

$$(P_T)_{ASD2} = \frac{1.0}{\dfrac{\Omega_c}{P_{no}} + \dfrac{e_T}{0.6M_{no}}} \qquad \text{(VI.38)}$$

For allowable stress design, Eq. (VI.3) is based on flexural failure when the effect of secondary moment is neglected. Equations (VI.13) and (VI.34) were substituted into Eq. (VI.3) to obtain the following expression:

$$\frac{\Omega_c P_T}{P_n} + \frac{e_T P_T}{0.6M_n} = 1.0 \qquad \text{(VI.39)}$$

The following equation for allowable load is obtained by solving for P_T in Eq. (VI.39):

$$(P_T)_{ASD3} = \frac{1.0}{\dfrac{\Omega_c}{P_n} + \dfrac{e_T}{0.6M_n}} \qquad \text{(VI.40)}$$

For LRFD, Eq. (VI.7) is based on flexural failure at the midlength of the beam-column. Equations (VI.23), (VI.25), and (VI.34) were substituted into Eq. (VI.7) to obtain the following expression:

$$\frac{1.2D/L+1.6}{D/L+1}\left\{\frac{P_T}{\phi_c P_n}+\frac{C_m e_T P_T}{\phi M_n[1-(1.2D/L+1.6)P_T/(D/L+1)\phi_c P_E]}\right\}=1.0 \quad \text{(VI.41)}$$

By solving for P_T in Eq. (VI.41), the following equation for allowable load is obtained :

$$(P_T)_{LRFD1}=\frac{(D/L+1)/(1.2D/L+1.6)}{\dfrac{1}{\phi_c P_n}+\dfrac{C_m e_T}{\phi M_n[1-(1.2D/L+1.6)P_T/(D/L+1)\phi_c P_E]}} \quad \text{(VI.42)}$$

Equation (VI.42) requires a solution by using iterations, since the allowable axial load is also a function of the actual axial load.

Equation (VI.8) for LRFD is based on flexural failure at the braced points. The following expression was obtained by substituting Eqs. (VI.28), (VI.29), and (VI.34) into Eq. (VI.8):

$$\frac{1.2D/L+1.6}{D/L+1}\left[\frac{P_T}{\phi_c P_{no}}+\frac{e_T P_T}{\phi M_{no}}\right]=1.0 \quad \text{(VI.43)}$$

By solving for P_T in Eq. (VI.43), the following equation for allowable load is obtained :

$$(P_T)_{LRFD2} = \frac{(D/L+1)/(1.2D/L+1.6)}{\dfrac{1}{\phi_c P_{no}} + \dfrac{e_T}{\phi M_{no}}} \qquad \text{(VI.44)}$$

Equation (VI.9) for LRFD is based on flexural failure when the effect of secondary moment is neglected. Equations (VI.23) and (VI.34) were substituted into Eq. (VI.9) to obtain the following expression:

$$\frac{1.2D/L+1.6}{D/L+1}\left[\frac{P_T}{\phi_c P_n} + \frac{e_T P_T}{\phi M_n}\right] = 1.0 \qquad \text{(VI.45)}$$

The following equation for allowable load was obtained by solving for P_T in Eq. (VI.45):

$$(P_T)_{LRFD3} = \frac{(D/L+1)/(1.2D/L+1.6)}{\dfrac{1}{\phi_c P_n} + \dfrac{e_T}{\phi M_n}} \qquad \text{(VI.46)}$$

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 41 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 41 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 42 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 Figure 41 and 42. 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 43 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 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 44. 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 41 for the 4 in. deep channel.

The top line in Figure 44 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 44.

Figure 45 shows the allowable load ratio versus dead-to-live load ratio for the channel used in Figure 44. 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.

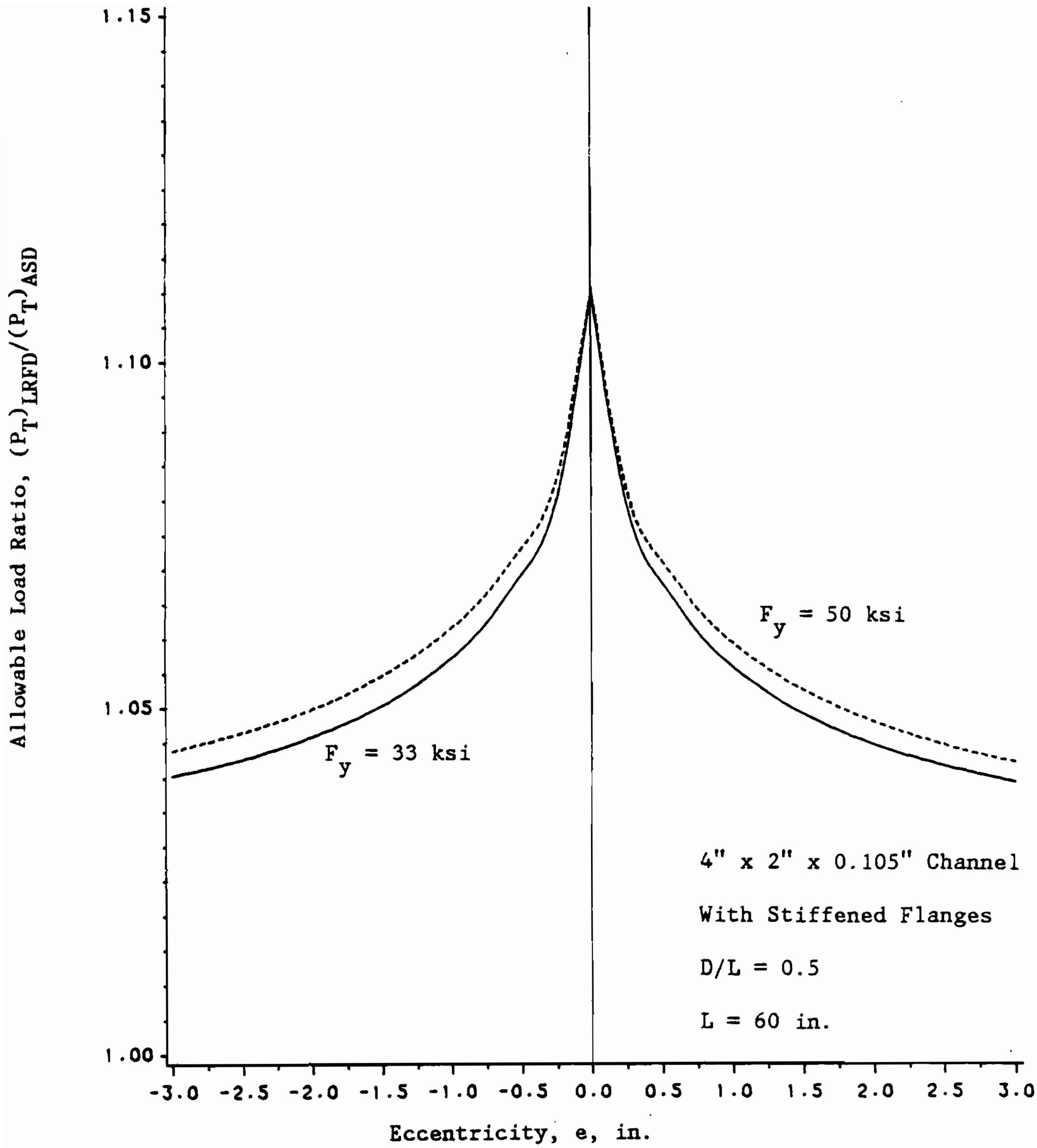

Figure 41. Allowable Load Ratio vs. Eccentricity for Beam-Columns - Case 1

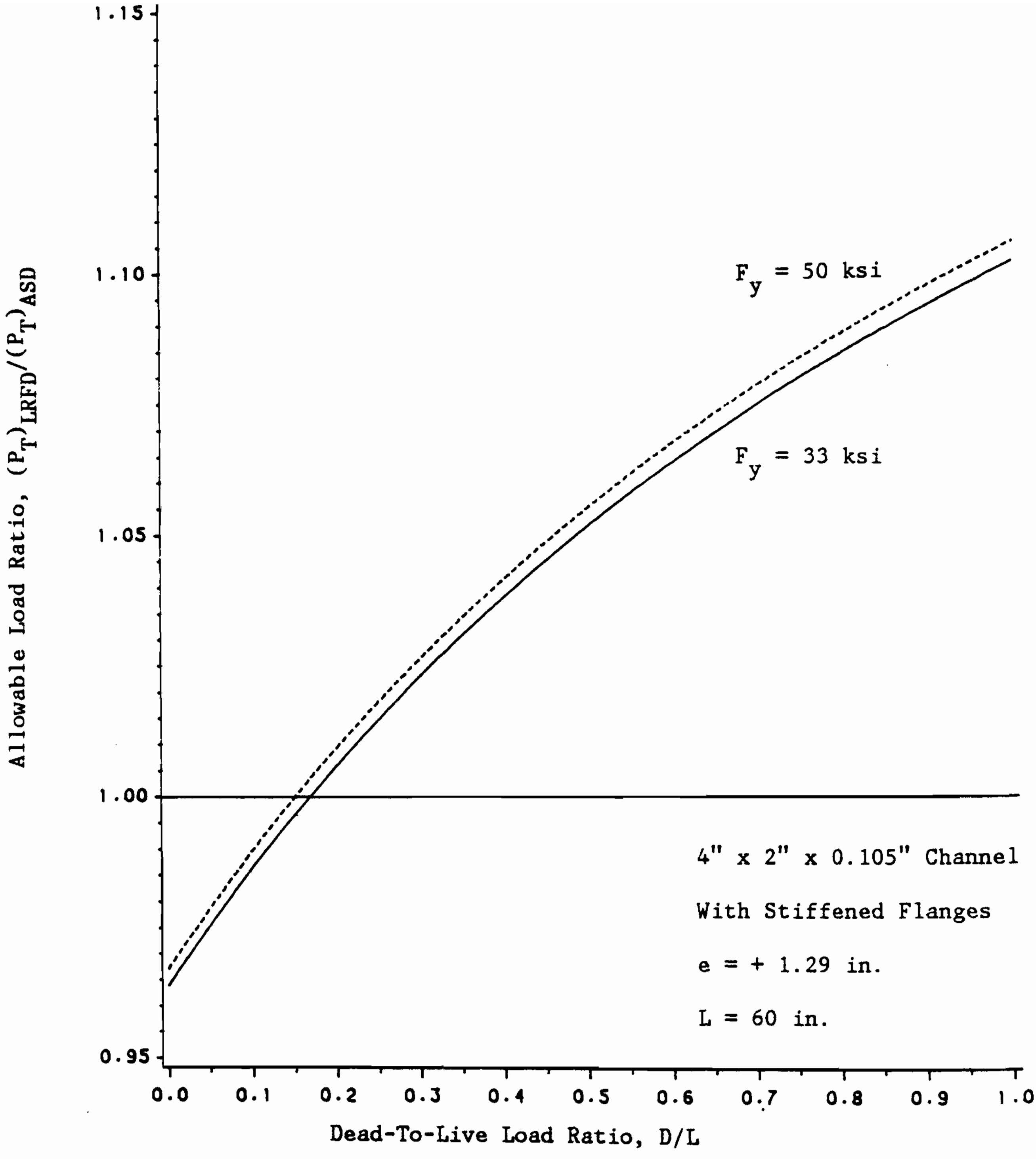

Figure 42. Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case 1

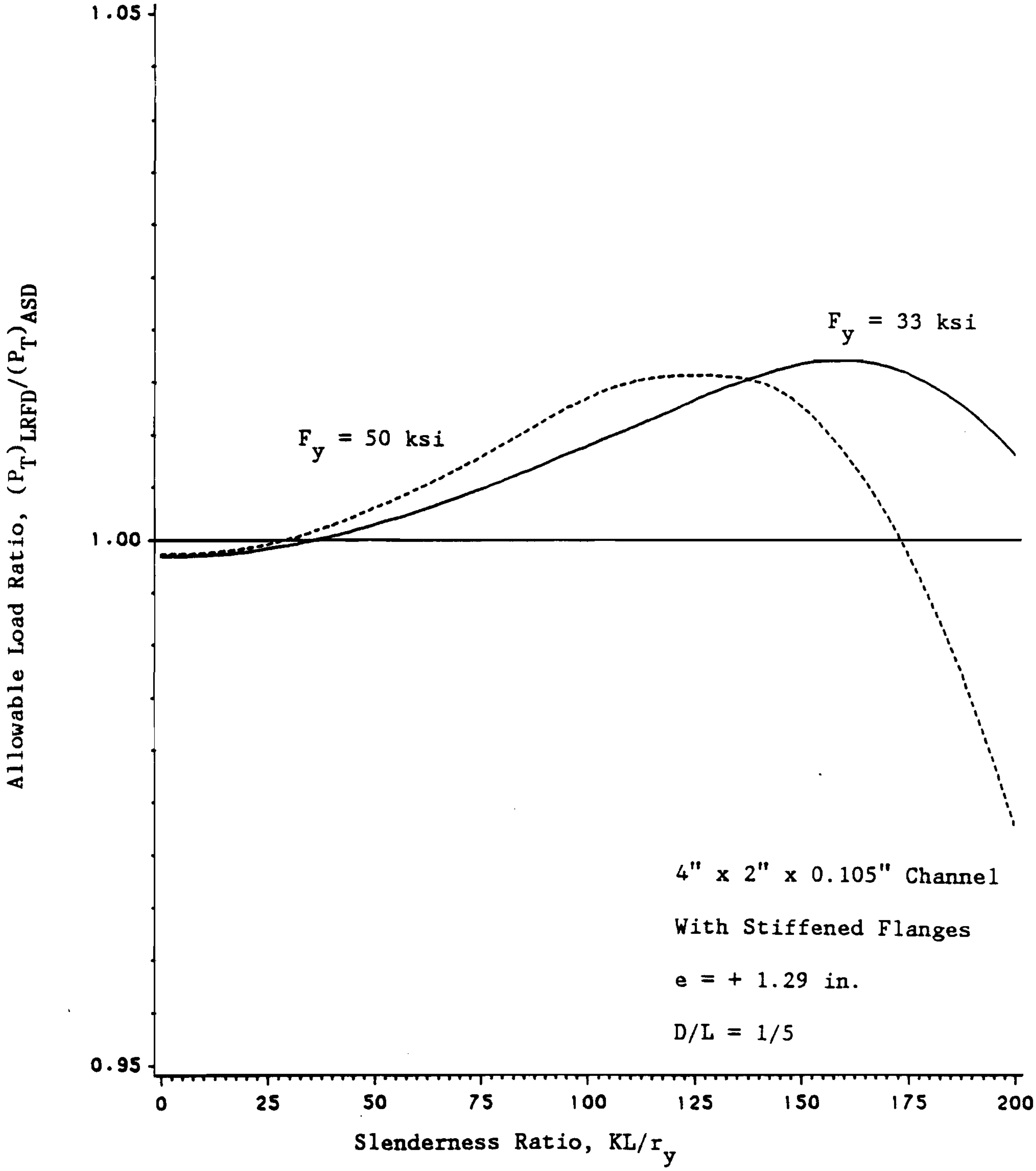

Figure 43. Allowable Load Ratio vs. Slenderness Ratio for Beam-Columns - Case 1

The relationship between allowable load ratio and dead-to-live load ratio for the 6 in. deep channel (6 in. x 2.5 in. x 0.105 in.) is shown in Figure 46 for various lengths and eccentricities. The curves without star symbols represent the values of allowable load ratios for e = + 1.73 in. and effective lengths of 3 and 11 ft. It should be noted that the effective length has a small effect on the allowable load ratio.

The curves with star symbols in Figure 46 represent the allowable load ratios determined for the same channel section with e = - 1.73 in. and effective lengths of 3 and 11 ft. It can be seen that for larger effective length, the allowable load ratios determined for e = - 1.73 in. are larger than those determined for e = + 1.73 in.. For smaller effective length, the allowable load ratios determined for e = - 1.73 in. are slightly smaller than those determined for e = + 1.73 in.

Channels with unstiffened flanges were studied in a similar manner. Figure 47 shows the allowable load ratio versus eccentricity for a channel (4 in. x 1.125 in. x 0.105 in.) with unstiffened flanges and an effective length of 5 ft. The curves in the figure are allowable load ratios computed for yield points of 33 and 50 ksi, respectively. These curves indicate similar relationships as compared with the curves in Figure 41 obtained from a 4 in. deep channel with stiffened flanges. As shown in Figure 47, allowable load ratios computed for F_y = 33 ksi are larger than those computed for F_y = 50 ksi.

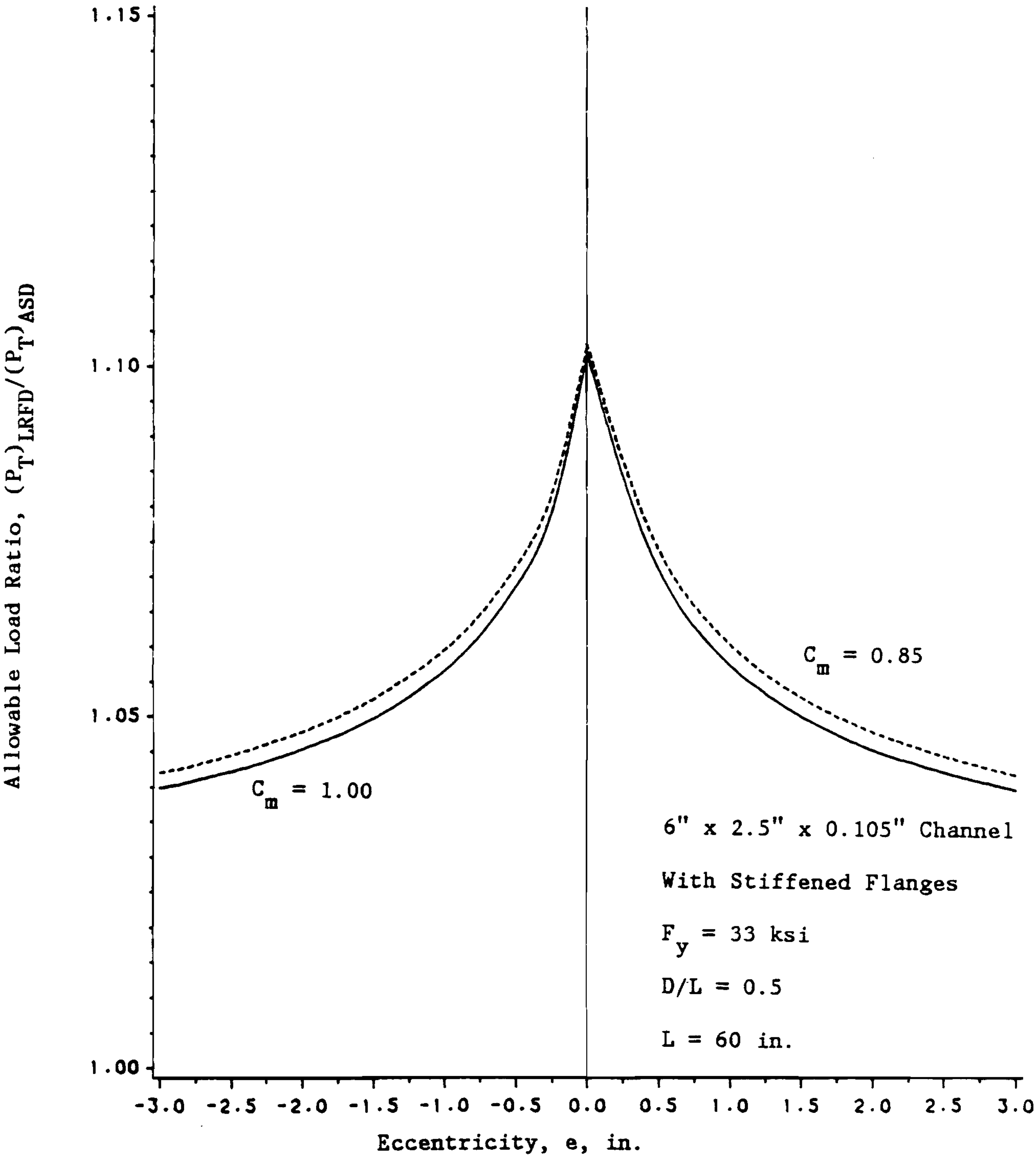

Figure 44. Allowable Load Ratio vs. Eccentricity for Beam-Columns - Case 2

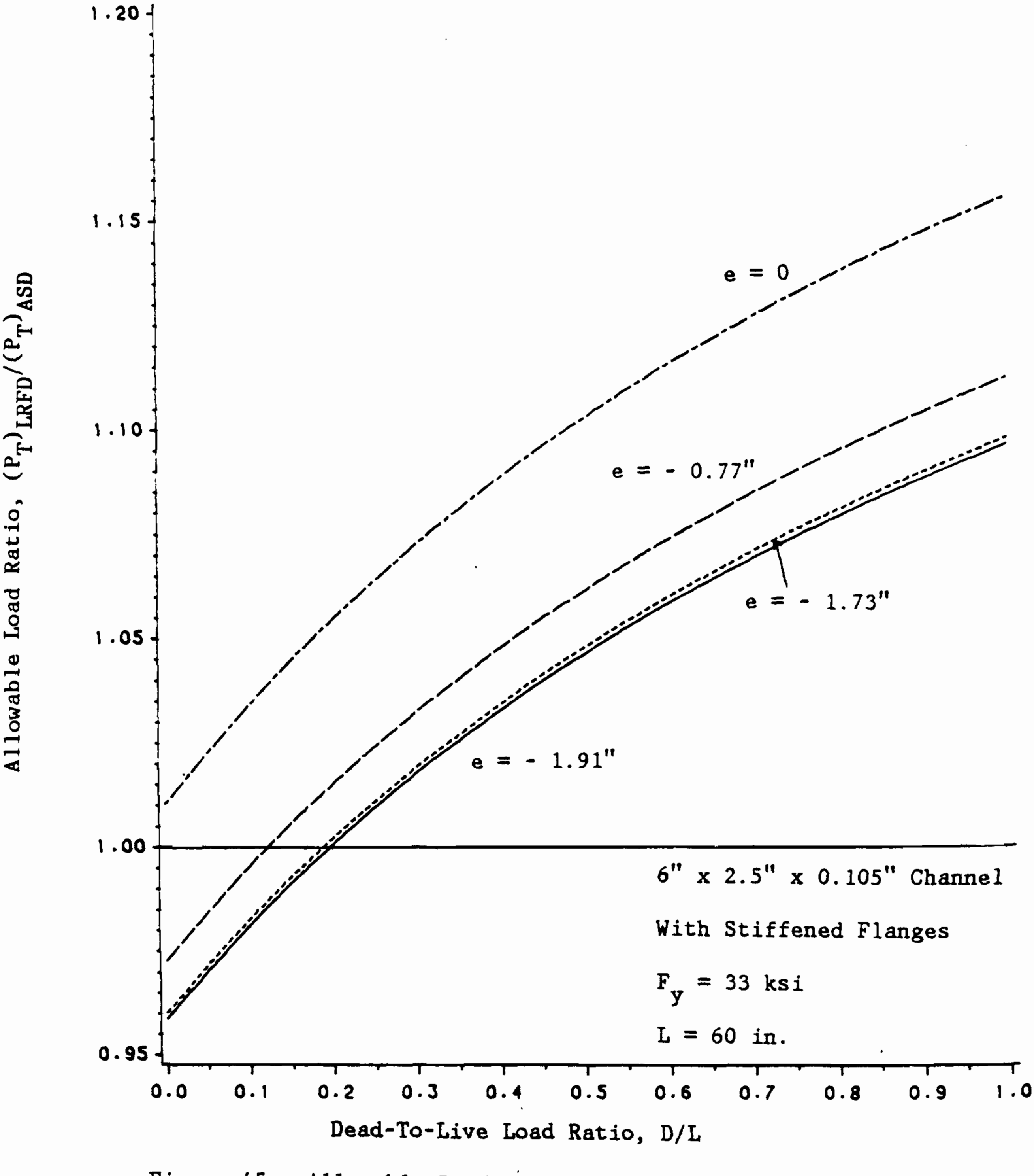

Figure 45. Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case 2

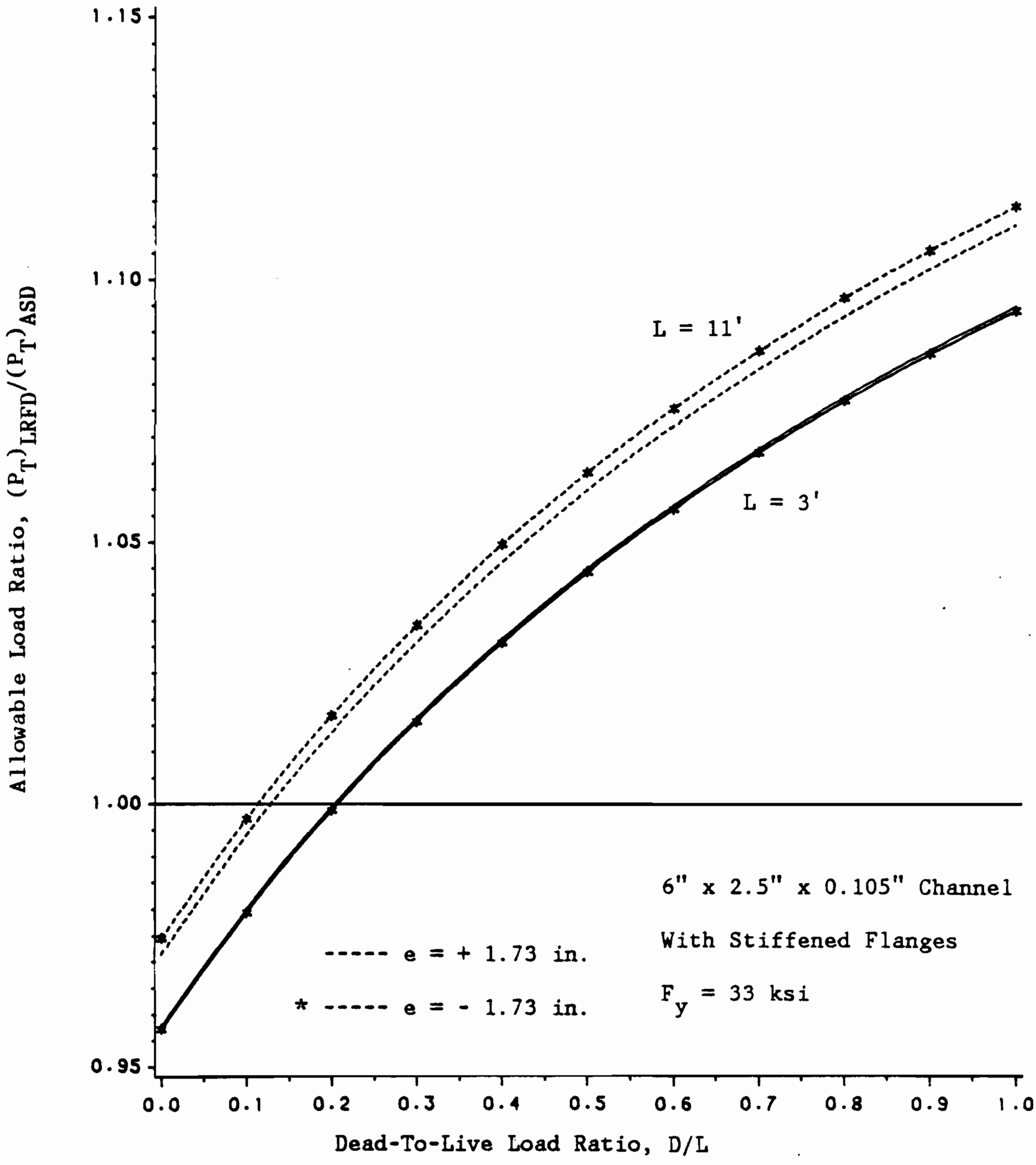

Figure 46. Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case 3

The curves in Figure 48 are for the same channel used in Figure 47 with an effective length of 4 ft. The yield points of steel vary from 33 to 50 ksi. This figure shows that the yield point has no effect and the sign of eccentricity has negligible effect on the allowable load ratio for various D/L ratios with L = 4 ft.

Figure 49 shows the allowable load ratio versus slenderness ratio, KL/r_y, for the same channel used in Figures 47 and 48 for D/L = 1/5 and e = + 1.20 in. The curves without star symbol represent the values of allowable load ratio for e = + 1.20 in; the curves with star symbols represent the values of allowable load ratio for e = - 1.20 in. The sign of eccentricity has negligible effect. The curve computed for yield point of 33 ksi indicates that the allowable load ratio increases slightly with increasing slenderness ratio up to $KL/r_y = 170$. For $KL/r_y > 170$, the allowable load ratio decreases as the slenderness ratio increases. The same relationships hold for $F_y = 50$ ksi but with the dividing point of $KL/r_y = 130$. The curves also show that for $KL/r_y = 150.47$ (L = 4 ft), the allowable load ratios for $F_y = 33$ and 50 ksi are the same. When $KL/r_y < 150.47$, the allowable load ratios computed for $F_y = 50$ ksi are larger than those computed for $F_y = 33$ ksi. For $KL/r_y > 150.47$, the allowable load ratio computed for $F_y =$ 33 ksi are larger than those computed for $F_y = 50$ ksi.

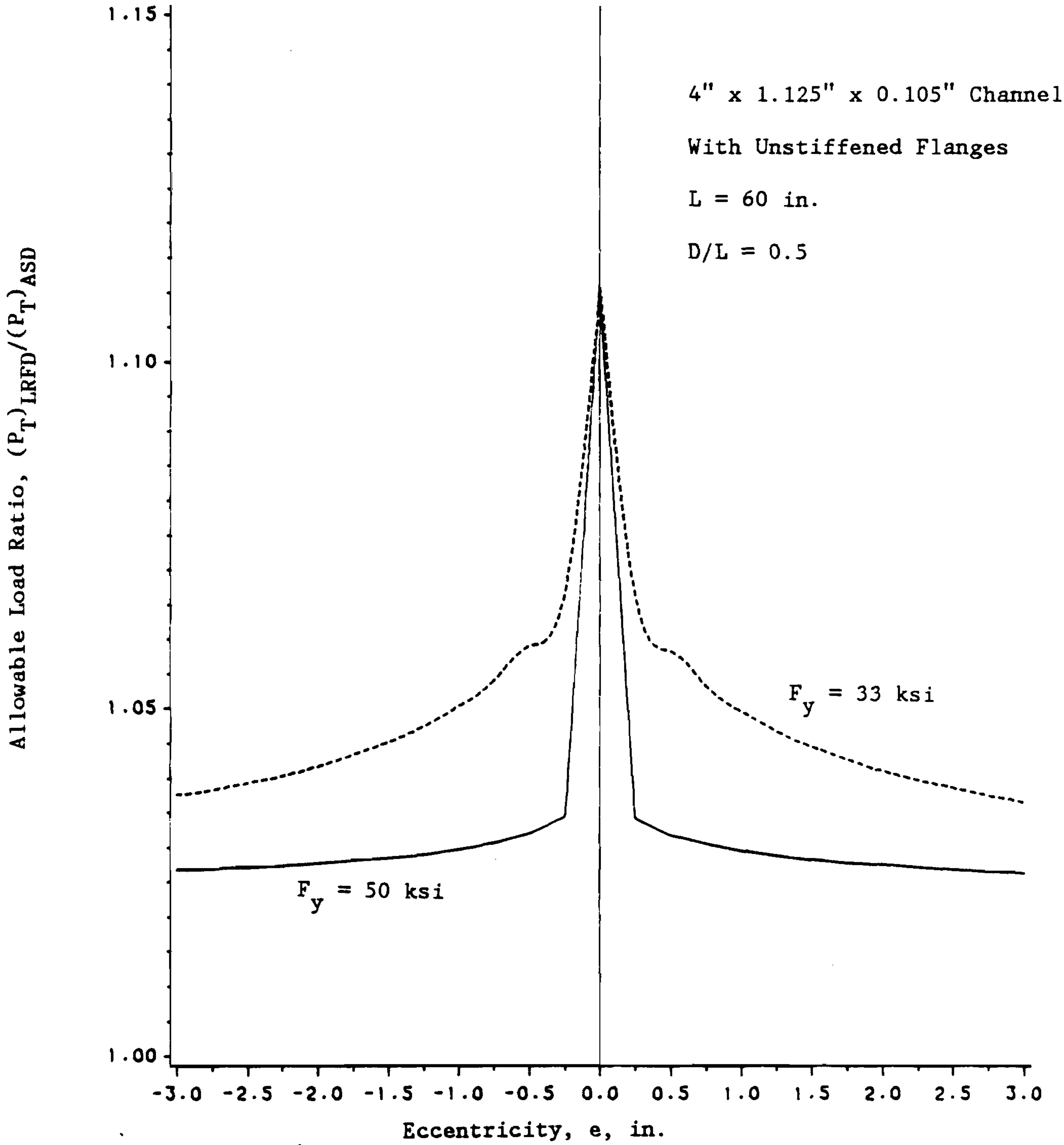

Figure 47. Allowable Load Ratio vs. Eccentricity for Beam-Columns - Case 4

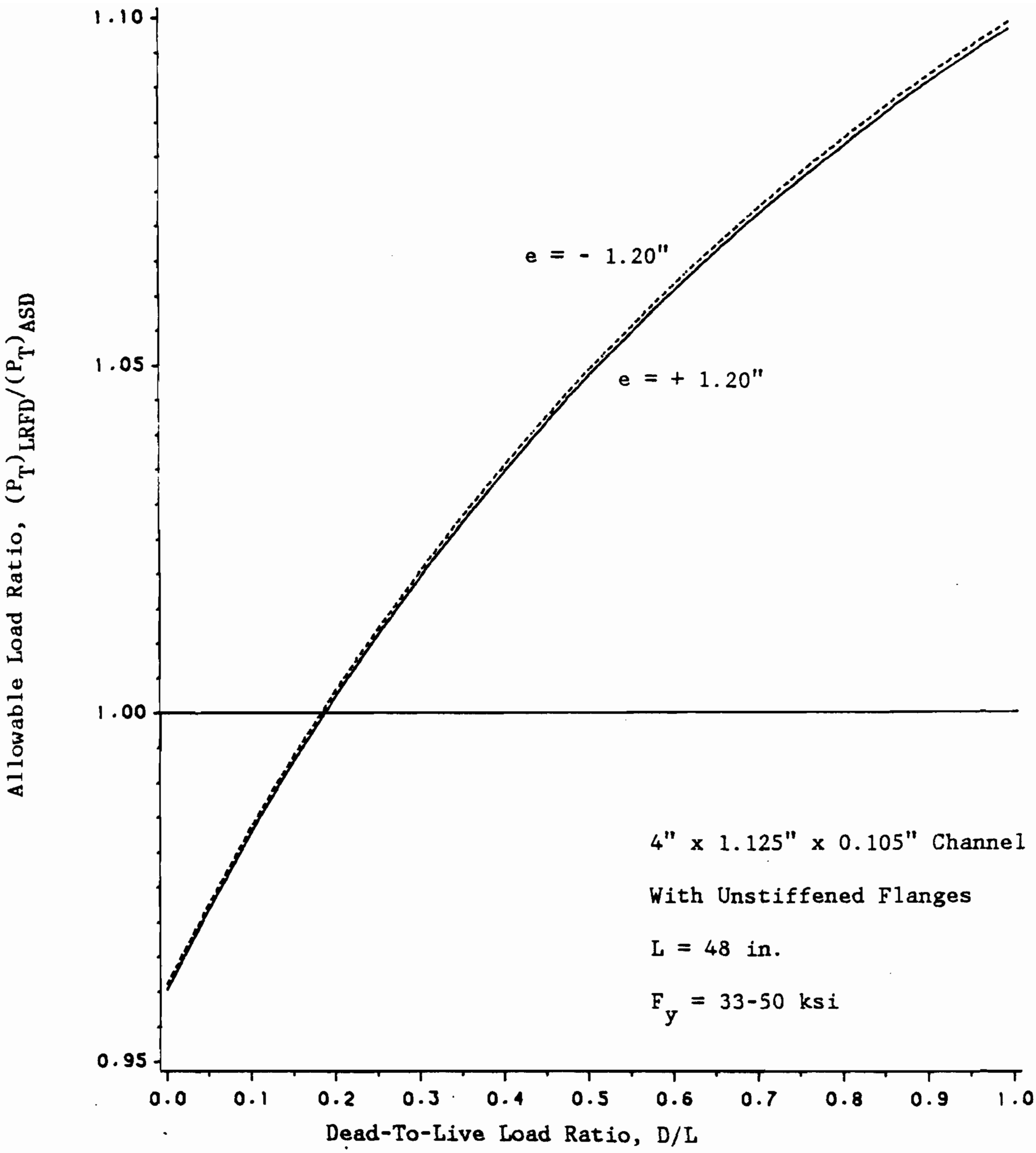

Figure 48. Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case 4

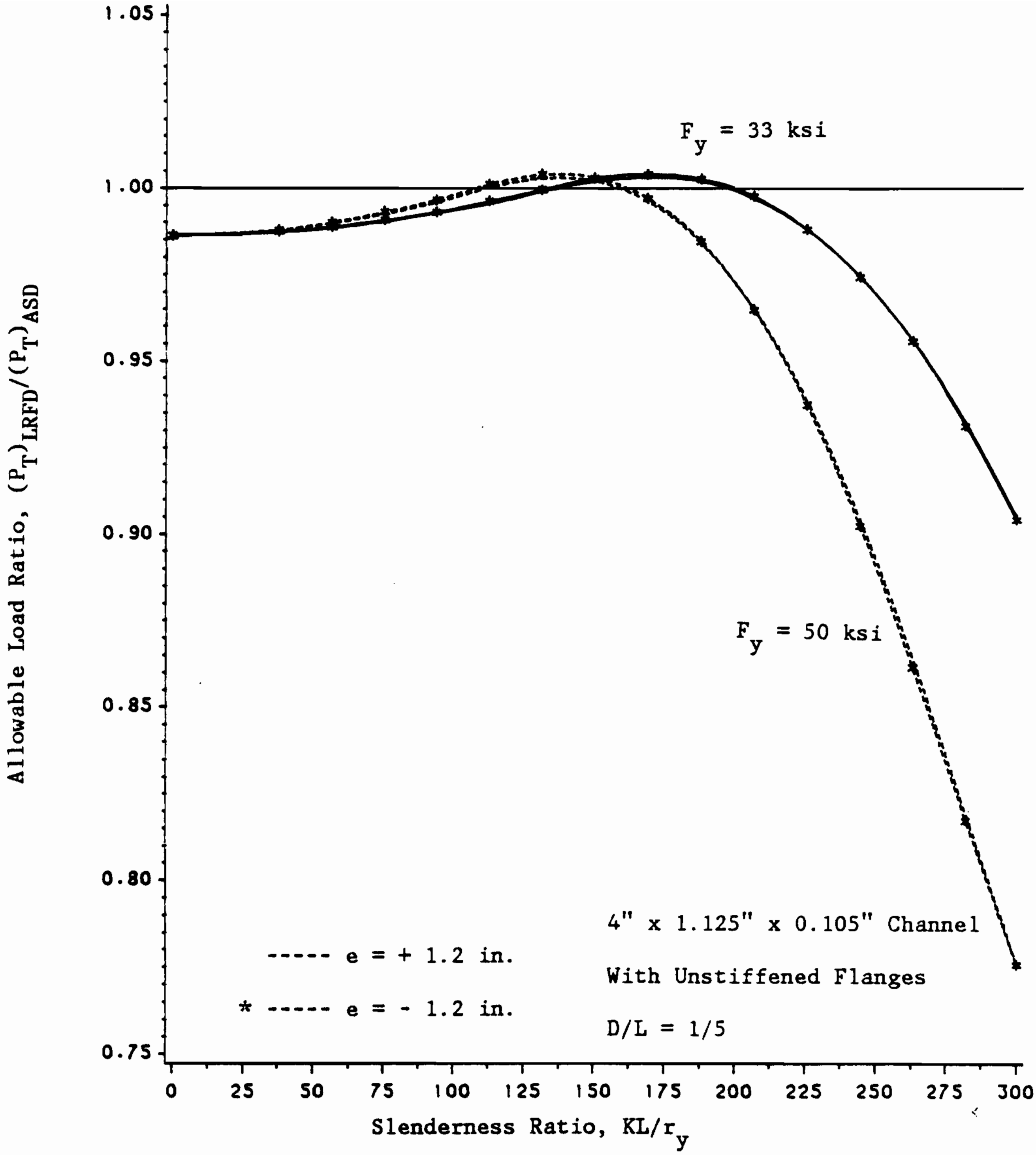

Figure 49. Allowable Load Ratio vs. Slenderness Ratio for Beam-Columns - Case 4

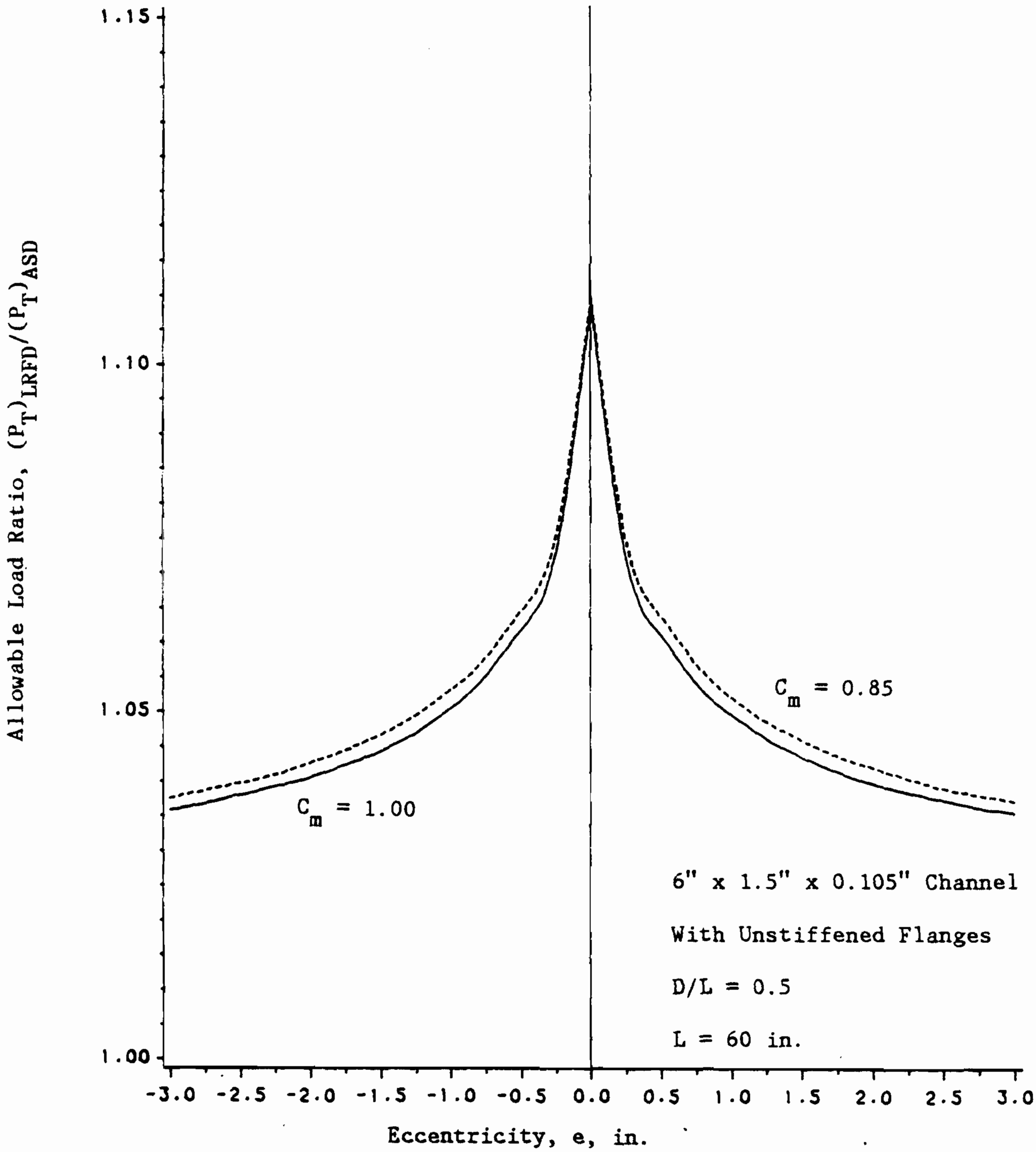

Figure 50. Allowable Load Ratio vs. Eccentricity for Beam-Columns - Case 5

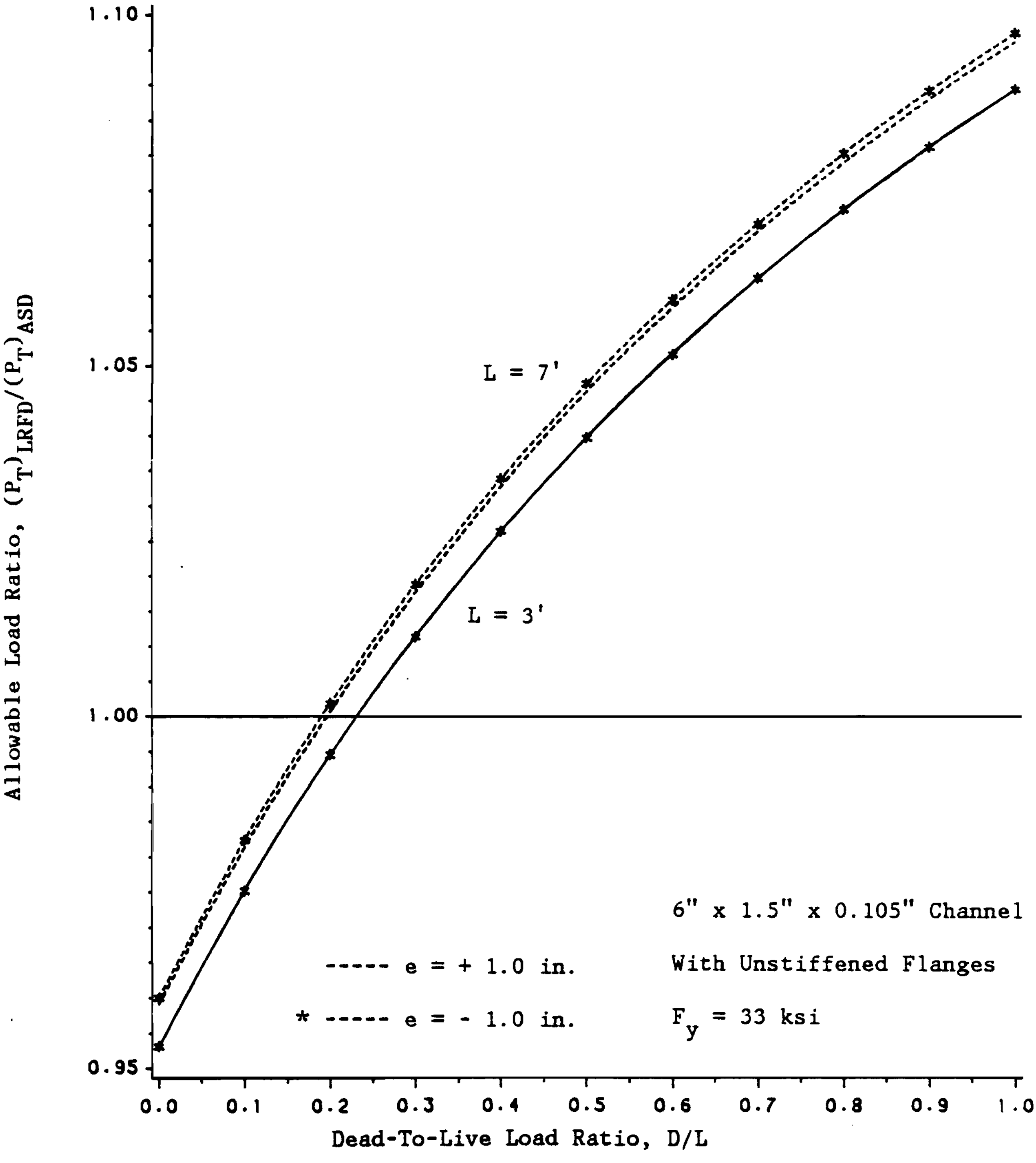

Figure 51. Allowable Load Ratio vs. D/L Ratio for Beam-Columns - Case 5

A deeper channel (6 in. x 1.5 in. x 0.105 in.) with unstiffened flanges was also studied. Figure 50 shows the allowable load ratio versus eccentricity for the 5 ft long channel with D/L = 0.5 and F_y = 33 ksi. The curves shown in the figure are obtained for C_m values of 1.0 and 0.85. It is similar in shape and magnitude to the allowable load ratio curve shown in Figure 47 for a 4 in. deep channel with unstiffened flanges and F_y = 33 ksi. Figure 50 shows that C_m value has negligible effect on the allowable load ratio. Also as shown in Figures 47 and 50, small eccentricities will result in relatively high allowable load ratios.

The relationship between allowable load ratio and dead-to-live load ratio for the channel used in Figure 50 is shown in Figure 51 for various lengths. The curves without star symbol represent the values of allowable load ratio for e = + 1.00 in; the curves with star symbols represent the values of allowable load ratio for e = - 1.00 in. The effective lengths used in this figure are 3 and 7 ft. This figure is similar to Figure 46 which was obtained from a channel of equal depth but with stiffened flanges. As shown in this figure, the effective length has a small effect on the allowable load ratio. It also shows that the sign of eccentricity has no effect for L = 3 ft and has negligible effect for L = 7 ft on the allowable load ratio.

VII. CONNECTIONS

A. GENERAL

Connections are required for joining individual structural members together and are used to fabricate structural members from sheet steel or structural components. The AISI Specification and the Specification for Load and Resistance Factor Design include requirements for welded and bolted connections which are frequently used in cold-formed steel construction. All connections should be designed to transmit the maximum load with proper regard for eccentricity.

B. WELDED CONNECTIONS

Welds are classified as fusion welds and resistance welds. Weld shearing and plate tearing are the common failure modes for welded connections.

1. Allowable Stress Design. Welded connections shall be designed to transmit the maximum load in the connected member. Proper regard shall be given to eccentricity.

a. Arc Welds. Arc welds are fusion welds produced by burning the metal to a molten state at the surface to be joined without the application of mechanical pressure or blows[45]. Pekoz and McGuire[46] studied the welding of sheet steel and provided most of the statistical data used for the development of the AISI design provisions for allowable stress design and the LRFD criteria for arc welds. According to Section E2 of the AISI Specification, the load on each arc weld shall not exceed P_a calculated as follows:

$$P_a = P_n/\Omega_w \qquad \text{(VII.1)}$$

where

Ω_w = Factor of safety for arc welded connections

= 2.50

P_n = Nominal strength of welds determined according to the following formulas.

i. <u>Arc Spot Welds.</u> Arc spot welds are produced by burning a hole in the top sheet and filling it with weld metal which fuses it to the bottom sheet or structural member. They are sometimes referred to as puddle welds. Arc spot welds permitted by the AISI Specification are for welding sheet steel to thicker supporting members in the flat position. Arc spot welds (puddle welds) shall not be made on steel where the thinnest connected part is over 0.15 inch thick, nor through a combination of steel sheets having a total thickness over 0.15 inch.

Weld washers shall be used when the thickness of the sheet is less than 0.028 inch. Weld washers shall have a thickness between 0.05 and 0.08 inch with a minimum prepunched hole of 3/8-inch diameter.

Arc spot welds shall be specified by minimum effective diameter of fused area, d_e. Minimum allowable effective diameter is 3/8 inch.

The nominal shear load, P_n, on each arc spot weld between sheet or sheets and supporting member shall not exceed the smaller of either

$$P_n = 0.625 d_e^2 F_{xx}; \text{ or} \qquad \text{(VII.2)}$$

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

$$P_n = 2.20 t d_a F_u \tag{VII.3}$$

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

$$P_n = 0.280[1+(5.59t\sqrt{E})/(d_a\sqrt{F_u})] t d_a F_u \tag{VII.4}$$

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

$$P_n = 1.40 t d_a F_u \tag{VII.5}$$

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

$$d_e = 0.7d-1.5t \text{ but } \leq 0.55d \tag{VII.6}$$

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 = Specified minimum tensile strength of steel

ii. Arc Seam Welds. Arc seam welds are produced in the same manner as arc spot welds except that a seam is formed. Arc seam welds covered by the AISI Specification apply only to the following joints:

(a) Sheet to thicker supporting member in the flat position.

(b) Sheet to sheet in the horizontal or flat position.

According to Section E2.3 of the AISI Specification, the shear load, P_n, on each arc seam weld shall not exceed either

$$P_n = (d_e^2/4+Ld_e/3)(2.5F_{xx}); \text{ or} \tag{VII.7}$$

$$P_n = 2.5tF_u(0.25L+0.96d_a) \quad \text{(VII.8)}$$

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

$$d_a = (d-t) \text{ for a single sheet, and} \quad \text{(VII.9)}$$

$$(d-2t) \text{ for a double sheet} \quad \text{(VII.10)}$$

d_e = Effective width of arc seam weld at fused surfaces

$$d_e = 0.7d-1.5t \quad \text{(VII.11)}$$

iii. Fillet Welds. Fillet welds are used to connect lap joints and T-joints. Fillet welds covered by the AISI Specification apply to the welding of joints in any position, either

(a) Sheet to sheet, or

(b) Sheet to thicker steel member.

According to Section E2.4 of the AISI Specification, the shear load, P_n, on a fillet weld in lap and T-joints shall not exceed the following:

For longitudinal loading:

For $L/t < 25$:

$$P_n = (1-0.01L/t)tLF_u \quad \text{(VII.12)}$$

For $L/t \geq 25$:

$$P_n = 0.75tLF_u \quad \text{(VII.13)}$$

For transverse loading:

$$P_n = tLF_u \quad \text{(VII.14)}$$

where

t = Least value of t_1 or t_2

In addition, for $t > 0.150$ inch the allowable load for fillet weld in lap and T-joints shall not exceed:

$$P_n = 0.75 t_w L F_{xx} \qquad \text{(VII.15)}$$

where

L = Length of fillet weld

t_w = Effective throat = $0.707w_1$ or $0.707w_2$, whichever is smaller. A larger effective throat may be taken if it can be shown by measurement that a given welding procedure will consistently give a larger value providing the particular welding procedure used for making the welds that are measured is followed.

w_1 and w_2 = leg on weld

iv. <u>Flare Groove Welds.</u> Flare groove welds are used in cold-formed steel construction to join rolled corners to sheets and to join two rolled corners. Flare groove welds covered by the AISI Specification apply to welding of joints in any position, either:

(a) Sheet to sheet for flare-V groove welds, or

(b) Sheet to sheet for flare-bevel groove welds, or

(c) Sheet to thicker steel member for flare-bevel groove welds.

According to Section E2.5 of the AISI Specification, the shear load, P_n, on a weld shall be governed by the thickness, t, of the sheet steel adjacent to the weld. The load shall not exceed:

For flare-bevel groove welds, transverse loading:

$$P_n = 0.833 t L F_u \qquad \text{(VII.16)}$$

For flare groove welds, longitudinal loading:

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

$$P_n = 0.75tLF_u \quad \text{(VII.17)}$$

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

$$P_n = 1.50tLF_u \quad \text{(VII.18)}$$

In addition, if $t > 0.15$ inch, then:

$$P_n = 0.75t_wLF_{xx} \quad \text{(VII.19)}$$

b. Resistance Welds. Resistance welding is a group of welding processes wherein coalescence is produced by the heat obtained from resistance to electric current through the work parts held together under presure by electrodes[45]. They are mostly used for shop welding in cold-formed steel fabrication. According to Section E2.6 of the AISI Specification, in sheets joined by spot welding the allowable shear per spot, P_a, shall be as follows (the safety factor is included in Table VII.1):

TABLE VII.1
Allowable Shear per Spot for Resistance Welds

Thickness of Thinnest Outside Sheet, in.	Allowable Shear Strength per Spot, kips	Thickness of Thinnest Outside Sheet, in.	Allowable Shear Strength per Spot, kips
0.010	0.050	0.080	1.330
0.020	0.175	0.094	1.725
0.030	0.400	0.109	2.395
0.040	0.570	0.125	2.880
0.050	0.660	0.188	4.000
0.060	0.910	0.250	6.000

2. LRFD Criteria

a. Arc Welds. According to Section E2 of the LRFD Specification, the force on each weld computed on the basis of factored loads shall not exceed the factored nominal strength, ϕP_n,

where

ϕ = Resistance factor for arc welded connections

P_n = Nominal strength of welds

i. Arc Spot Welds. Arc spot welds permitted by the LRFD Specification are for welding sheet steel to thicker supporting members in the flat position. Arc spot welds (puddle welds) shall not be made on steel where the thinnest connected part is over 0.15 inch thick, nor through a combination of steel sheets having a total thickness over 0.15 inch.

Weld washers shall be used when the thickness of the sheet is less than 0.028 inch. Weld washers shall have a thickness between 0.05 and 0.08 inch with a minimum prepunched hole of 3/8-inch diameter.

Arc spot welds shall be specified by minimum effective diameter of fused area, d_e. Minimum allowable effective diameter is 3/8 inch.

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

(a) ϕ = 0.60

$$P_n = 0.589 d_e^2 F_{xx}; \text{ or} \qquad \text{(VII.20)}$$

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

ϕ = 0.60

$$P_n = 2.20 t d_a F_u \qquad \text{(VII.21)}$$

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

$\phi = 0.50$

$$P_n = 0.280\left[1+5.59\sqrt{E/F_u}/(d_a/t)\right] t d_a F_u \quad \text{(VII.22)}$$

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

$\phi = 0.50$

$$P_n = 1.40 t d_a F_u \quad \text{(VII.23)}$$

where

ϕ = Resistance factor for welded connections

P_n = Nominal ultimate shear strength of an arc spot weld

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

$$d_e = 0.7d-1.5t \text{ but } \leq 0.55d \quad \text{(VII.24)}$$

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

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

ii. Arc Seam Welds. Arc seam welds covered by the LRFD Specification apply only to the following joints:

(a) Sheet to thicker supporting member in the flat position.

(b) Sheet to sheet in the horizontal or flat position.

The factored nominal shear strength, ϕP_n, of arc seam welds shall be determined by using the smaller of either

(a) $\phi = 0.60$

$$P_n = (\pi d_e^2/4+Ld_e)(0.75F_{xx}); \text{ or} \tag{VII.25}$$

(b) $\phi = 0.60$

$$P_n = 2.5tF_u(0.25L+0.96d_a) \tag{VII.26}$$

where

ϕ = Resistance factor for welded connections

P_n = Nominal ultimate shear strength of an arc seam weld

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

iii. <u>Fillet Welds.</u> Fillet welds covered by the LRFD Specification apply to the welding of joints in any position, either

(a) Sheet to sheet, or

(b) Sheet to thicker steel member.

The factored nominal shear strength, ϕP_n, of a fillet weld shall be determined as follows:

(a)For longitudinal loading:

For $L/t < 25$:

$\phi = 0.60$

$$P_n = (1-0.01L/t)tLF_u \tag{VII.27}$$

For $L/t \geq 25$:

$\phi = 0.55$

$$P_n = 0.75tLF_u \tag{VII.28}$$

(b)For transverse loading:

$\phi = 0.60$

$$P_n = tLF_u \tag{VII.29}$$

where

t = Least value of t_1 or t_2

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

ϕ = 0.60

$$P_n = 0.75 t_w L F_{xx} \tag{VII.30}$$

where

ϕ = Resistance factor for welded connections

P_n = Nominal ultimate strength of a fillet weld

L = Length of fillet weld

t_w = Effective throat = $0.707w_1$ or $0.707w_2$, whichever is smaller. A larger effective throat may be taken if it can be shown by measurement that a given welding procedure will consistently give a larger value providing the particular welding procedure used for making the welds that are measured is followed.

w_1 and w_2 = leg on weld

iv. <u>Flare Groove Welds.</u> Flare groove welds covered by the LRFD Specification apply to welding of joints in any position, either:

(a) Sheet to sheet for flare-V groove welds, or

(b) Sheet to sheet for flare-bevel groove welds, or

(c) Sheet to thicker steel member for flare-bevel groove welds.

The factored nominal shear strength, ϕP_n, of a flare groove weld shall be determined as follows:

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

ϕ = 0.55

$$P_n = 0.833tLF_u \qquad \text{(VII.31)}$$

(b)For flare groove welds, longitudinal loading:

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

$\phi = 0.55$

$$P_n = 0.75tLF_u \qquad \text{(VII.32)}$$

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

$\phi = 0.55$

$$P_n = 1.50tLF_u \qquad \text{(VII.33)}$$

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

$\phi = 0.60$

$$P_n = 0.75t_wLF_{xx} \qquad \text{(VII.34)}$$

b. Resistance Welds. The factored nominal shear strength, ϕP_n, of spot welding shall be determined as follows:

$\phi = 0.65$

P_n = Tabulated value given in Table VII.2

TABLE VII.2

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

3. <u>Comparison.</u> The allowable load per weld for allowable stress design is P_a computed from Eq. (VII.1). For the LRFD criteria, the allowable load per weld can be calculated from the follwoing equation developed from Eq. (II.6):

$$(P_a)_{LRFD} = \phi P_n(D/L+1)/(1.2D/L+1.6) \qquad \text{(VII.35)}$$

a. <u>Arc Spot Welds.</u> Equation (VII.2) from allowable stress design and Eq. (VII.20) for LRFD criteria are based on shearing of the weld. The allowable load ratio based on shearing of arc spot welds and ϕ = 0.60 is as follows:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = \frac{0.589}{0.625}(2.5\phi)\frac{D/L+1}{1.2D/L+1.6} = 1.414\frac{D/L+1}{1.2D/L+1.6} \quad \text{(VII.36)}$$

Figure 52 shows the allowable load ratio versus dead-to-live load ratio determined from Eq. (VII.36) 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 (VII.3), (VII.4), and (VII.5) from allowable stress design and Eqs. (VII.21), (VII.22), and (VII.23) for LRFD are based on failure in the plate. The allowable load ratios for plate failure are as follows:

For $(d_a/t) \leq 0.815\sqrt{(E/F_u)}$ and $\phi = 0.60$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.5\phi\frac{D/L+1}{1.2D/L+1.6} = 1.50\frac{D/L+1}{1.2D/L+1.6} \quad \text{(VII.37)}$$

For $0.815\sqrt{(E/F_u)} < (d_a/t) < 1.397\sqrt{(E/F_u)}$ and $\phi = 0.50$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.5\phi\frac{D/L+1}{1.2D/L+1.6} = 1.25\frac{D/L+1}{1.2D/L+1.6} \quad \text{(VII.38)}$$

For $(d_a/t) \geq 1.397\sqrt{(E/F_u)}$ and $\phi = 0.50$

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.5\phi\frac{D/L+1}{1.2D/L+1.6} = 1.25\frac{D/L+1}{1.2D/L+1.6} \quad \text{(VII.39)}$$

Equations (VII.37), (VII.38), and (VII.39) 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 range of D/L ratios used in cold-formed steel, LRFD is conservative for the design of arc spot welds compared with allowable stress design.

b. Arc Seam Welds. Equation (VII.7) from allowable stress design and Eq. (VII.25) for LRFD criteria are based on shearing of the weld. The allowable load ratio based on shear failure of arc seam welds and $\phi = 0.60$ is as follows:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 0.75\pi\phi \frac{D/L+1}{1.2D/L+1.6} = 1.414 \frac{D/L+1}{1.2D/L+1.6} \qquad (VII.40)$$

Equation (VII.40) is identical to Eq. (VII.36) 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 (VII.8) from allowable stress design and Eq. (VII.26) for LRFD criteria are based on plate tearing. The allowable load ratio for plate failure and $\phi = 0.60$ is as follows:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.5\phi \frac{D/L+1}{1.2D/L+1.6} = 1.50 \frac{D/L+1}{1.2D/L+1.6} \qquad (VII.41)$$

Figure 54 shows the allowable load ratio versus dead-to-live load ratio determined from Eq. (VII.41) 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.

c. Fillet Welds. Equations (VII.12), (VII.13), and (VII.14) from allowable stress design and Eqs. (VII.27), (VII.28), and (VII.29) for LRFD design are based on plate tearing. The allowable load ratio can be computed using the following formula:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.5\phi \frac{D/L+1}{1.2D/L+1.6} \tag{VII.42}$$

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:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.50 \frac{D/L+1}{1.2D/L+1.6} \tag{VII.43}$$

For longitudinal loading with $L/t \geq 25$, the resistance factor is 0.55. Therefore, the following equation can be used to calculate the allowable load ratio:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.375 \frac{D/L+1}{1.2D/L+1.6} \tag{VII.44}$$

For transverse loading with $\phi = 0.6$, Eq. (VII.45) can be used to calculate the allowable load ratio.

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.50 \frac{D/L+1}{1.2D/L+1.6} \tag{VII.45}$$

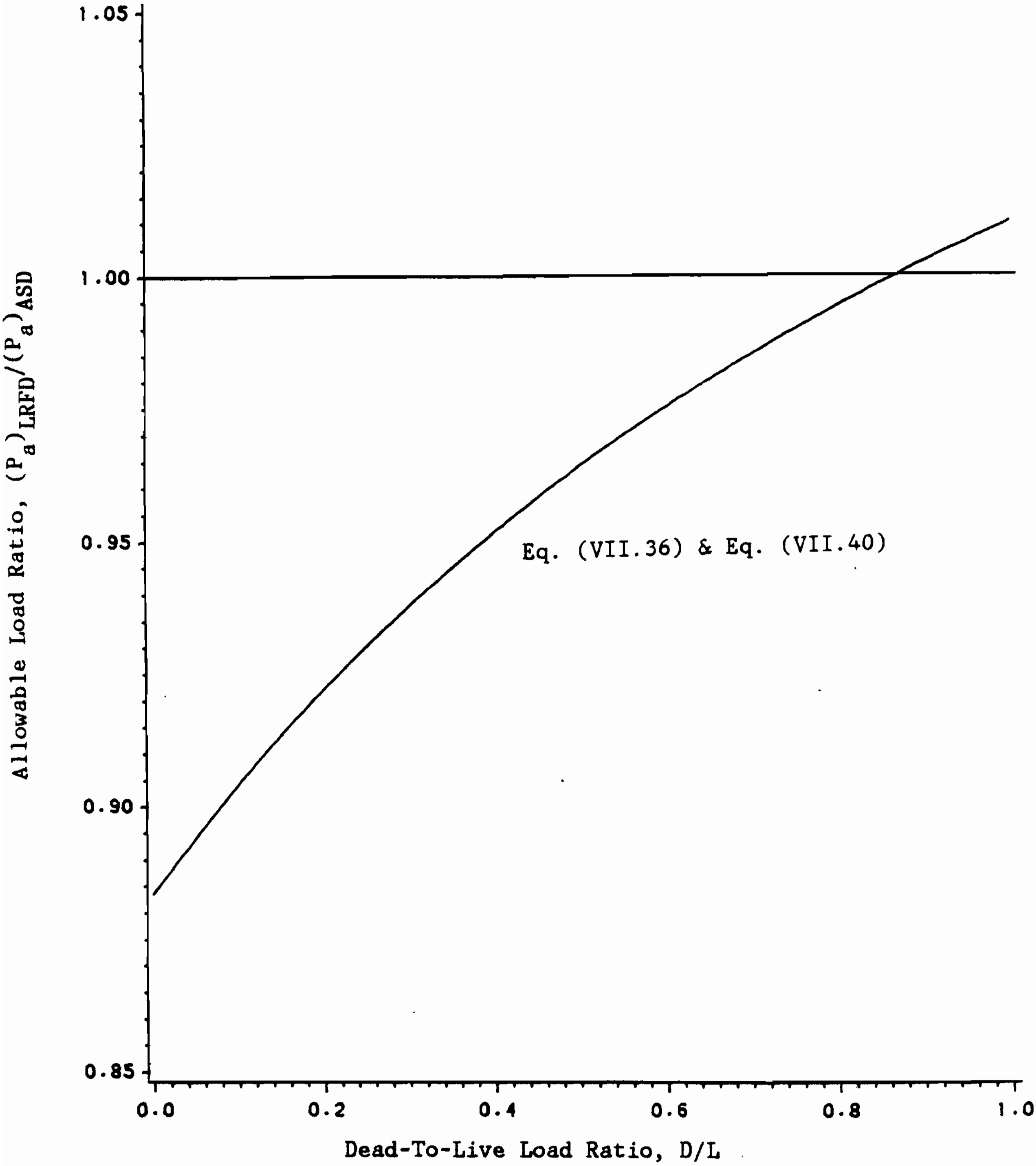

Figure 52. Allowable Load Ratio vs. D/L Ratio for Shear Failure of Arc Spot and Arc Seam Welds

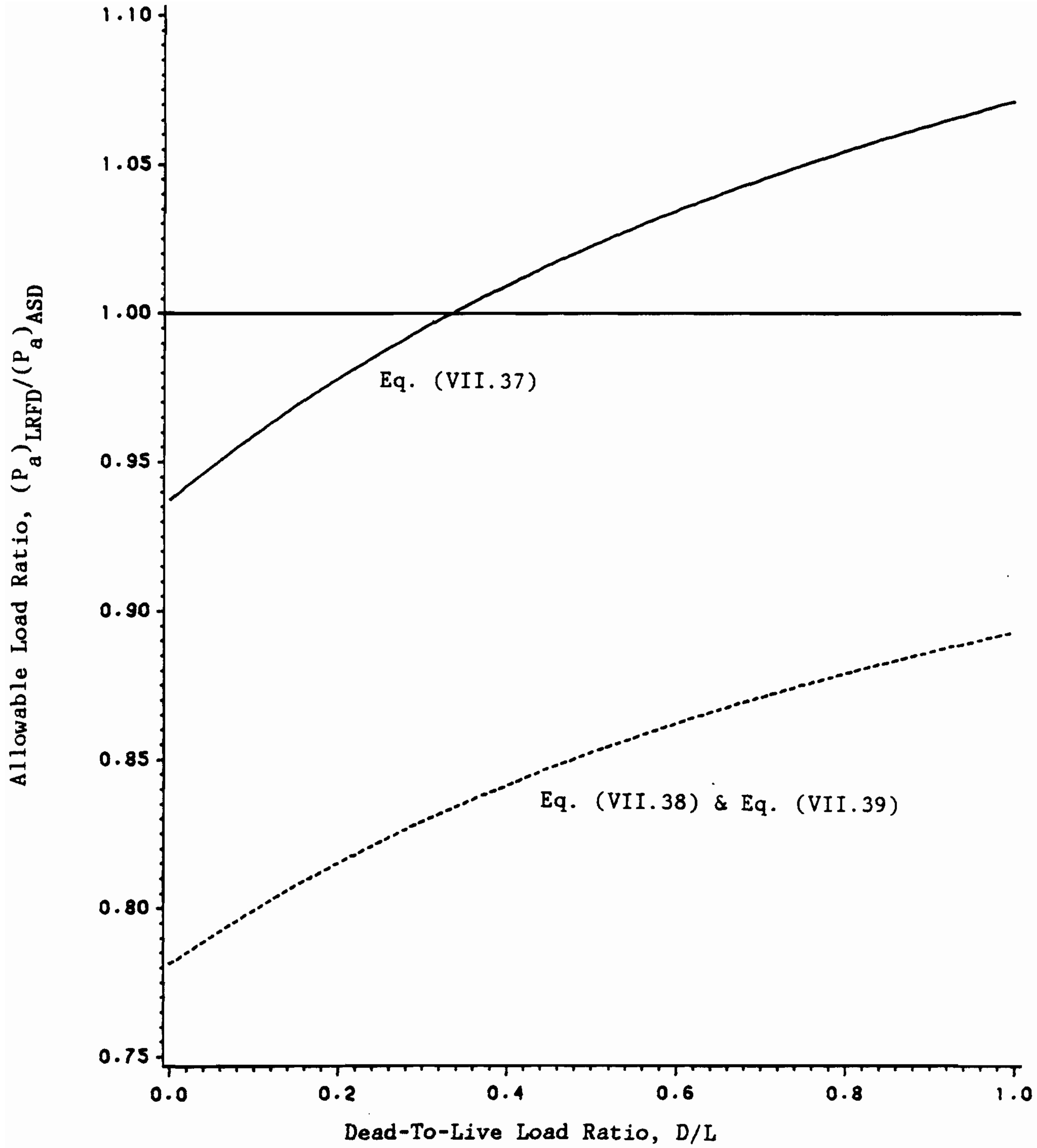

Figure 53. Allowable Load Ratio vs. D/L Ratio for Arc Spot Welds

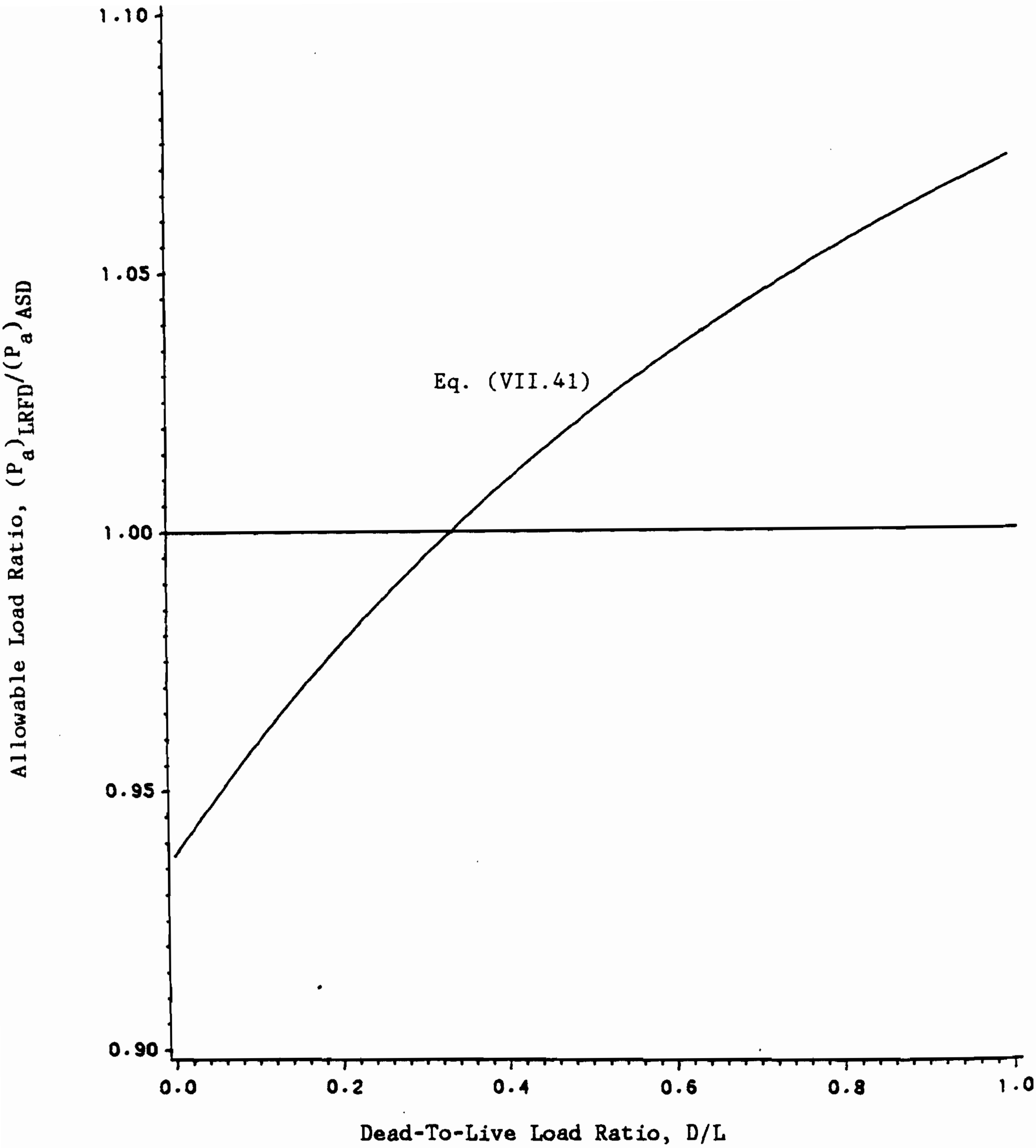

Figure 54. Allowable Load Ratio vs. D/L Ratio for Plate Tearing of Arc Seam Welds

The relationship between allowable load ratio and dead-to-live load ratio is shown in Figure 55 for plate tearing failure based on Eqs. (VII.43), (VII.44), and (VII.45). 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 (VII.15) from allowable stress design and Eq. (VII.30) for LRFD design are based on weld shearing of fillet welds. The allowable load ratio can be computed using the following formula with $\phi = 0.60$:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.5\phi \frac{D/L+1}{1.2D/L+1.6} = 1.50 \frac{D/L+1}{1.2D/L+1.6} \qquad \text{(VII.46)}$$

The relationship between allowable load ratio and dead-to-live load ratio for weld failure of fillet welds is shown in Figure 56. From the figure, LRFD criteria result in an allowable load 2.3% larger than the value computed from allowable stress design for $D/L = 0.5$.

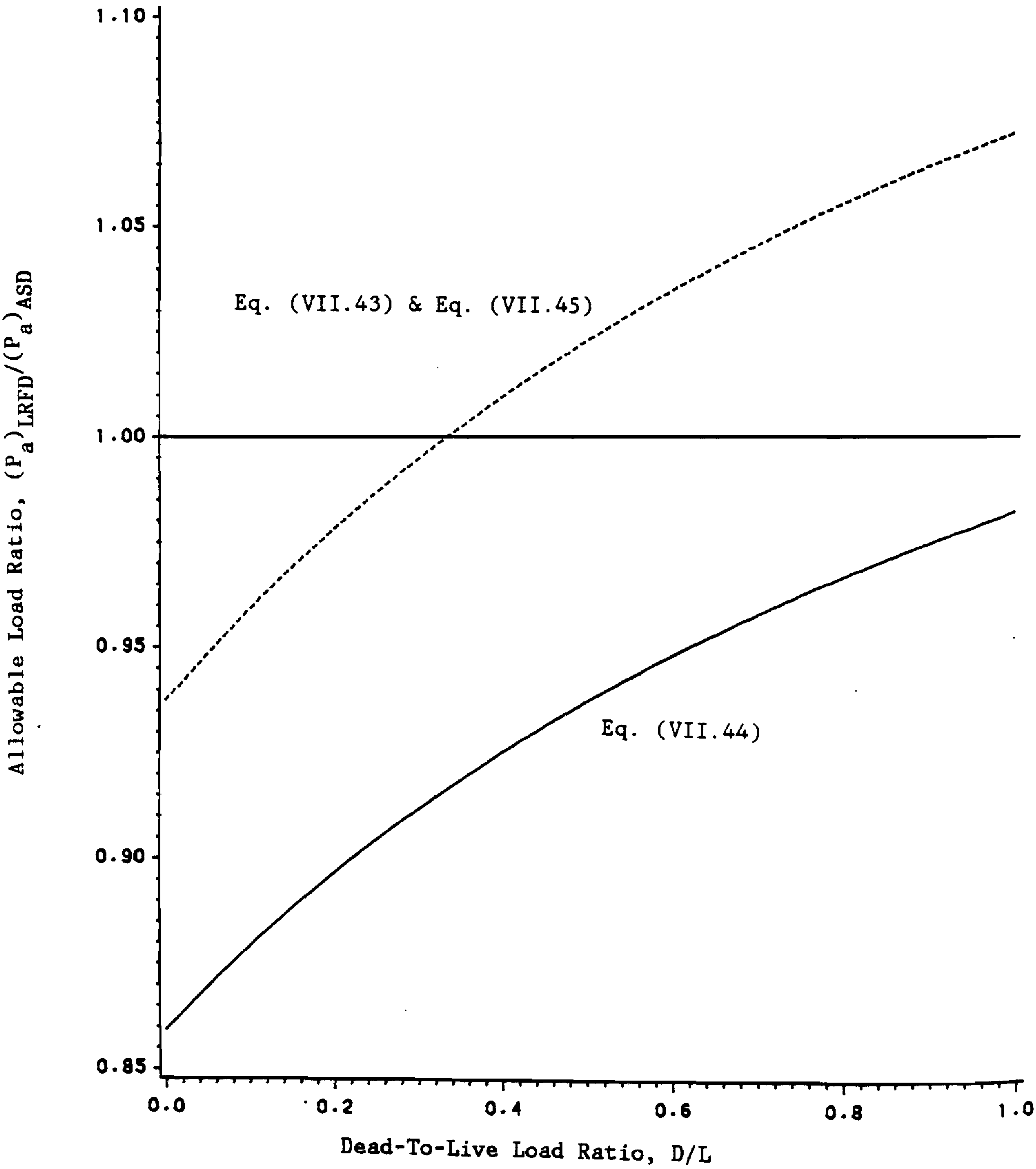

Figure 55. Allowable Load Ratio vs. D/L Ratio for Plate Tearing of Fillet Welds

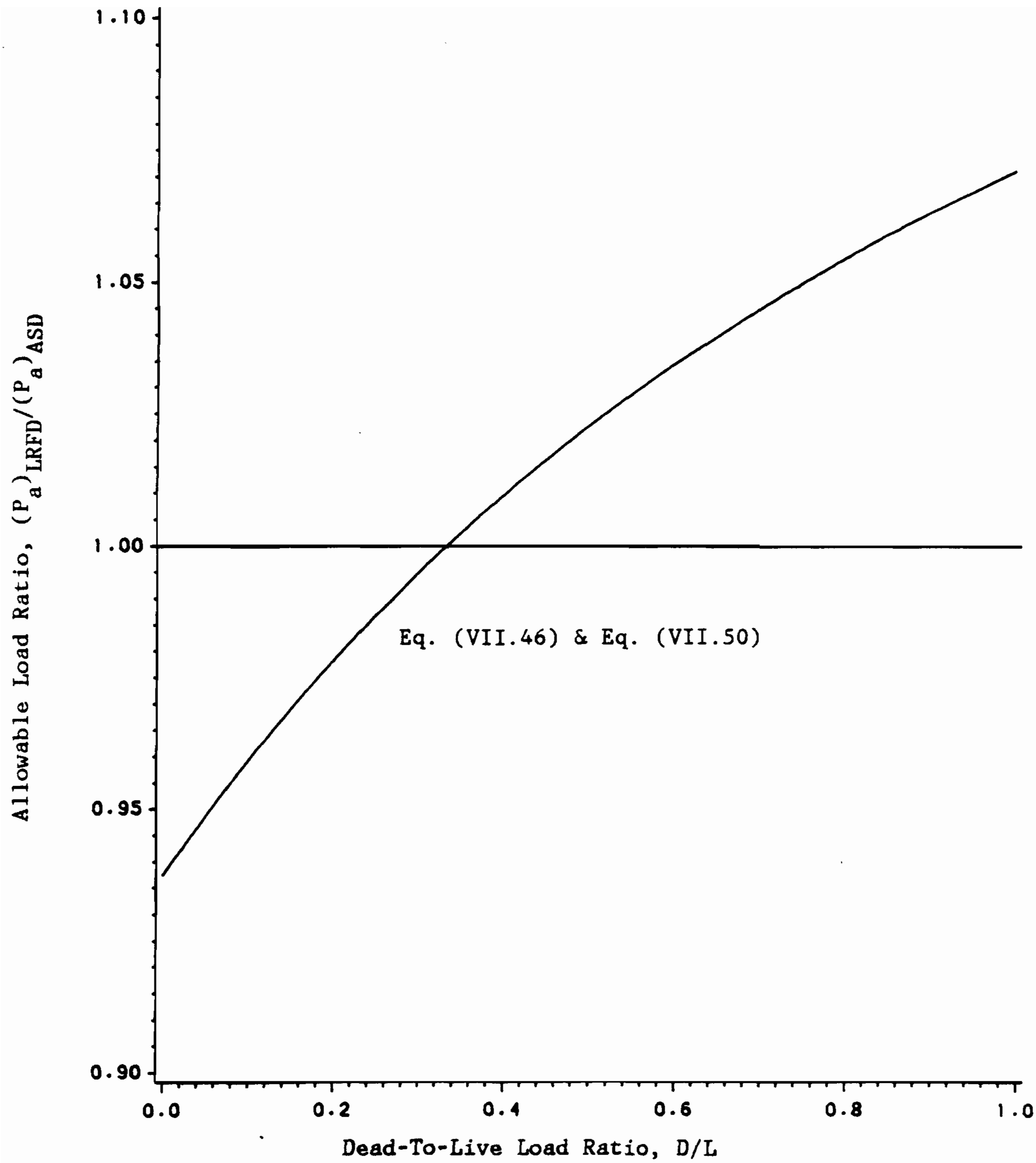

Figure 56. Allowable Load Ratio vs. D/L Ratio for Weld Failure of Fillet and Flare Groove Welds

d. Flare Groove Welds. Equations (VII.16), (VII.17), and (VII.18) from allowable stress design and Eqs. (VII.31), (VII.32), and (VII.33) for LRFD design are based on plate failure. The allowable load ratio can be computed using the following formula:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.5\Phi \frac{D/L+1}{1.2D/L+1.6} \tag{VII.47}$$

For flare-bevel groove welds loaded in the transverse direction and $\Phi = 0.55$, the following equation can be used for allowable load ratio:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.375 \frac{D/L+1}{1.2D/L+1.6} \tag{VII.48}$$

For flare groove welds loaded in the longitudinal direction and $\Phi = 0.55$, the allowable load ratio can be computed as follows:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.375 \frac{D/L+1}{1.2D/L+1.6} \tag{VII.49}$$

Figure 57 shows the relationship between allowable load ratio and dead-to-live load ratio computed from Eqs. (VII.48) and (VII.49). 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 (VII.19) from allowable stress design and Eq. (VII.34) for LRFD design are based on shear failure of the weld. With $\phi = 0.60$, the allowable load ratio can be computed as follows:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.5\phi \frac{D/L+1}{1.2D/L+1.6} = 1.50 \frac{D/L+1}{1.2D/L+1.6} \qquad \text{(VII.50)}$$

Equation (VII.50) is identical to Eq. (VII.46) which is the allowable load ratio for fillet welds based on the same type of failure. Figure 56 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.

e. Resistance Welds. The allowable loads per spot weld for allowable stress design in Table VII.1 were derived from the values in Table VII.2 using a factor of safety of 2.5. Therefore, the following equation for allowable load ratio can be used for $\phi = 0.65$:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.5\phi \frac{D/L+1}{1.2D/L+1.6} = 1.625 \frac{D/L+1}{1.2D/L+1.6} \qquad \text{(VII.51)}$$

The relationship between allowable load ratio and dead-to-live load ratio is shown in Figure 58 for resistance welds. As shown from the figure, LRFD criteria always result in higher values of allowable load than allowable stress design for all dead-to-live load ratios. For D/L = 0.5, the difference between the allowable loads is 10.8%.

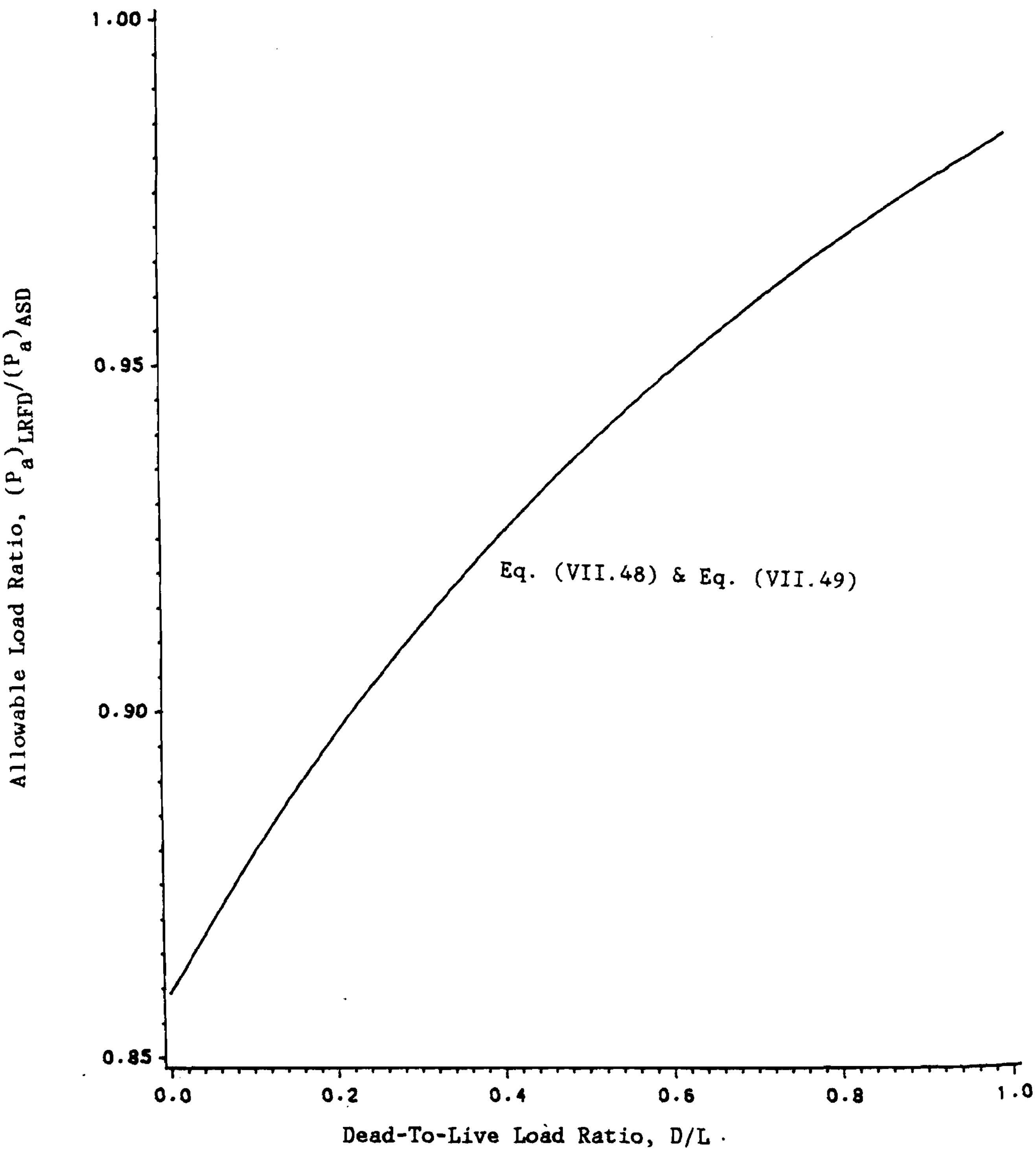

Figure 57. Allowable Load Ratio vs. D/L Ratio for Plate Tearing of Flare Groove Welds

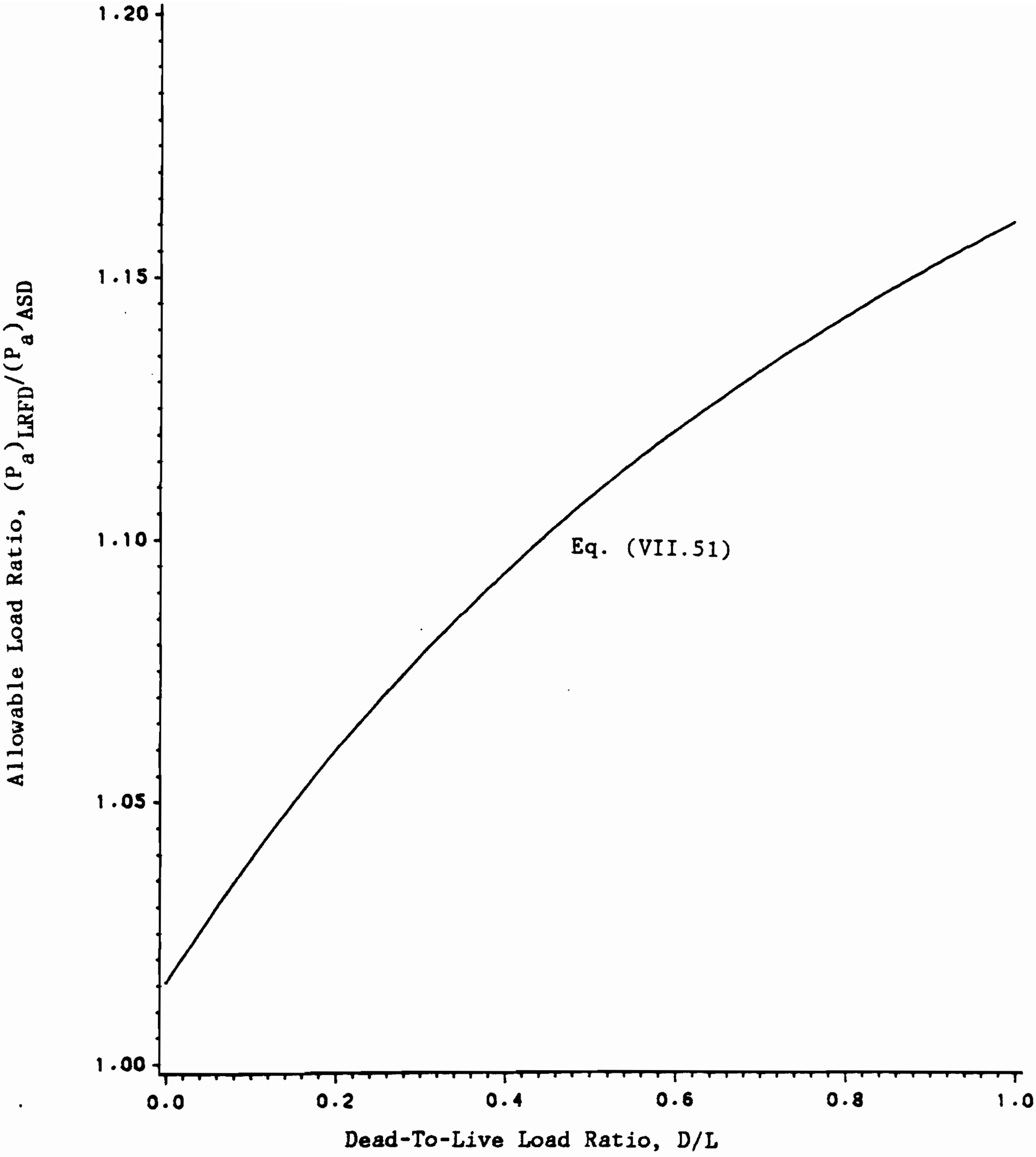

Figure 58. Allowable Load Ratio vs. D/L Ratio for Resistance Welds

C. BOLTED CONNECTIONS

The AISI Specification and the LRFD Specification for bolted connections of cold-formed steel structural members apply to members in which the thickness of the thinnest connected part is less than 3/16 in. The AISC Specification should be used for bolted connections when the thickness of the thinnest connected part is greater than or equal to 3/16 in.

1. Allowable Stress Design. According to Section E3 of the AISI Specification, the following requirements govern bolted connections of cold-formed steel structural members in which the thickness of the thinnest connected part is less than 3/16 inch and there are no gaps between connected parts. For bolted connections in which the thinnest connected part is equal to or greater than 3/16 inch, refer to AISC Specification.

a. Spacing and Edge Distance. The minimum spacing and edge distance in the line of the stress has to be checked to prevent tearing of the steel sheet due to shear. According to Section E3.1 of the AISI Specification, the distance, e, 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 toward which the force is directed shall not be less than the value of e_{min} determined as follows:

$$e_{min} = e\Omega_e \tag{VII.52}$$

where

$$e = P/F_u t \tag{VII.53}$$

(a) When $F_u/F_{sy} \geq 1.15$:

Ω_e = Factor of safety for sheet tearing

$= 2.0$

(b) When $F_u/F_{sy} < 1.15$:

Ω_e = Factor of safety for sheet tearing

$= 2.22$

where

P = Force transmitted by bolt

t = Thickness of thinnest connected part

F_u = Tensile strength of the connected part

F_{sy} = Specified yield point of the connected part

b. <u>Tension in Connected Part.</u> Tearing of the net section in tension is caused by stress concentrations resulting from the presence of holes and the concentrated force transmitted by the bolt to the sheets. According to Section E3.2 of the AISI Specification, the tension force on the net section of a bolted connection shall not exceed T_a from Section C2 of the AISI Specification or P_a calculated as follows:

$$P_a = P_n/\Omega_t \qquad \text{(VII.54)}$$

where

$P_n = A_n F_t$

A_n = Net section area

F_t and Ω_t are defined as follows:

(a) When $t \geq 3/16$ in.:

Use AISC Specification

(b) When $t < 3/16$ inch and washers are provided under both the bolt head and nut

$$F_t = (1.0-0.9r+3rd/s)F_u \leq F_u \qquad \text{(VII.55)}$$

Ω_t = Factor of safety for tension on the net section

= 2.0 for double shear

= 2.22 for single shear

(c) When t < 3/16 inch and either washers are not provided under the bolt head and nut, or only one washer is provided under either the bolt head or nut

$$F_t = (1.0 - r + 2.5rd/s)F_u \leq F_u \qquad \text{(VII.56)}$$

Ω_t = Factor of safety for tension on the net section

= 2.22

where

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

F_t = Nominal tension stress limit on net section

d = Diameter of bolt

c. Bearing. Bearing failure occurs when the steel sheet piles up in front of the bolts. This occurs when the edge distance or longitudinal spacing of the bolts is relatively large. According to Section E3.3 of the AISI Specification, the bearing force shall not exceed P_a calculated as follows:

$$P_a = P_n/\Omega_b \qquad \text{(VII.57)}$$

where

$$P_n = F_p dt \qquad \text{(VII.58)}$$

Ω_b = Safety factor for bearing

$= 2.22$

F_p = Nominal bearing stress as given in Tables VII.3 and VII.4

Table VII.3
Nominal Bearing Stress 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 bearing stress, F_p
$\geq$ 0.024 but < 3/16	Inside sheet of double shear connection	$\geq$ 1.15	$3.33F_u$
		< 1.15	$3.00F_u$
	Single shear and outside sheets of double shear connection	No limit	$3.00F_u$
$\geq$ 3/16	See AISC Specification		

Table VII.4
Nominal Bearing Stress 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 bearing stress, F_p
$\geq$ 0.036 but < 3/16	Inside sheet of double shear connection	$\geq$ 1.15	$3.00F_u$
	Single shear and outside sheets of double shear connection	$\geq$ 1.15	$2.22F_u$
$\geq$ 3/16	See AISC Specification		

d. Shear and Tension in Bolts. The strengths of the bolts in shear and tension have to be checked for bolted connections. According to Section E3.4 of the AISI Specification, the bolt force resulting from shear, tension or combination of shear and tension shall not exceed allowable bolt force, P_a, calculated as follows:

$$P_a = A_b F \tag{VII.59}$$

where

A_b = Gross cross-sectional area of bolt

F is given by F_v, F_t or F_t' in Tables VII.5 and VII.6

Table VII.5
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

When bolts are subject to a combination of shear and tension, the tension force shall not exceed to a combination of shear and tension, the tension force shall not exceed the allowable force, P_a, based on F_t', given in Table VII.6, where f_v, the shear stress produced by the same forces, shall not exceed the allowable value F_v given in Table VII.5.

Table VII.6
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.	$26 - 1.8f_v \leq 20$	

2. LRFD Criteria. According to Section E3 of the LRFD Specification, the following LRFD design criteria govern bolted connections used for cold-formed steel structural members in which the thickness of the thinnest connected part is less than 3/16 inch. For bolted connections in which the thickness of the thinnest connected part is equal to or greater than 3/16 inch, refer to AISC Specification.

a. Spacing and Edge Distance. According to Section E3.1 of the LRFD Specification, the factored nominal shear strength, ϕP_n, of the connected part along two parallel lines in the direction of applied force shall be determined as follows:

$$P_n = teF_u \tag{VII.60}$$

(a) When $F_u/F_{sy} \geq 1.15$:

$\phi = 0.70$

(b) When $F_u/F_{sy} < 1.15$:

$\phi = 0.60$

where

ϕ = Resistance factor

P_n = Nominal resistance per bolt

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} = Specified yield point of the connected part

b. Tension in Connected Part. According to Section E3.2 of the LRFD Specification, the factored nominal tensile strength, ϕP_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

$$P_n = (1.0-0.9r+3rd/s)F_uA_n \leq F_uA_n \qquad \text{(VII.61)}$$

$\phi = 0.65$ for double shear connection

$\phi = 0.55$ for single shear connection

(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

$\phi = 0.65$

$$P_n = (1.0-r+2.5rd/s)F_uA_n \leq F_uA_n \qquad \text{(VII.62)}$$

In addition, the factored nominal tensile strength shall not exceed the following value:

$\phi = 0.95$

$$P_n = F_{sy} A_n \tag{VII.63}$$

where

A_n = Net area of the connected part

d = Diameter of bolt

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

c. Bearing. According to Section E3.3 of the LRFD Specification, the factored nominal bearing strength, ϕP_n, shall be determined by the values of ϕ and P_n given in Tables VII.7 and VII.8 for the applicable thickness and F_u/F_{sy} ratio of the connected part and the type of joint used in the connection.

TABLE VII.7
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	Resistance Factor ϕ	Nominal Resistance P_n
≥ 0.024 but $< 3/16$	Inside sheet of double shear connection	≥ 1.15	0.55	$3.33F_u dt$
		< 1.15	0.65	$3.00F_u dt$
	Single shear and outside sheets of double shear connection	No limit	0.60	$3.00F_u dt$
$\geq 3/16$	See Section E3			

TABLE VII.8
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	Resistance Factor ϕ	Nominal Resistance P_n
$\geq$ 0.036 but < 3/16	Inside sheet of double shear connection	≥ 1.15	0.70	$3.00F_u dt$
	Single shear and outside sheets of double shear connection	≥ 1.15	0.65	$2.22F_u dt$
$\geq$ 3/16	See Section E3			

d. <u>Shear and Tension in Bolts.</u> According to Section E3.4 of the LRFD Specification, the bolt force in shear or tension produced by factored loads shall not exceed the factored nominal strength, ϕP_n, determined as follows:

ϕ = Resistance factor given in Table VII.9

$$P_n = A_b F_n \tag{VII.64}$$

where

A_b = Gross cross-sectional area of bolt

F_n is given by F_{nv} or F_{nt} in Table VII.9.

When bolts are subject to a combination of shear and tension produced by factored loads, the tension force shall not exceed the factored nominal strength, ϕP_n, based on ϕ = 0.75 and $P_n = A_b F'_{nt}$, where F'_{nt} is given in Table VII.10, in which f_v is the shear stress produced by the same factored loads. The shear force shall not exceed the factored shear strength, $\phi A_b F_{nv}$, determined in accordance with Table VII.9.

3. Comparison. 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 follwoing equation developed from Eq. (II.6):

$$(P_a)_{LRFD} = \phi P_n(D/L+1)/(1.2D/L+1.6) \qquad \text{(VII.65)}$$

a. Spacing and Edge Distance. For allowable stress design, the allowable load can be computed for a given edge distance by solving for P in Eqs. (VII.52) and (VII.53).

For $F_u/F_{sy} \geq 1.15$,

$$(P_a)_{ASD} = 0.5teF_u \qquad \text{(VII.66)}$$

For $F_u/F_{sy} < 1.15$,

$$(P_a)_{ASD} = 0.45teF_u \qquad \text{(VII.67)}$$

The allowable load for LRFD can be computed using Eq. (VII.65). The allowable loads from Eqs. (VII.66) and (VII.67) were derived from the ultimate load in Eq. (VII.60) 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 $F_u/F_{sy} \geq 1.15$, $\phi = 0.70$:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.4\,\frac{D/L+1}{1.2D/L+1.6} \qquad \text{(VII.68)}$$

For $F_u/F_{sy} < 1.15$, $\phi = 0.60$:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.332\,\frac{D/L+1}{1.2D/L+1.6} \qquad \text{(VII.69)}$$

TABLE VII.9
Nominal Tensile and Shear Strengths for Bolts

Description of Bolts	Tensile Strength		Shear Strength	
	Resistance Factor ϕ	Nominal Stress F_{nt}	Resistance Factor ϕ	Nominal Stress F_{nv}
A307 Bolts, Grade A (1/4 in. $\leq$ d < 1/2 in.)	0.75	40.5	0.65	24.0
A307 Bolts, Grade A (d $\geq$ 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. $\leq$ d < 1/2 in.), when threads are not excluded from shear planes		101.0		59.0
A354 Grade B Bolts (1/4 in. $\leq$ d < 1/2 in.), when threads are excluded from shear plans		101.0		90.0
A449 Bolts (1/4 in. $\leq$ d < 1/2 in.), when threads are not excluded from shear planes		81.0		47.0
A449 Bolts (1/4 in. $\leq$ 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 VII.10

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. $\leq d <$ 1/2 in.	$47-2.4f_v \leq 40.5$	
when $d \geq$ 1/2 in.	$52-2.4f_v \leq 45$	

Figure 59 shows the relationships between allowable load ratio and dead-to-live load ratio for Eqs. (VII.68) and (VII.69). 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.

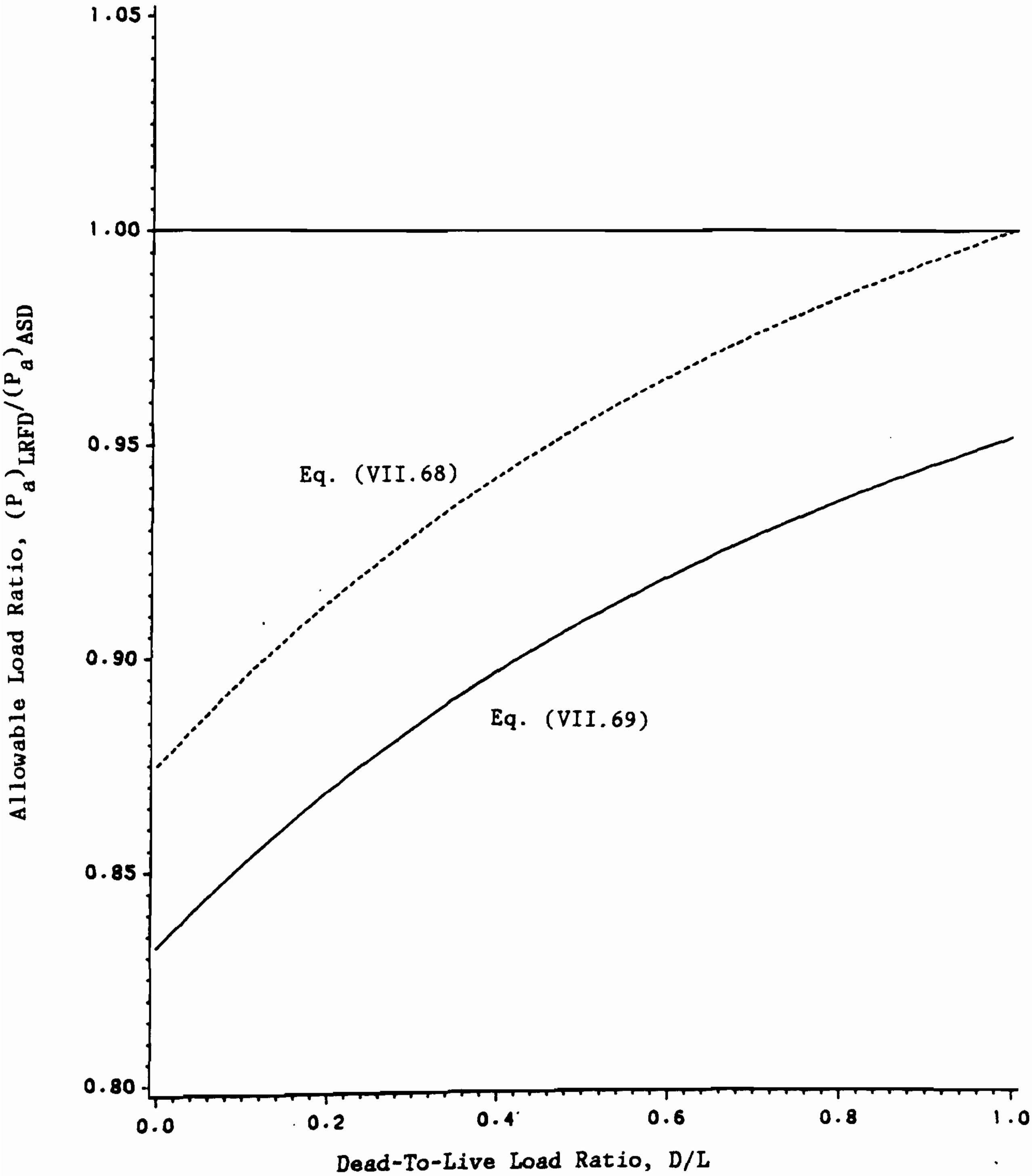

Figure 59. Allowable Load Ratio vs. D/L Ratio for Minimum Edge Distance of Bolts

b. Tension in Connected Parts. For allowable stress design, the allowable tension on the net section can be computed by Eq. (VII.70).

$$(P_a)_{ASD} = A_n F_t / \Omega_t \qquad \text{(VII.70)}$$

For LRFD, the allowable tension on the net section can be computed using Eq. (VII.65).

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 Chapter III of this report. The allowable load ratios can be computed as follows:

For double shear connections with washers and $\phi = 0.65$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.0\phi \frac{D/L+1}{1.2D/L+1.6} = 1.30 \frac{D/L+1}{1.2D/L+1.6} \qquad \text{(VII.71)}$$

For single shear connections with washers and $\phi = 0.55$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.22\phi \frac{D/L+1}{1.2D/L+1.6} = 1.221 \frac{D/L+1}{1.2D/L+1.6} \qquad \text{(VII.72)}$$

For connections without washers and $\phi = 0.65$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 2.22\phi \frac{D/L+1}{1.2D/L+1.6} = 1.443 \frac{D/L+1}{1.2D/L+1.6} \qquad \text{(VII.73)}$$

Figure 60 shows the allowable load ratio versus dead-to-live load ratio for the three cases represented by Eqs. (VII.71), (VII.72), and (VII.73). 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 60 also shows that LRFD is very conservative for connections with washers under the bolt head and nut compared with allowable stress design.

c. Bearing. The allowable load based on allowable stress design can be computed using the following equation:

$$(P_a)_{ASD} = F_p td/\Omega_b \tag{VII.74}$$

For LRFD, Eq. (VII.65) can be used to calculate the allowable load.

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:

(i) Connections with washers:

For inside sheets of double shear connections with

$F_u/F_{sy} \geq 1.15$ and $\phi = 0.55$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.221 \frac{D/L+1}{1.2D/L+1.6} \tag{VII.75}$$

For inside sheets of double shear connections with

$F_u/F_{sy} < 1.15$ and $\phi = 0.65$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.443 \frac{D/L+1}{1.2D/L+1.6} \tag{VII.76}$$

For single shear and outside sheets of double shear connections with $\phi = 0.60$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.332 \frac{D/L+1}{1.2D/L+1.6} \tag{VII.77}$$

(ii) Connections without washer or with only one washer:

For inside sheets of double shear connections with

$F_u/F_{sy} \geq 1.15$ and $\phi = 0.70$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.554 \frac{D/L+1}{1.2D/L+1.6} \quad \text{(VII.78)}$$

For single shear and outside sheets of double shear connections with $F_u/F_{sy} \geq 1.15$ and $\phi = 0.65$,

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 1.443 \frac{D/L+1}{1.2D/L+1.6} \quad \text{(VII.79)}$$

The relationships between allowable load ratio and dead-to-live load ratio for Eqs. (VII.75) through (VII.79) are shown in Figure 61. 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 will be very conservative compared with allowable stress design.

d. <u>Shear and Tension in Bolts.</u> The allowable load based on allowable stress design can be computed as follows:

$$(P_a)_{ASD} = A_bF \quad \text{(VII.80)}$$

For LRFD, Eq. (VII.65) can be used to calculate the allowable load.

Therefore, the allowable load ratio for shear or tension of bolts is:

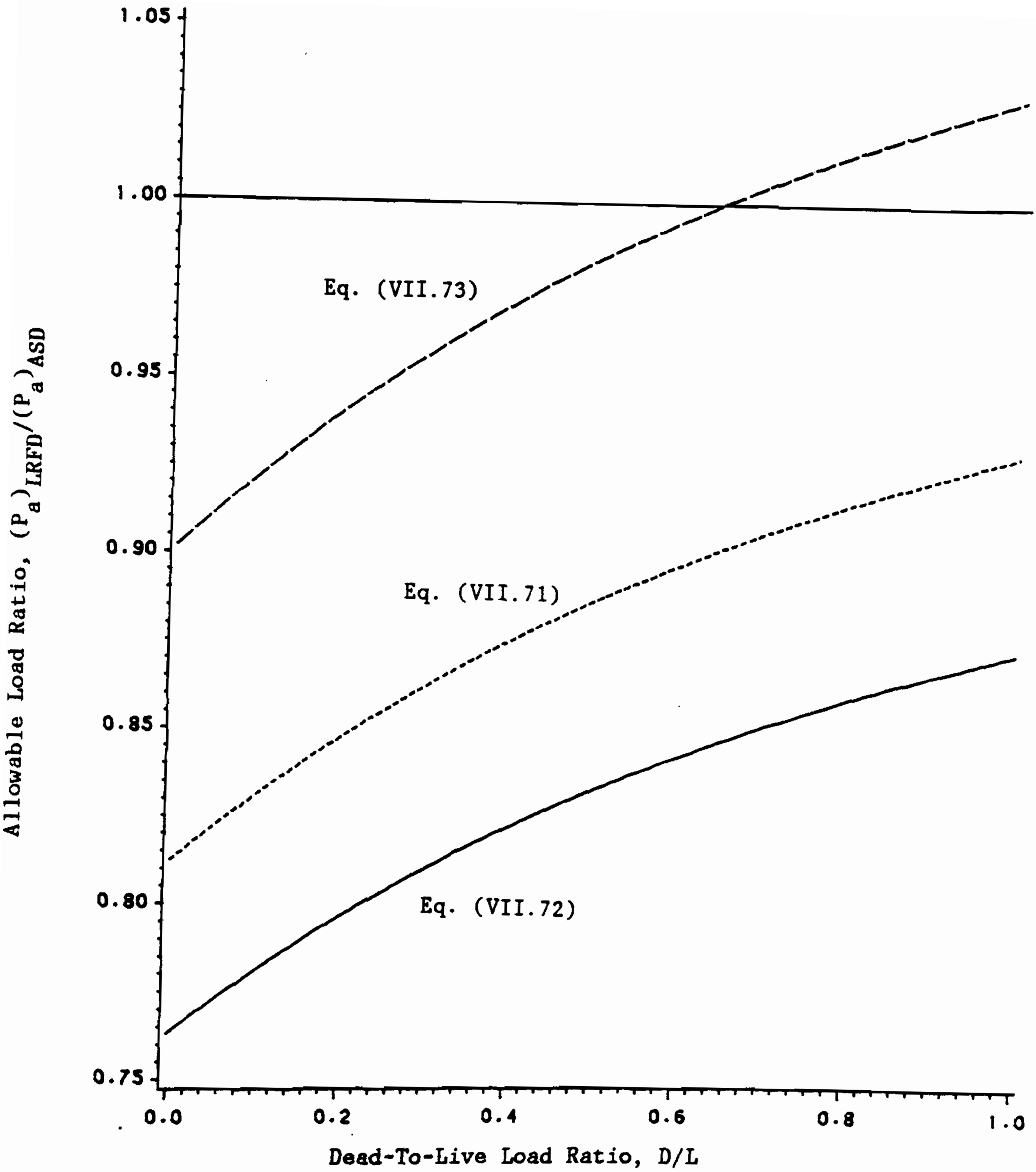

Figure 60. Allowable Load Ratio vs. D/L Ratio for Tension on Net Section

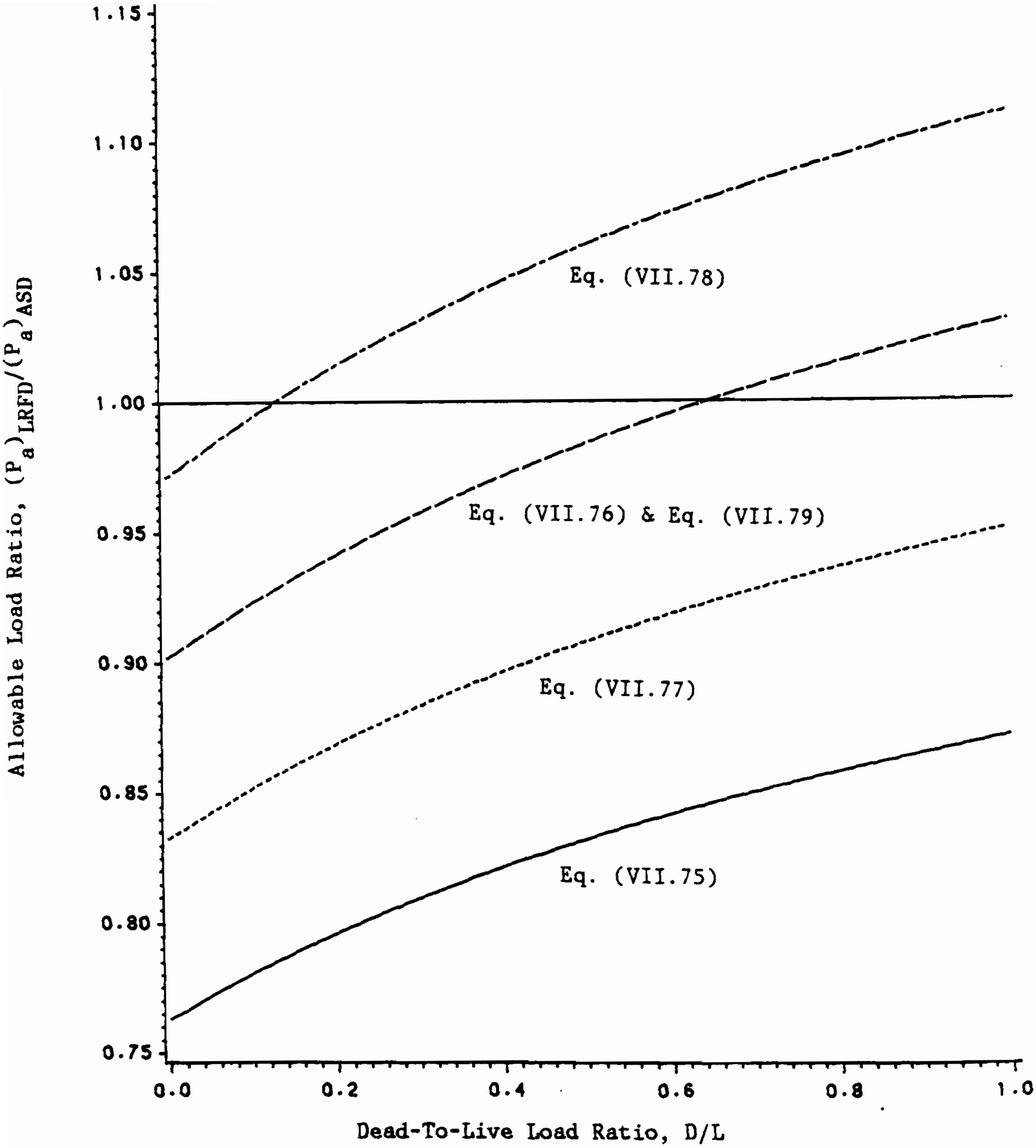

Figure 61. Allowable Load Ratio vs. D/L Ratio for Bearing Strength of Bolted Connections

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = \frac{\phi A_b F_n}{A_b F}\left[\frac{D/L+1}{1.2D/L+1.6}\right] = \frac{\phi F_n}{F}\left[\frac{D/L+1}{1.2D/L+1.6}\right] \qquad \text{(VII.81)}$$

Equation (VII.81) can be expressed in the following form:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = (K_b)\ \frac{D/L+1}{1.2D/L+1.6} \qquad \text{(VII.82)}$$

where

$$K_b = \phi F_n/F \qquad \text{(VII.83)}$$

Table VII.11 lists the values of K_b calculated from the values of F, F_n, and ϕ provided in Tables VII.5 and VII.9. Figures 62 through 66 show the relationships between allowable load ratio and dead-to-live load ratio for the bolts in Table VII.11 using Eq. (VII.82).

Figure 62 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.

Table VII.11
K_b Values for Standard Bolts

Description of Bolts	Shear Strength		Tension Strength
	Threads not Excluded from Shear Plane	Threads Excluded from Shear Plane	
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 63 shows the allowable load ratio versus dead-to-live load ratio for A354 Grade B bolts (1/4 in. ≤ d < 1/2 in.) 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 5.4% 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 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 the same as 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.

Figure 64 shows the allowable load ratio versus dead-to-live load ratio for A449 bolts (1/4 in. $\leq$ d < 1/2 in.) 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 3.6% 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 15.7% 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.

Figure 65 shows the allowable load ratio versus dead-to-live load ratio for A490 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 6.6% 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 6.8% 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 the same as the value based on allowable stress design.

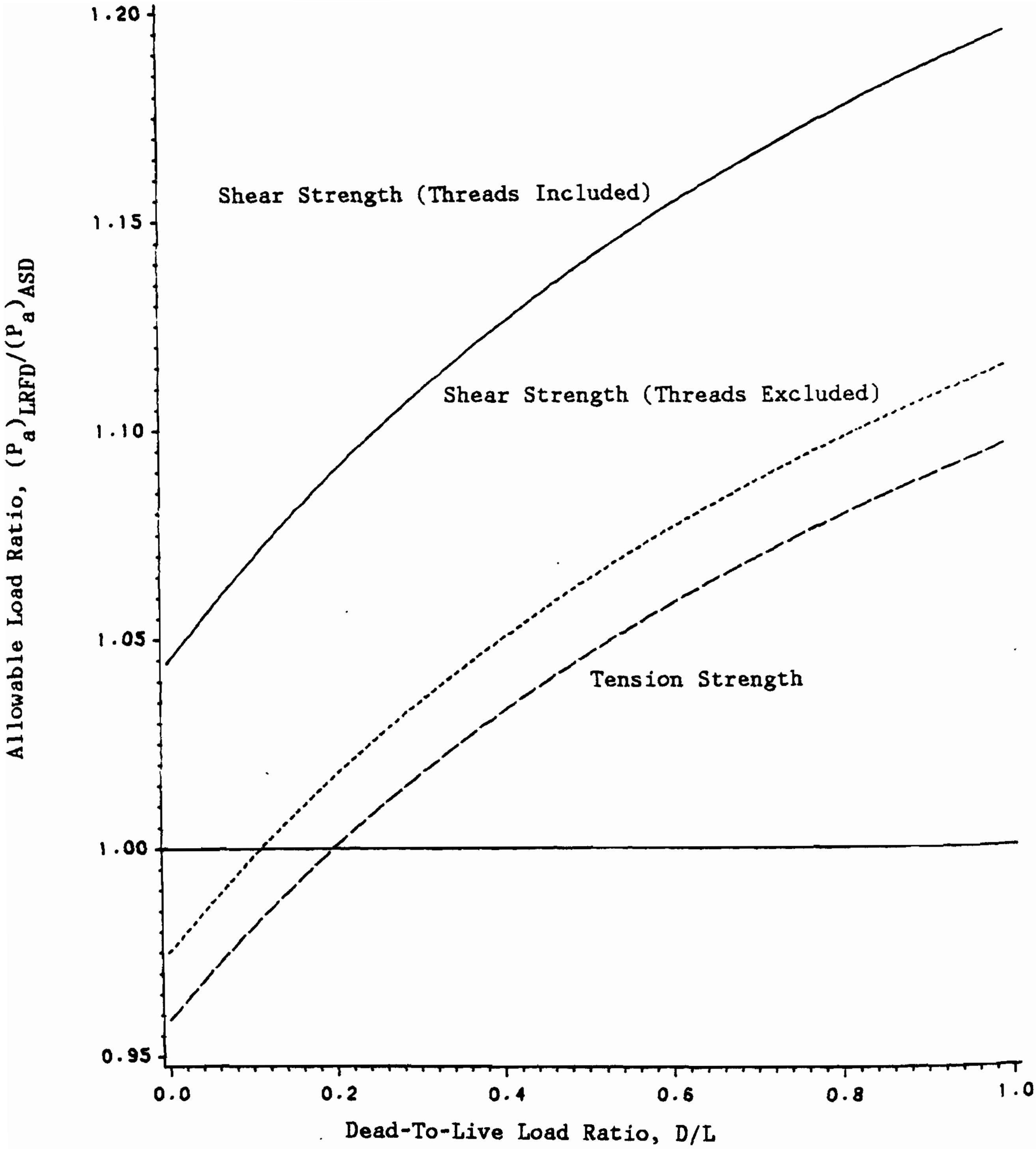

Figure 62. Allowable Load Ratio vs. D/L Ratio for Shear or Tension Strength on A325 Bolts

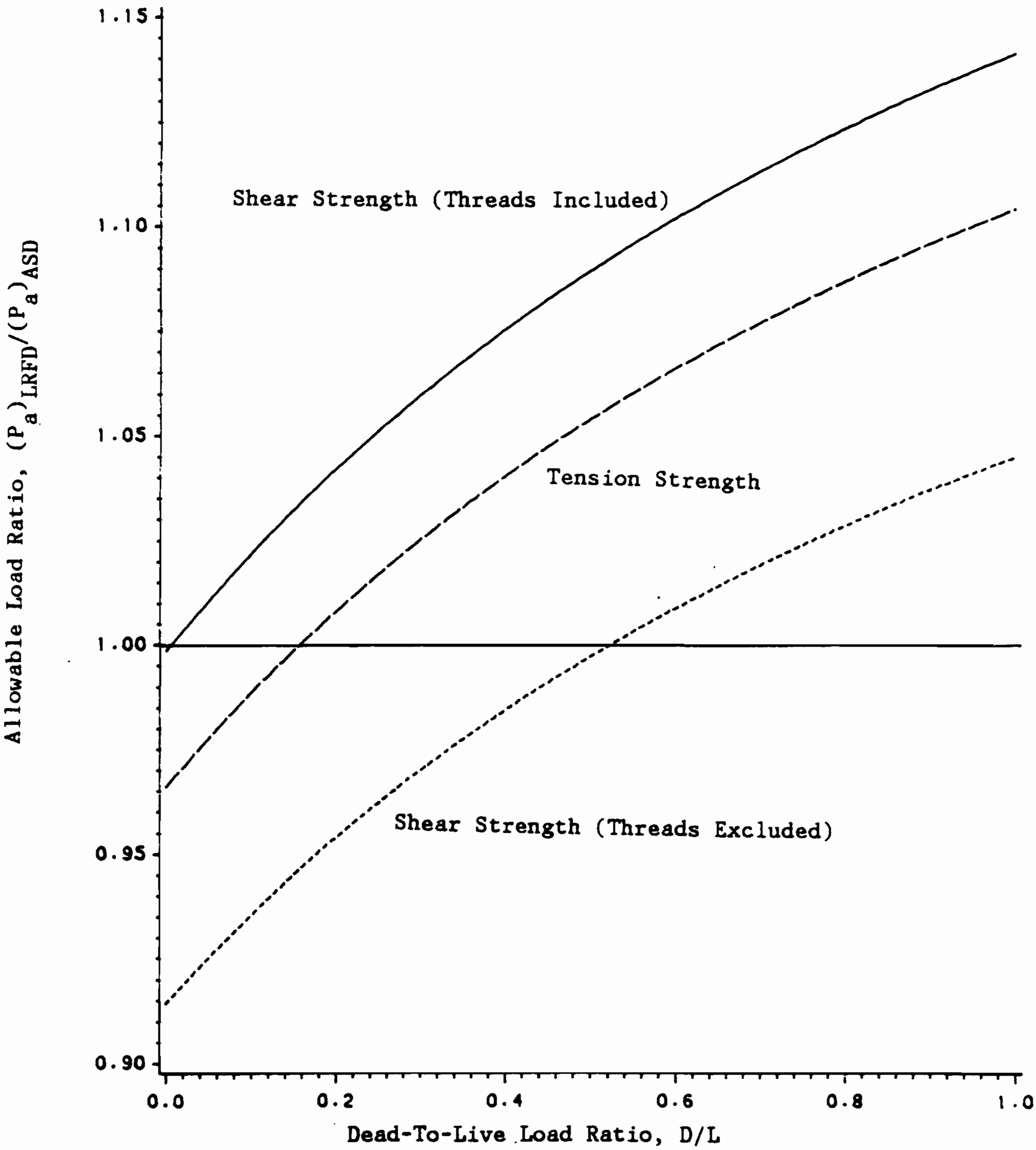

Figure 63. Allowable Load Ratio vs. D/L Ratio for Shear or Tension Strength on A354 Grade B Bolts (1/4 in. ≤ d < 1/2 in.)

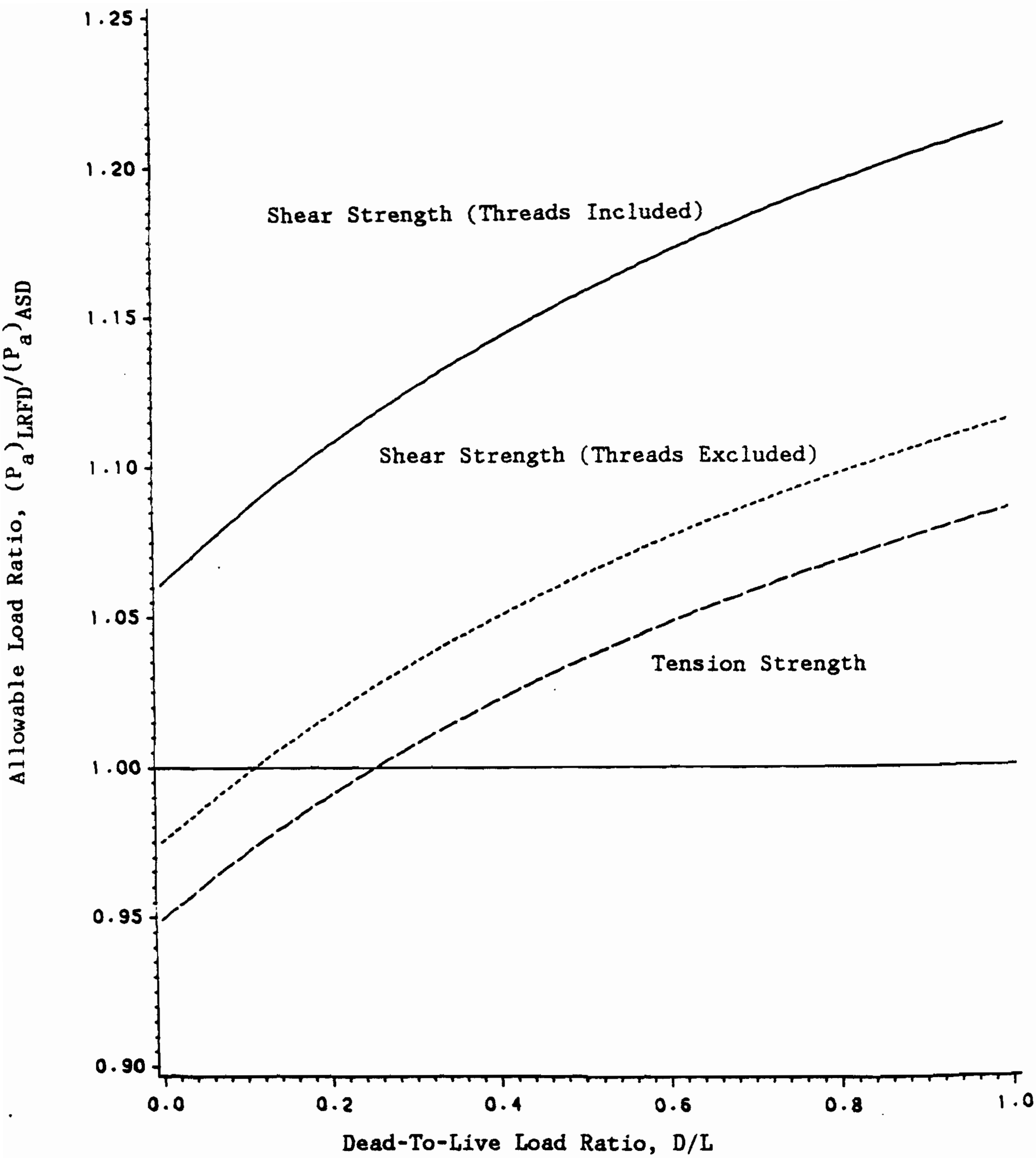

Figure 64. Allowable Load Ratio vs. D/L Ratio for Shear or Tension Strength on A449 Bolts (1/4 in. $\leq$ d < 1/2 in.)

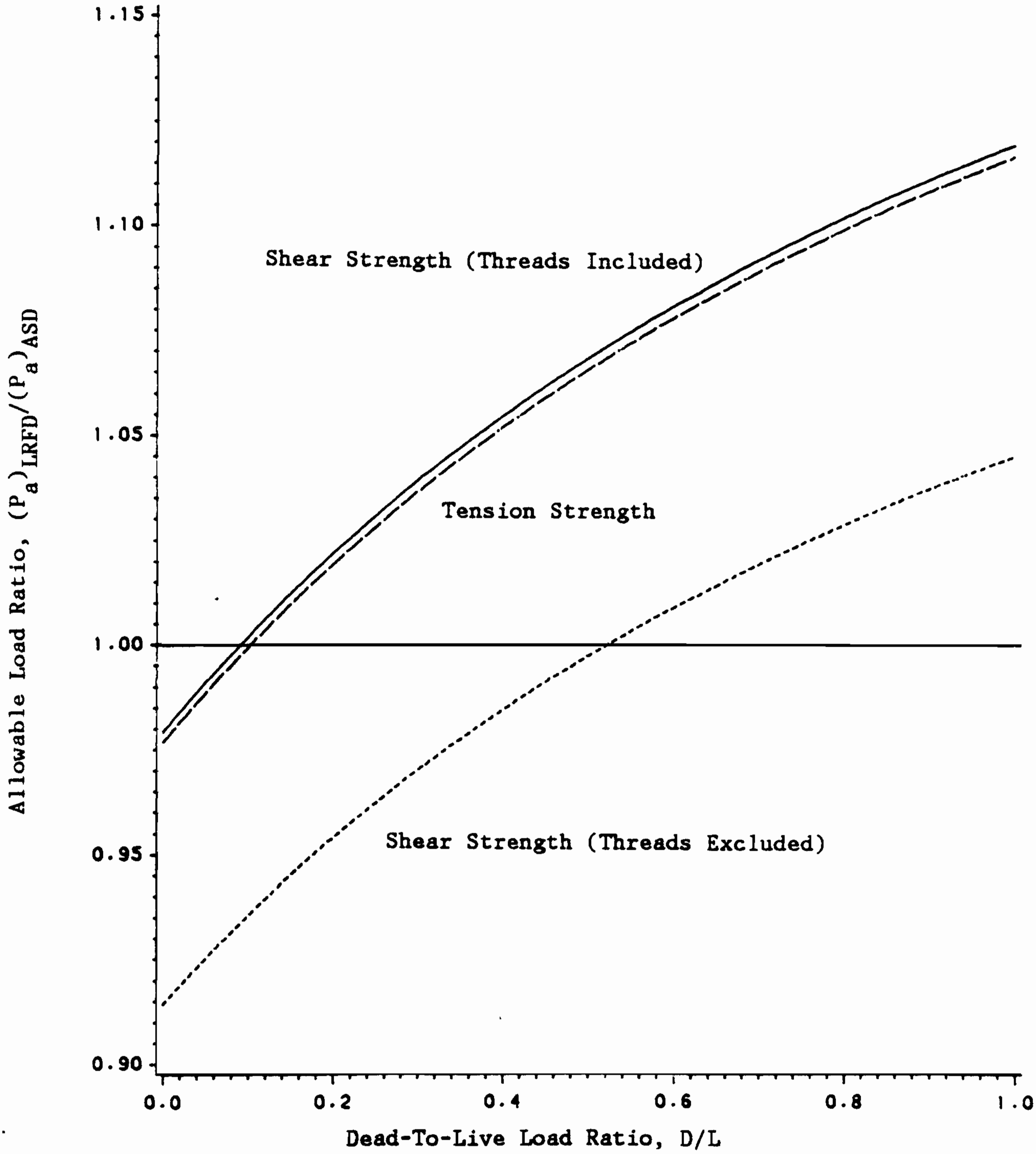

Figure 65. Allowable Load Ratio vs. D/L Ratio for Shear or Tension Strength on A490 Bolts

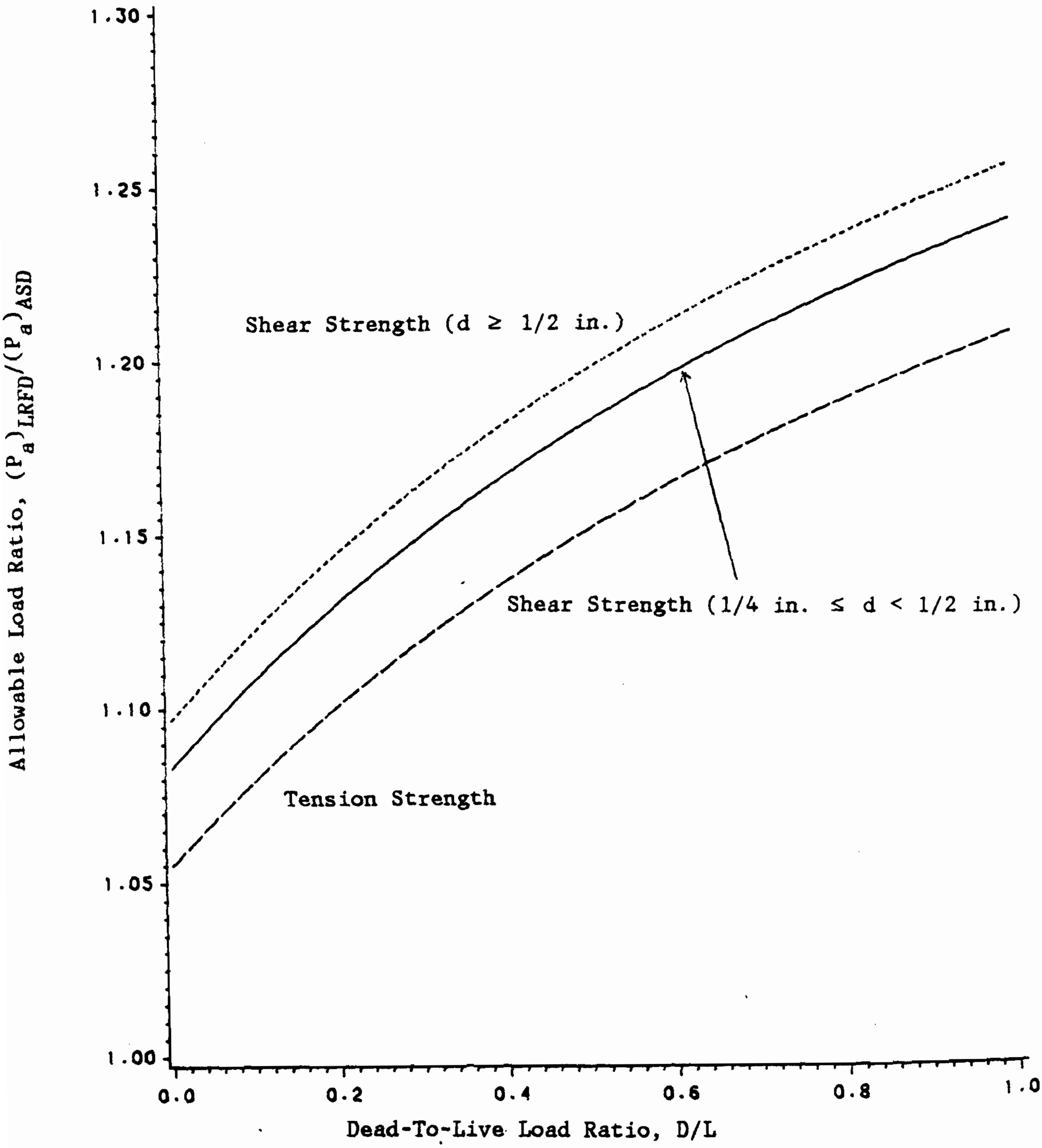

Figure 66. Allowable Load Ratio vs. D/L Ratio for Shear or Tension Strength on A307 Bolts, Grade A

Figure 66 shows the allowable load ratio versus dead-to-live load ratio for A307 Grade A 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 15.1% larger than the value based on allowable stress design. Also for $D/L = 0.5$, when 1/4 in. $\leq d <$ 1/2 in., the allowable shear load based on LRFD design is 18.2% larger than the value based on allowable stress design; when $d \geq$ 1/2 in., the allowable shear load based on LRFD design is 19.7% 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 load than allowable stress design for all three cases.

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:

$$V_T = V_{DL} + V_{LL} \tag{VII.84}$$

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. (VII.85) by using Eq. (II.6):

$$V_D = V_T \frac{1.2D/L+1.6}{D/L+1} \tag{VII.85}$$

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. (VII.65):

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = \frac{\phi\left(C_1 - D_1 f \dfrac{1.2D/L+1.6}{D/L+1}\right)}{C - Df} \; \frac{D/L+1}{1.2D/L+1.6} \qquad \text{(VII.86)}$$

where

$\phi = 0.75$

$f = V_T/A_b$

C and D are tabulated in Table VII.6

C_1 and D_1 are tabulated in Table VII.10

Figures 67 through 76 show the relationships between allowable load ratio and dead-to-live load ratio for tensile strength of bolts which are subject to a combination of shear and tension by using Eq. (VII.86).

Figure 67 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. For D/L ratio around 0.18, both design methods would result in the same allowable tensile load for the unfactored shear stresses f 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 = 21 ksi and 7 ksi, respectively.

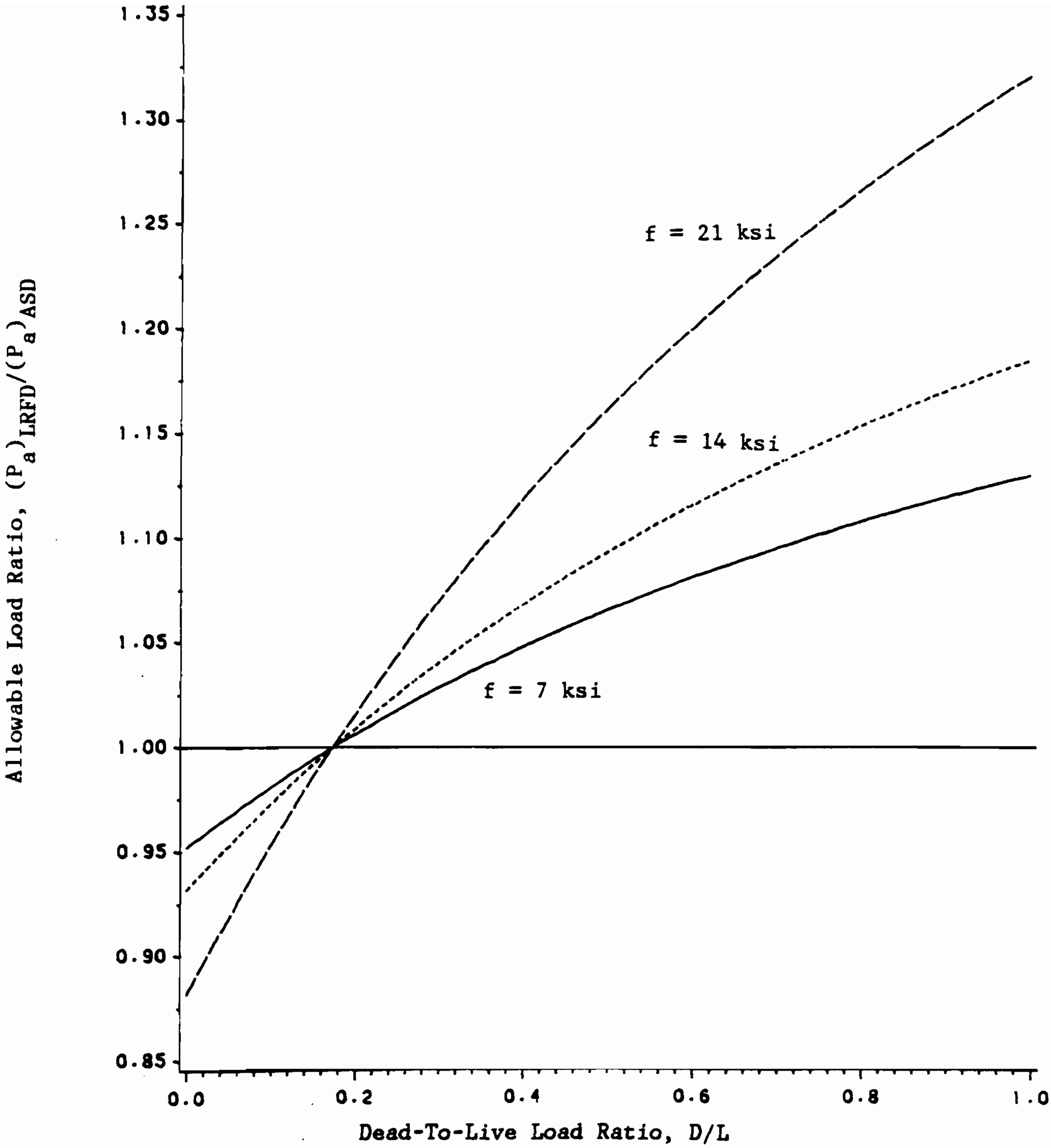

Figure 67. Allowable Load Ratio vs. D/L Ratio for Tension Strength on A325 Bolts Subject to the Combination of Shear and Tension - Case A

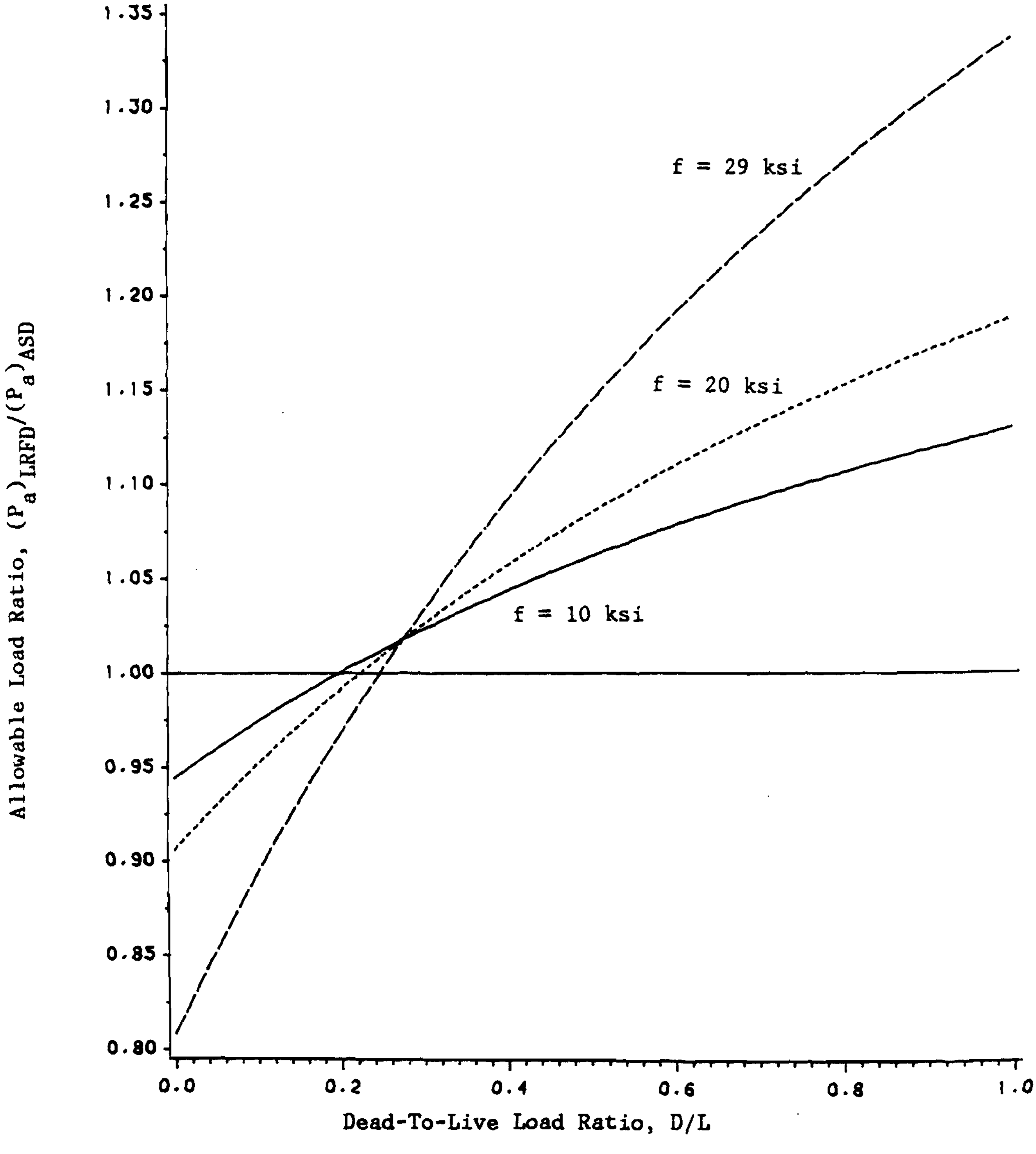

Figure 68. Allowable Load Ratio vs. D/L Ratio for Tension Strength on A325 Bolts Subject to the Combination of Shear and Tension - Case B

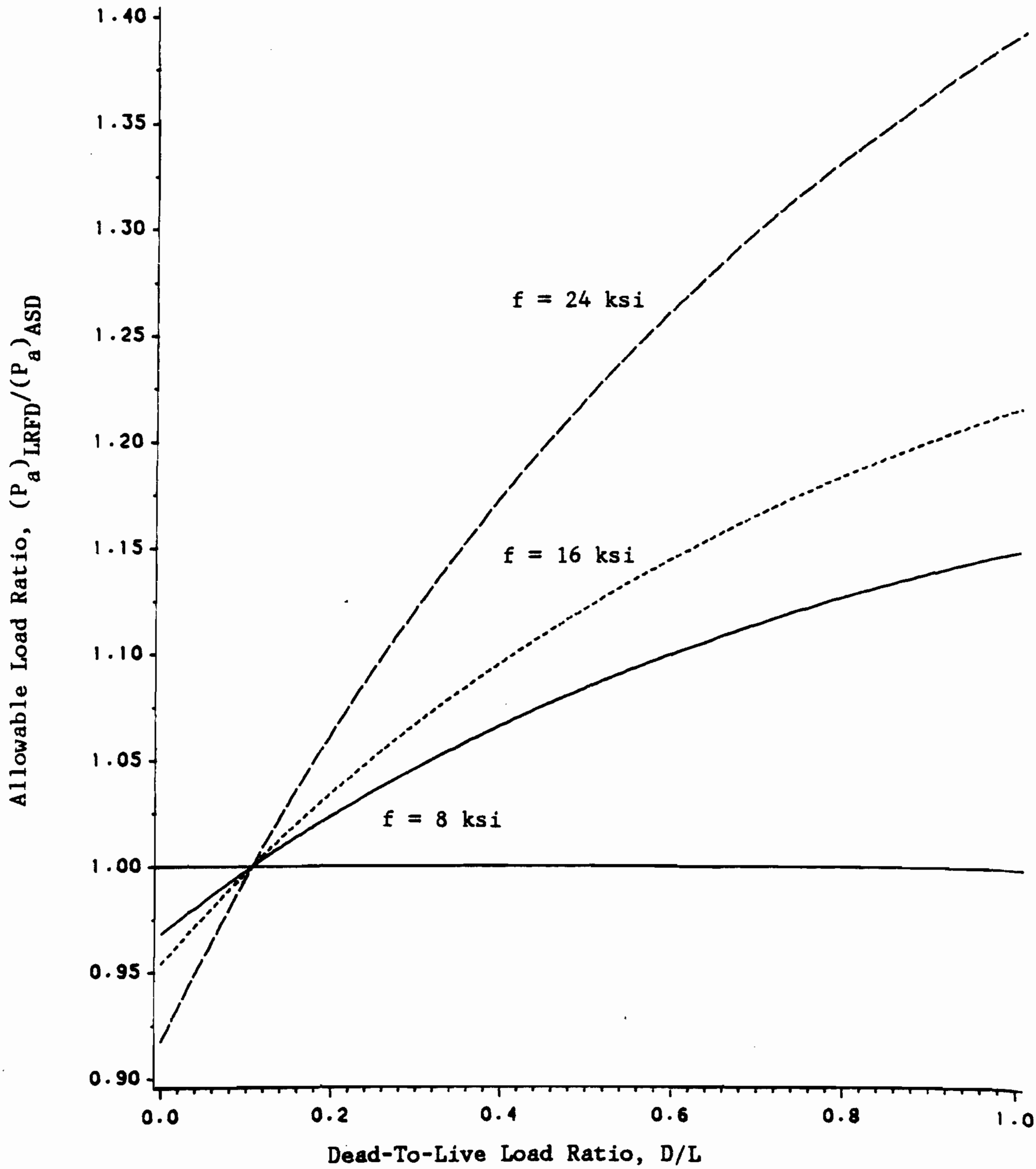

Figure 69. Allowable Load Ratio vs. D/L Ratio for Tension Strength on A354 Grade BD Bolts Subject to the Combination of Shear and Tension - Case A

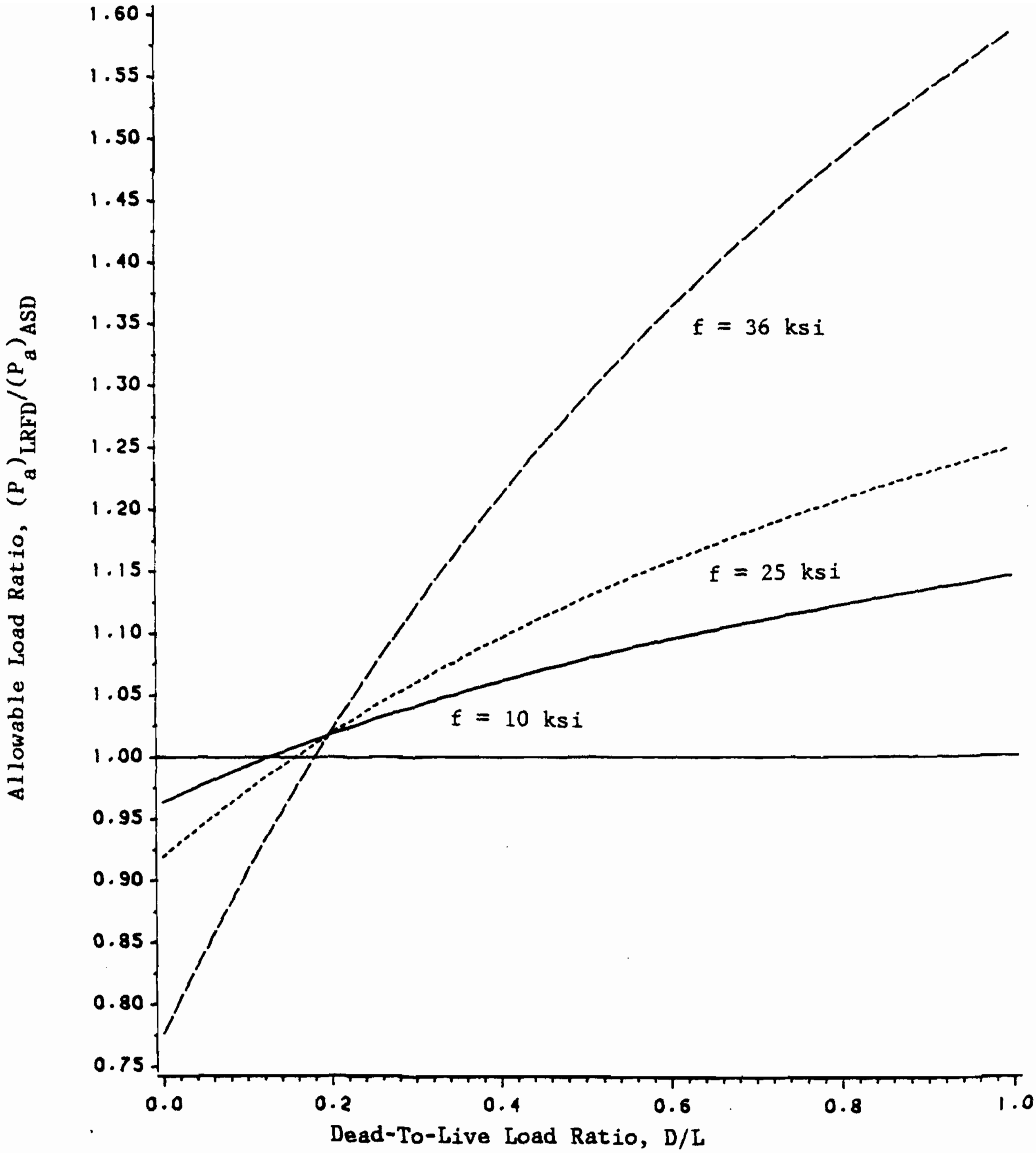

Figure 70. Allowable Load Ratio vs. D/L Ratio for Tension Strength on A354 Grade BD Bolts Subject to the Combination of Shear and Tension - Case B

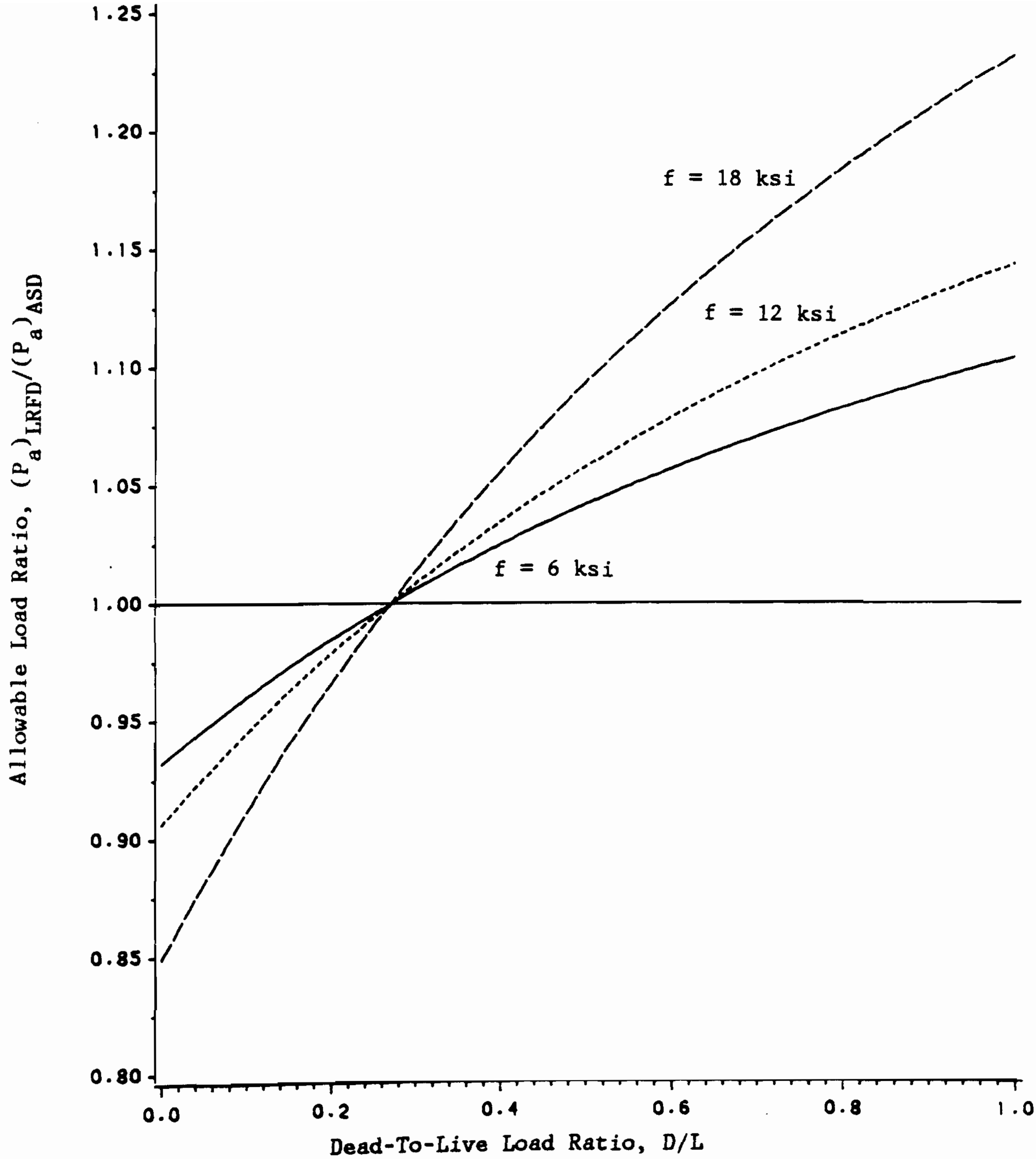

Figure 71. Allowable Load Ratio vs. D/L Ratio for Tension Strength on A449 Bolts Subject to the Combination of Shear and Tension - Case A

Figure 68 shows the allowable load ratio versus dead-to-live load ratio for A325 bolts when threads are excluded in shear plane. The different curves in this figure represent different unfactored shear stresses f. For D/L ratio around 0.28, the LRFD criteria gives an allowable load about 2% more than the value computed from allowable stress design for all unfactored shear stresses f shown in the figure. For D/L > 0.28, the larger the unfactored shear stress, the higher the allowable load ratio. For D/L = 0.5, the $(P_a)_{LRFD}/(P_a)_{ASD}$ ratios are 1.143 and 1.062 for f = 29 ksi and 10 ksi, respectively.

Figure 69 shows the allowable load ratio versus dead-to-live load ratio for A354 Grade BD bolts when threads are included in shear plane. The different curves in this figure represent different unfactored shear stresses f. For D/L ratio around 0.1, both design methods would result in the same allowable tensile load for the unfactored shear stresses f shown in the figure. For D/L > 0.1, the larger the unfactored shear stress, the higher the allowable load ratio. For D/L = 0.5, the $(P_a)_{LRFD}/(P_a)_{ASD}$ ratios are 1.222 and 1.085 for f = 24 ksi and 8 ksi, respectively.

Figure 70 shows the allowable load ratio versus dead-to-live load ratio for A354 Grade BD bolts when threads are excluded in shear plane. The different curves in this figure represent different unfactored shear stresses f. For D/L ratio around 0.2, the LRFD criteria give an allowable load about 2% more than the value computed from allowable stress design for all unfactored shear stresses f shown in the figure. For D/L > 0.2, the larger the unfactored shear stress, the higher the allowable load ratio. For D/L = 0.5, the $(P_a)_{LRFD}/(P_a)_{ASD}$ ratios are 1.287 and 1.079 for f = 36 ksi and 10 ksi, respectively.

Figure 71 shows the allowable load ratio versus dead-to-live load ratio for A449 bolts when threads are included in shear plane. The different curves in this figure represent different unfactored shear stresses f. For D/L ratio around 0.27, both design methods would result in the same allowable tensile load for the unfactored shear stresses f shown in the figure. For D/L > 0.27, the larger the unfactored shear stress, the higher the allowable load ratio. For D/L = 0.5, the $(P_a)_{LRFD}/(P_a)_{ASD}$ ratios are 1.094 and 1.042 for f = 18 ksi and 6 ksi, respectively.

Figure 72 shows the allowable load ratio versus dead-to-live load ratio for A449 bolts when threads are excluded in shear plane. The different curves in this figure represent different unfactored shear stresses f. For D/L ratio around 0.38, the LRFD criteria gives an allowable load about 2% more than the value computed from allowable stress design for all unfactored shear stresses f shown in the figure. For D/L > 0.38, the larger the unfactored shear stress, the higher the allowable load ratio. For D/L = 0.5, the $(P_a)_{LRFD}/(P_a)_{ASD}$ ratios are 1.098 and 1.039 for f = 29 ksi and 10 ksi, respectively.

Figure 73 shows the allowable load ratio versus dead-to-live load ratio for A490 bolts when threads are included in shear plane. The different curves in this figure represent different unfactored shear stresses f. For D/L ratio around 0.12, both design methods would result in the same allowable tensile load for the unfactored shear stresses f shown in the figure. For D/L > 0.12, the larger the unfactored shear stress, the higher the allowable load ratio. For D/L = 0.5, the $(P_a)_{LRFD}/(P_a)_{ASD}$ ratios are 1.211 and 1.079 for f = 27 ksi and 9 ksi, respectively.

Figure 74 shows the allowable load ratio versus dead-to-live load ratio for A490 bolts when threads are excluded in shear plane. The different curves in this figure represent different unfactored shear stresses f. For D/L ratio around 0.22, the LRFD criteria give an allowable load about 2% more than the value computed from allowable stress design for all unfactored shear stresses f shown in the figure. For D/L > 0.22, the larger the unfactored shear stress, the higher the allowable load ratio. For D/L = 0.5, the $(P_a)_{LRFD}/(P_a)_{ASD}$ ratios are 1.182 and 1.074 for f = 36 ksi and 12 ksi, respectively.

Figure 75 shows the allowable load ratio versus dead-to-live load ratio for A307 Grade A bolts when 1/4 in. ≤ d < 1/2 in.. The different curves in this figure represent different unfactored shear stresses f. For D/L ratio around 0.2, both design methods would result in the same allowable tensile load for the unfactored shear stresses f shown in the figure. For D/L > 0.2, the larger the unfactored shear stress, the higher the allowable load ratio. For D/L = 0.5, the $(P_a)_{LRFD}/(P_a)_{ASD}$ ratios are 1.152 and 1.059 for f = 9 ksi and 3 ksi, respectively.

Figure 76 shows the allowable load ratio versus dead-to-live load ratio for A307 Grade A bolts when d ≥ 1/2 in.. The different curves in this figure represent different unfactored shear stresses f. For D/L ratio around 0.32, both design methods would result in the same allowable tensile load for the unfactored shear stresses f shown in the figure. For D/L > 0.32, the larger the unfactored shear stress, the higher the allowable load ratio. For D/L = 0.5, the $(P_a)_{LRFD}/(P_a)_{ASD}$ ratios are 1.074 and 1.030 for f = 10 ksi and 3.5 ksi, respectively.

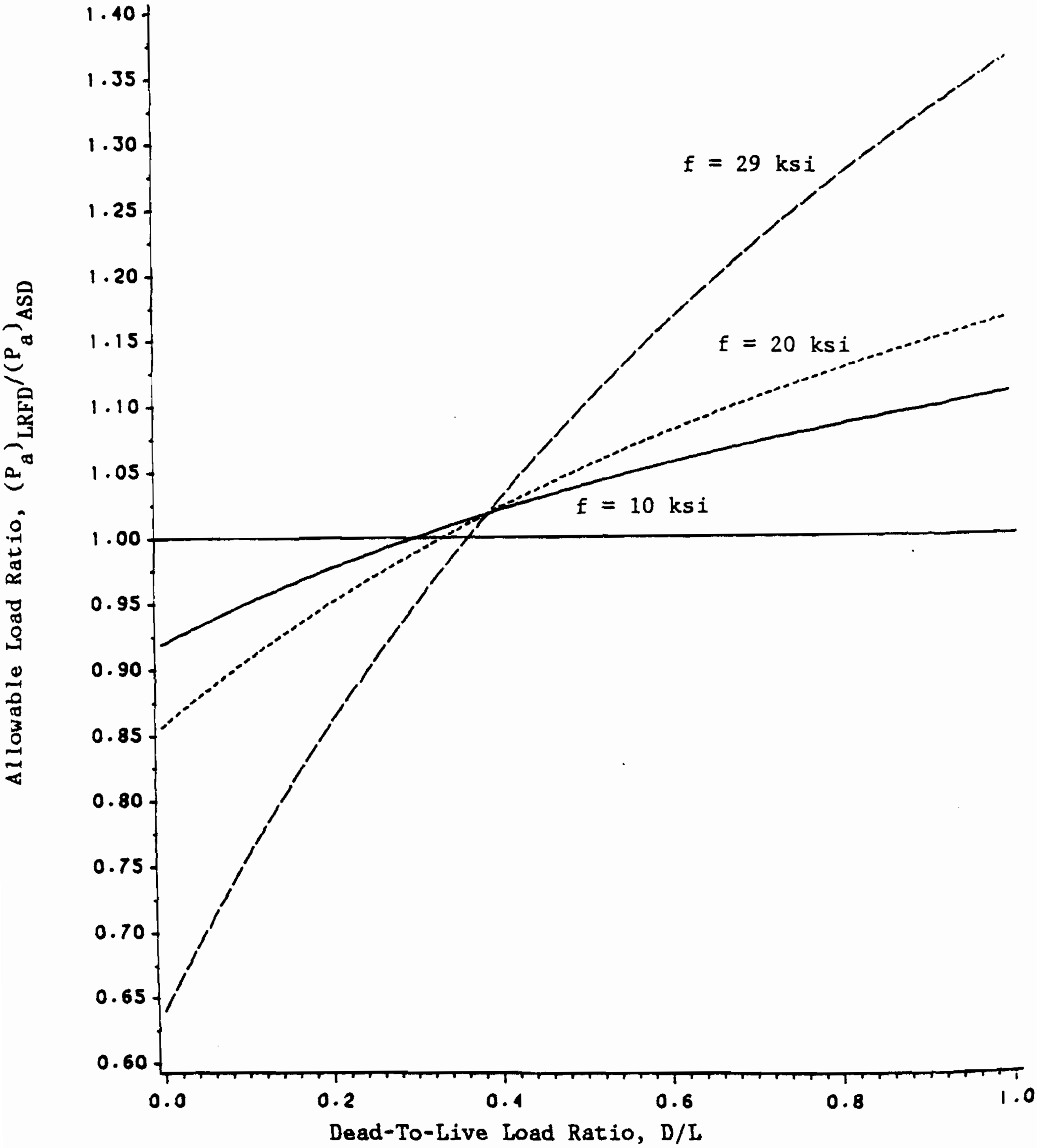

Figure 72. Allowable Load Ratio vs. D/L Ratio for Tension Strength on A449 Bolts Subject to the Combination of Shear and Tension - Case B

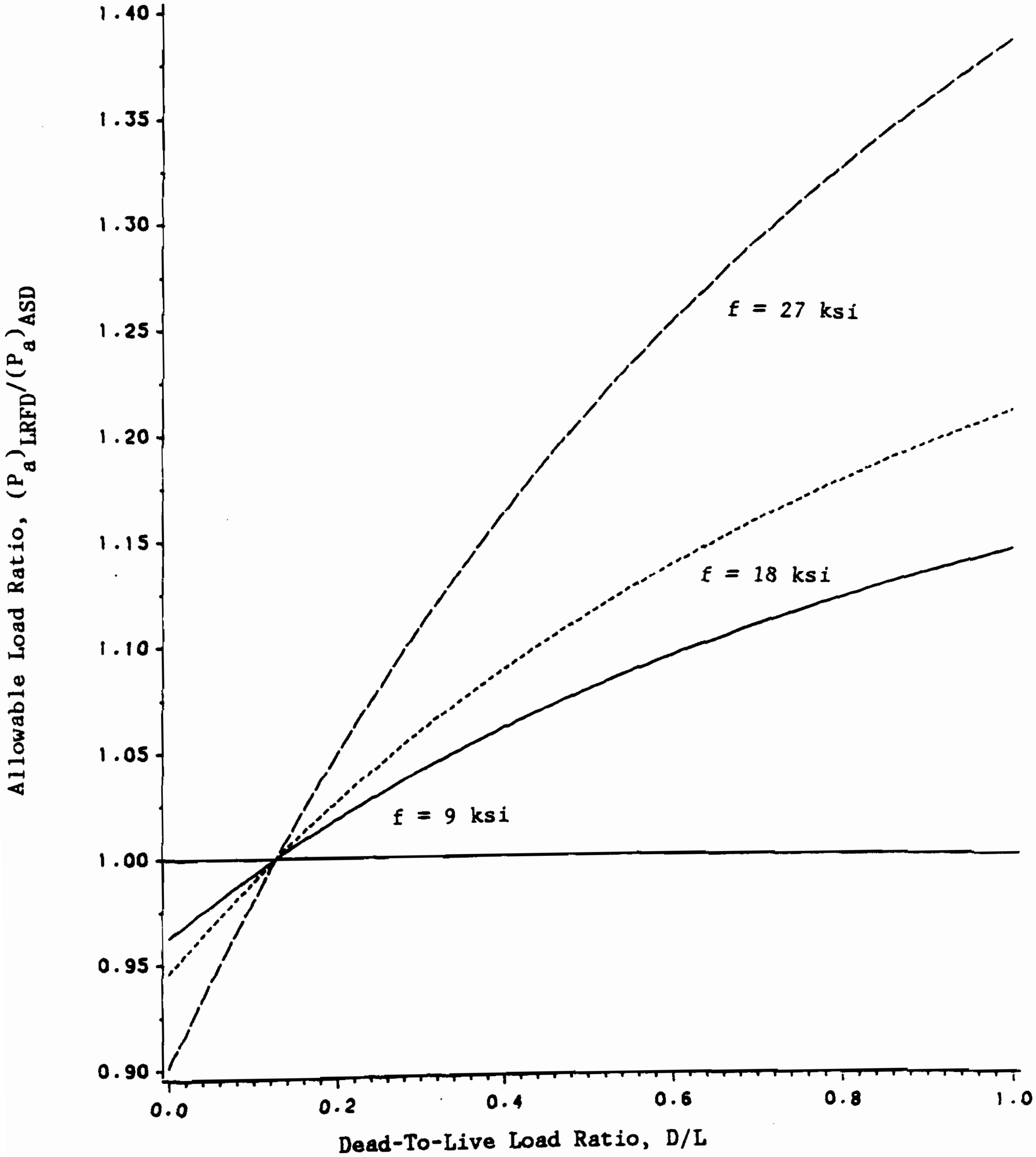

Figure 73. Allowable Load Ratio vs. D/L Ratio for Tension Strength on A490 Bolts Subject to the Combination of Shear and Tension - Case A

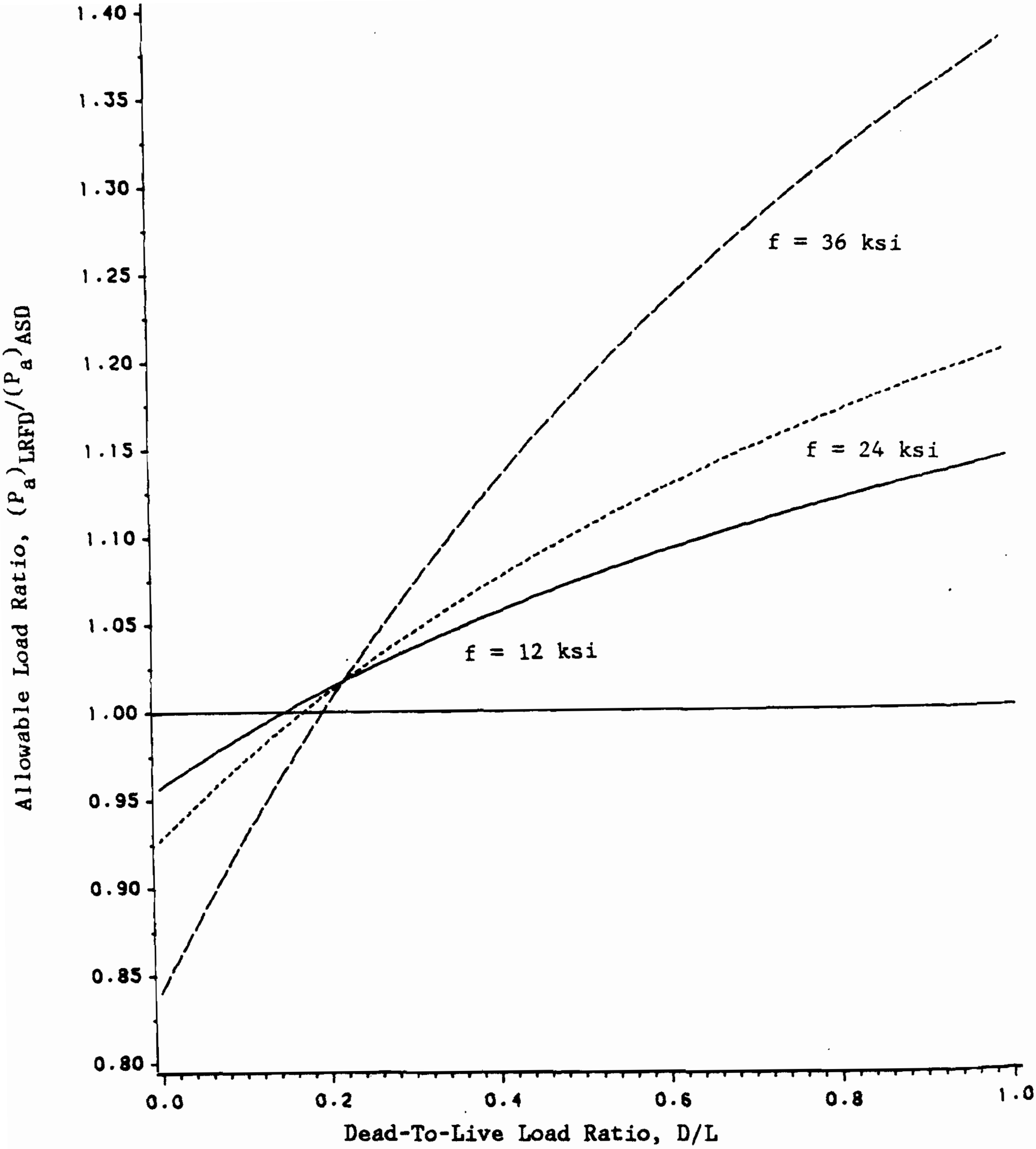

Figure 74. Allowable Load Ratio vs. D/L Ratio for Tension Strength on A490 Bolts Subject to the Combination of Shear and Tension - Case B

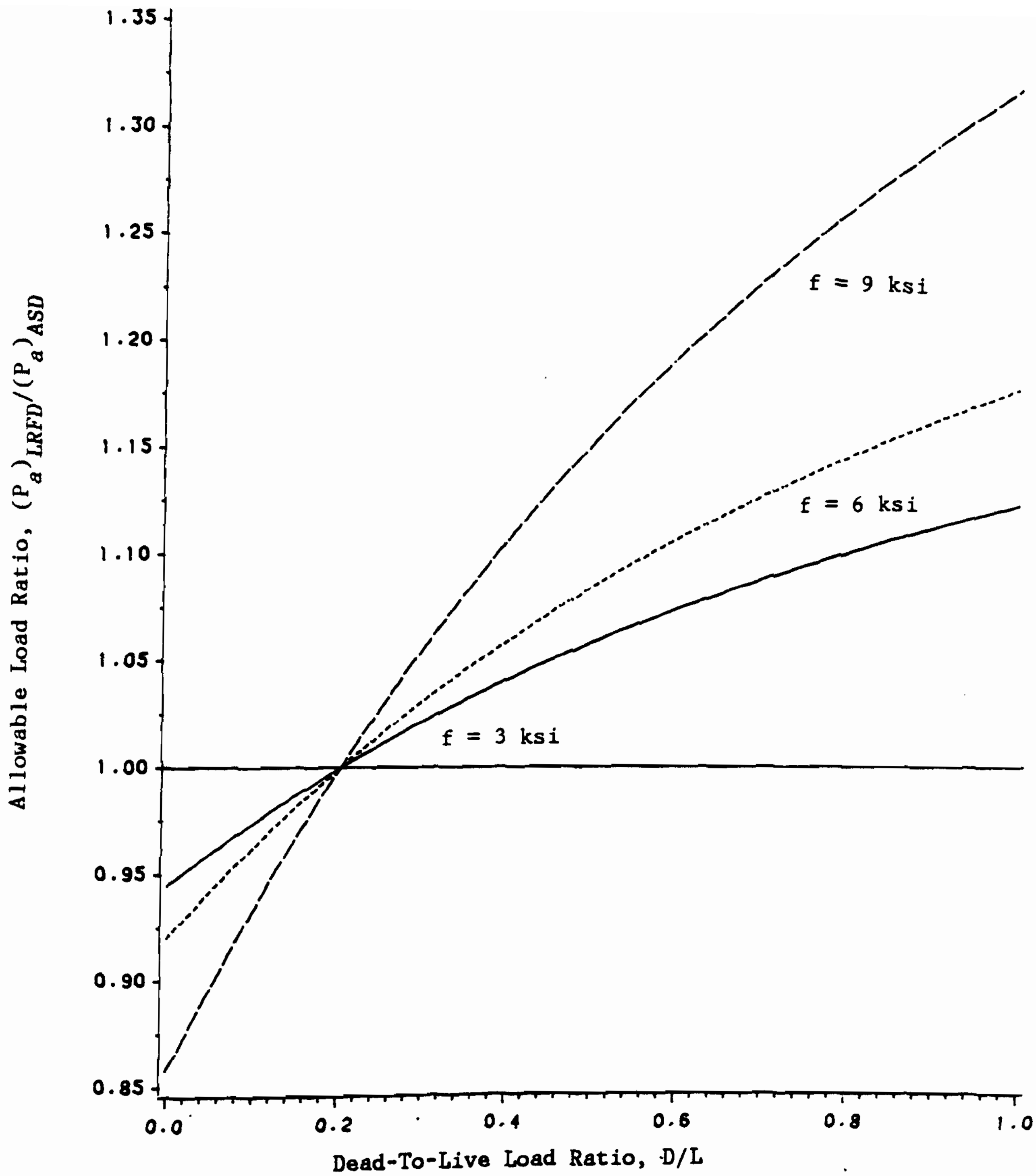

Figure 75. Allowable Load Ratio vs. D/L Ratio for Tension Strength on A307 Bolts, Grade A, Subject to the Combination of Shear and Tension (1/4 in. $\leq$ d < 1/2 in.)

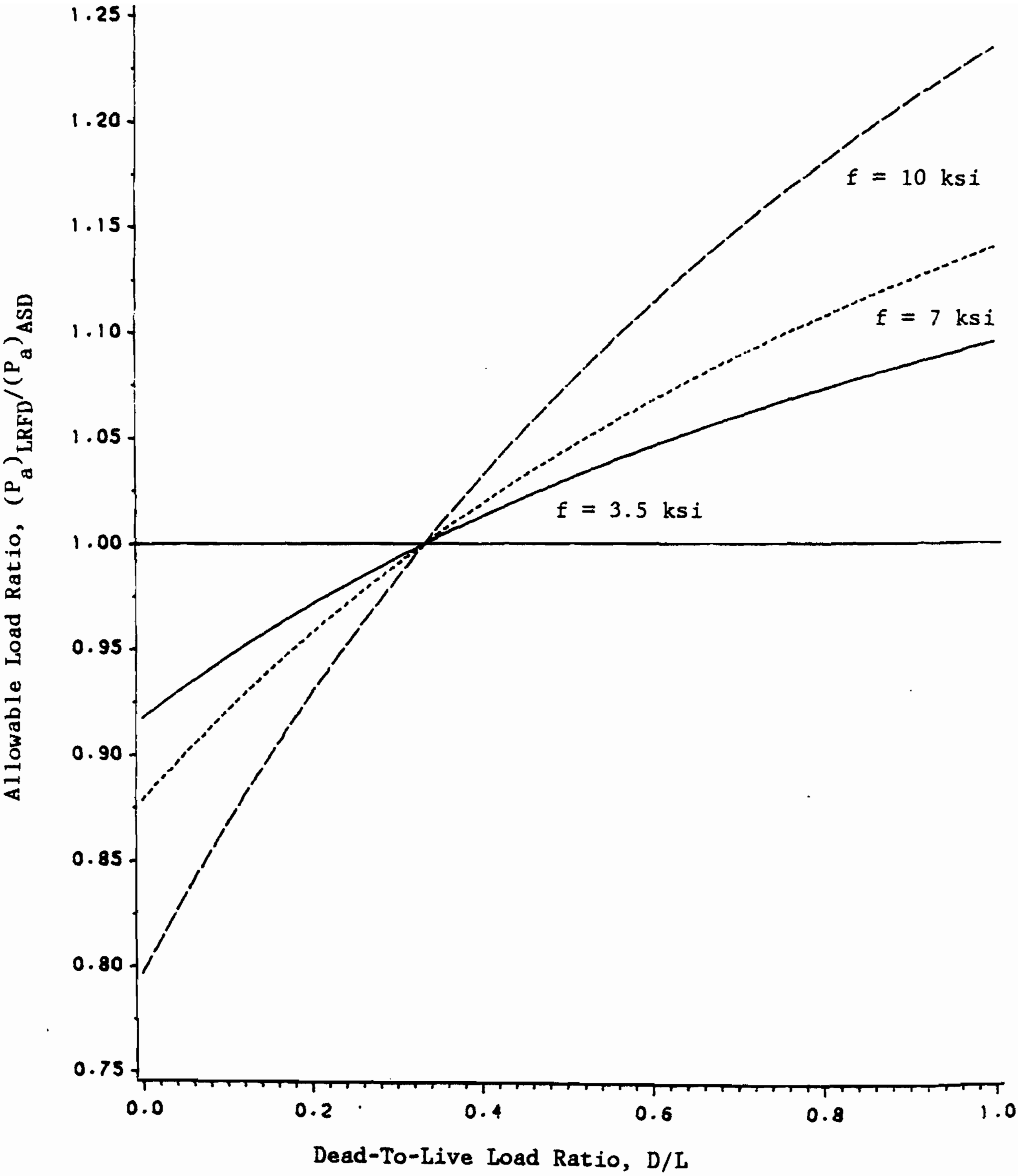

Figure 76. Allowable Load Ratio vs. D/L Ratio for Tension Strength on A307 Bolts, Grade A, Subject to the Combination of Shear and Tension (d ≥ 1/2 in.)

VIII. STIFFENERS

A. TRANSVERSE STIFFENERS

1. Allowable Stress Design. According to Section B6.1 of the AISI Specification, 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. Means for shear transfer between the stiffener and the web shall be provided according to Chapter E of the AISI Specification. The concentrated loads or reactions shall not exceed the smaller of the allowable loads, P_a, given by (a) and (b) as follows:

(a) $P_a = P_n/\Omega_{st}$ (VIII.1)

where

$P_n = F_{wy}A_c$ (VIII.2)

$\Omega_{st} = 2.00$

$A_c = 18t^2+A_s$, for transverse stiffeners at interior support and under concentrated load (VIII.3)

$A_c = 10t^2+A_s$, for transverse stiffeners at end support(VIII.4)

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

(b) $P_a = P_n/\Omega_c$ (VIII.5)

where

P_n = Nominal axial load evaluated according to Section C4(a) of the AISI Specification with A_e replaced by A_b

Ω_c = Factor of safety for axial compression evaluated according to Section C4(a) of the AISI Specification

A_b = b_1t+A_s, for transverse stiffeners at interior support and under concentrated load (VIII.6)

A_b = b_2t+A_s, for transverse stiffeners at end support (VIII.7)

A_s = Cross sectional area of transverse stiffeners

b_1 = $25t[0.0024(L_{st}/t)+0.72] \leq 25t$ (VIII.8)

b_2 = $12t[0.0044(L_{st}/t)+0.83] \leq 12t$ (VIII.9)

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, F_y, and t_s the thickness of the stiffener steel.

2. LRFD Criteria. According to Section B6.1 of the LRFD Specification, 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. Means for shear transfer between the stiffener and the web shall be provided according to Chapter E of the LRFD Specification. The concentrated loads or reactions determined on the basis of factored loads shall not exceed the factored nominal strength, ϕ_cP_n, where ϕ_c = 0.85 and P_n is the smaller value given by provisions

(a) and (b) in Section B6.1 of the LRFD Specification which are the same as those specified in Section B6.1 of the AISI Specification mentioned above.

3. <u>Comparison.</u> The unfactored load applied to the stiffener can be computed for both design methods by using the following formula:

$$P_T = P_{DL}+P_{LL} \tag{VIII.10}$$

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

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

$$(P_a)_{ASD1} = P_{n1}/\Omega_{st} \tag{VIII.11}$$

$$(P_a)_{ASD2} = P_{n2}/\Omega_c \tag{VIII.12}$$

For LRFD, the allowable axial loads can be computed by using the following equations developed from Eq. (II.6):

$$(P_a)_{LRFD1} = \phi_c P_{n1}(D/L+1)/(1.2D/L+1.6) \tag{VIII.13}$$

$$(P_a)_{LRFD2} = \phi_c P_{n2}(D/L+1)/(1.2D/L+1.6) \tag{VIII.14}$$

where

P_{n1} = Nominal compression strength specified in provision (a) of Section B6.1

P_{n2} = Nominal compression strength specified in provision (b) of Section B6.1

In order to study the allowable load ratios, three different cases were considered:

(1) Case 1: $P_{n1} \le P_{n2}$, then Eqs. (VIII.11) and (VIII.13) can be used to determine the allowable load ratio as follow:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = \Omega_{st}\Phi_c \frac{D/L+1}{1.2D/L+1.6} = 1.7 \frac{D/L+1}{1.2D/L+1.6} \qquad \text{(VIII.15)}$$

(2) Case 2: $P_{n1} > P_{n2}$ and $P_{n1}/\Omega_{st} > P_{n2}/\Phi_c$, then Eqs. (VIII.12) and (VIII.14) can be used to determine the allowable load ratio as follow:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = \Omega_c\Phi_c \frac{D/L+1}{1.2D/L+1.6} = 0.85\Omega_c \frac{D/L+1}{1.2D/L+1.6} \qquad \text{(VIII.16)}$$

where

$$\Omega_c = 5/3+(3/8)R-(1/8)R^3 \qquad \text{(VIII.17)}$$

$$R = \sqrt{(F_y/2F_e)} \qquad \text{(VIII.18)}$$

(3) Case 3: $P_{n1} > P_{n2}$ and $P_{n1}/\Omega_{st} < P_{n2}/\Omega_c$, then Eqs. (VIII.11) and (VIII.14) can be used to determine the allowable load ratio as follow:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = \Omega_{st}\Phi_c \frac{P_{n2}}{P_{n1}} \left[\frac{D/L+1}{1.2D/L+1.6} \right] = 1.7 \frac{P_{n2}}{P_{n1}} \left[\frac{D/L+1}{1.2D/L+1.6} \right] \text{(VIII.19)}$$

Figure 77 shows the allowable load ratio versus dead-to-live load ratio for the compression strength of transverse stiffeners determined by Eq. (VIII.15). 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.

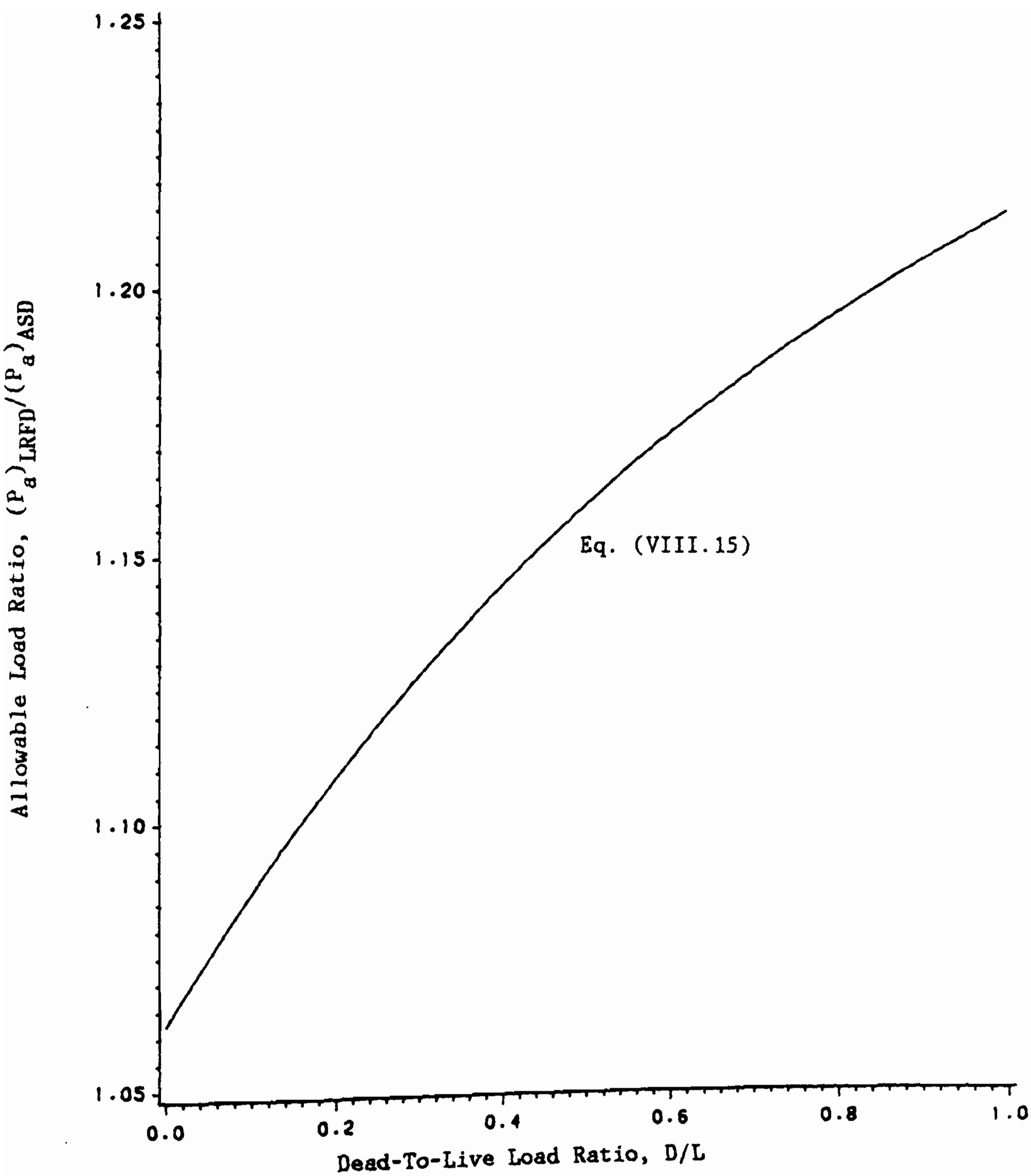

Figure 77. Allowable Load Ratio vs. D/L Ratio for Compression Strength of Transverse Stiffeners-Case 1

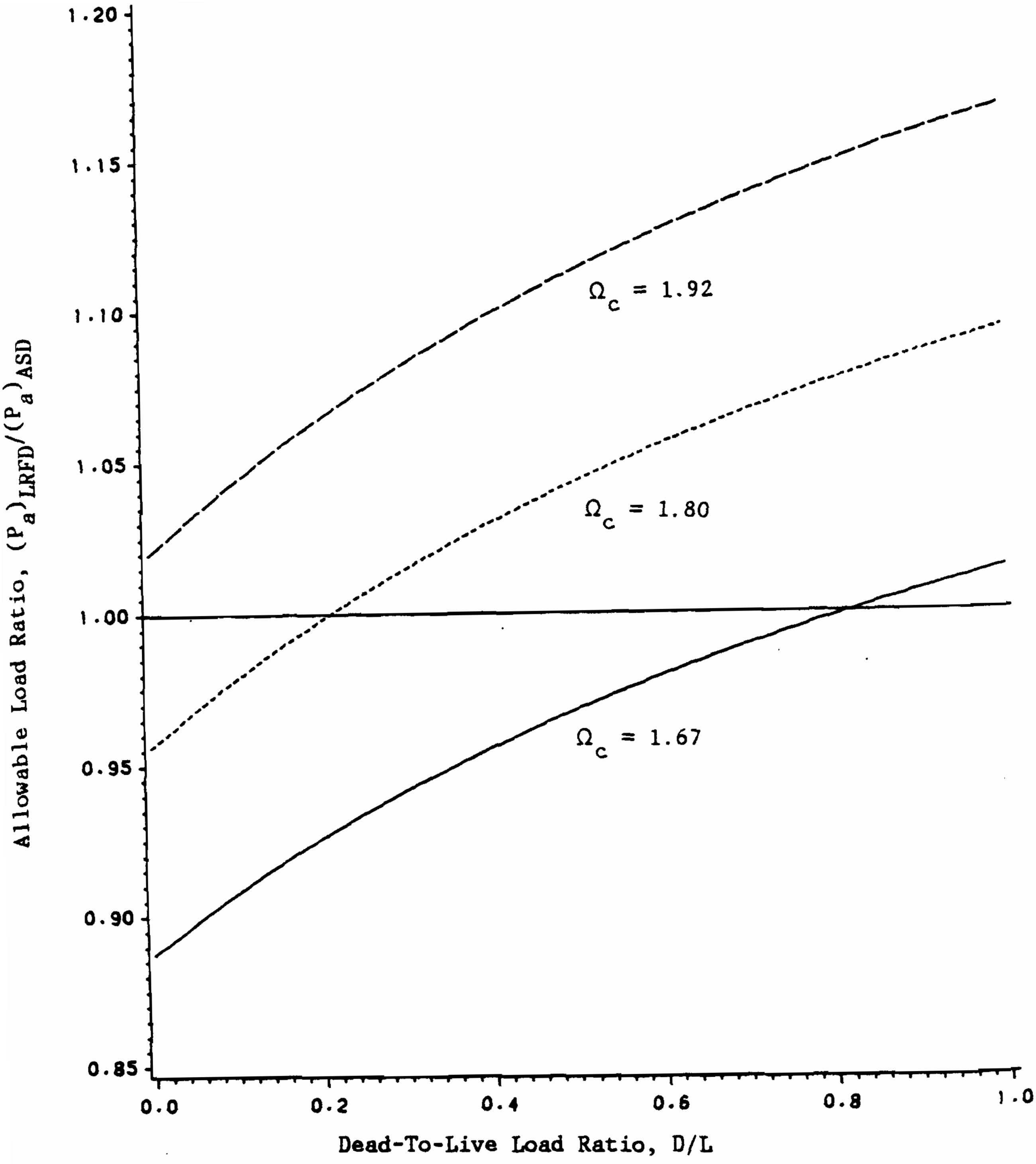

Figure 78. Allowable Load Ratio vs. D/L Ratio for Compression Strength of Transverse Stiffeners-Case 2

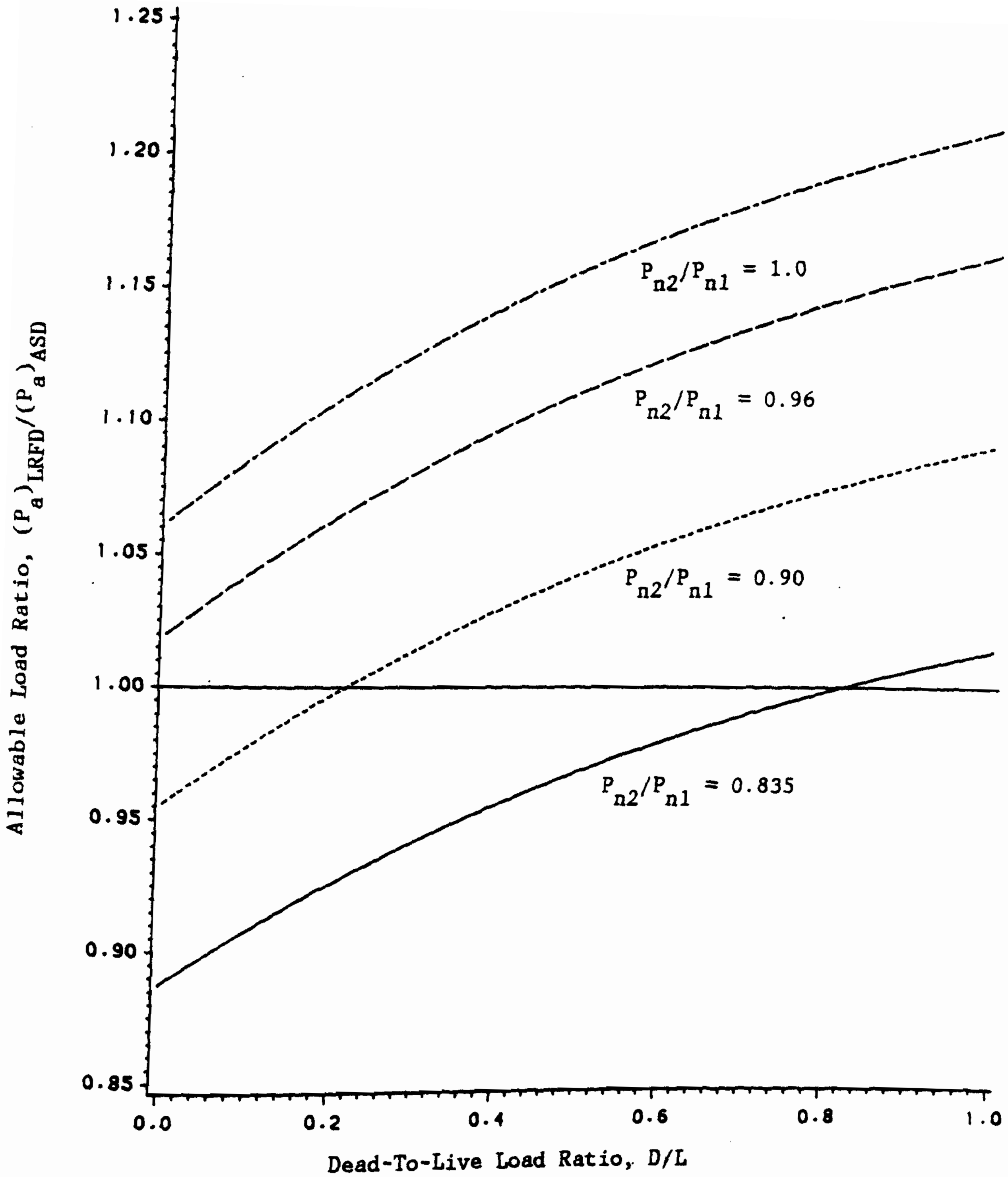

Figure 79. Allowable Load Ratio vs. D/L Ratio for Compression Strength of Transverse Stiffeners-Case 3

Figure 78 shows the allowable load ratio versus dead-to-live load ratio for the compression strength of transverse stiffeners determined by Eq. (VIII.16). 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 79 shows the allowable load ratio versus dead-to-live load ratio for the compression strength of transverse stiffeners determined by Eq. (VIII.19). 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.

B. SHEAR STIFFENERS

1. Allowable Stress Design. According to Section B6.2 of the AISI Specification, where shear stiffeners are required, the spacing shall be such that the web shear force shall not exceed the allowable shear force, V_a, permitted by Section C3.2 of the AISI Specification, 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

$$I_{smin} = 5ht^3[h/a-0.7(a/h)] \geq (h/50)^4 \qquad \text{(VIII.20)}$$

The gross area of shear stiffeners shall be not less than

$$A_{st} = [(1-C_v)/2]\{a/h-(a/h)^2/[(a/h)+\sqrt{1+(a/h)^2}]\}YDht \qquad \text{(VIII.21)}$$

where

$$C_v = 45{,}000k_v/[F_y(h/t)^2] \text{ when } C_v \leq 0.8 \tag{VIII.22}$$

$$C_v = [190/(h/t)](\sqrt{k_v/F_y}) \text{ when } C_v > 0.8 \tag{VIII.23}$$

$$k_v = 4.00 + 5.34/(a/h)^2 \text{ when } a/h \leq 1.0 \tag{VIII.24}$$

$$k_v = 5.34 + 4.00/(a/h)^2 \text{ when } a/h > 1.0 \tag{VIII.25}$$

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

2. LRFD Criteria. According to Section B6.2 of the LRFD Specification, where shear stiffeners are required, the spacing shall be such that the web shear force determined on the basis of factroed loads shall not exceed the factored nominal shear strength, $\phi_v V_n$, permitted by Section C3.2, and the ratio a/h shall not exceed $[260/(h/t)]^2$ nor 3.0.

The requirements for the actual moment of inertia and gross area of shear stiffeners are the same as those specified in Section B6.2 of the AISI Specification.

3. Comparison. The unfactored shear force can be calculated for both ASD and LRFD methods by using the following equation.

$$V_T = V_{DL}+V_{LL} \tag{VIII.26}$$

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

$$(V_a)_{ASD} = V_n/\Omega \qquad \text{(VIII.27)}$$

For LRFD, the allowable shear load equation was developed from Eq. (II.6) and is

$$(V_a)_{LRFD} = \phi_v V_n (D/L+1)/(1.2D/L+1.6) \qquad \text{(VIII.28)}$$

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. For example, 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.

The allowable shear ratios are:

For $h/t \leq \sqrt{Ek_v/F_y}$ and $\phi_v = 1.0$,

$$\frac{(V_a)_{LRFD}}{(V_a)_{ASD}} = 1.443\phi_v \frac{D/L+1}{1.2D/L+1.6} = 1.443 \frac{D/L+1}{1.2D/L+1.6} \qquad \text{(VIII.29)}$$

For $\sqrt{Ek_v/F_y} < h/t \leq 1.38\sqrt{Ek_v/F_y}$ and $\phi_v = 0.90$

$$\frac{(V_a)_{LRFD}}{(V_a)_{ASD}} = 1.674\phi_v \frac{D/L+1}{1.2D/L+1.6} = 1.507 \frac{D/L+1}{1.2D/L+1.6} \qquad \text{(VIII.30)}$$

For $h/t > 1.415\sqrt{Ek_v/F_y}$ and $\phi_v = 0.90$

$$\frac{(V_a)_{LRFD}}{(V_a)_{ASD}} = 1.712\phi_v \frac{D/L+1}{1.2D/L+1.6} = 1.541 \frac{D/L+1}{1.2D/L+1.6} \qquad \text{(VIII.31)}$$

Figure 80 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 5 which was obtained for the shear strength of the beam webs.

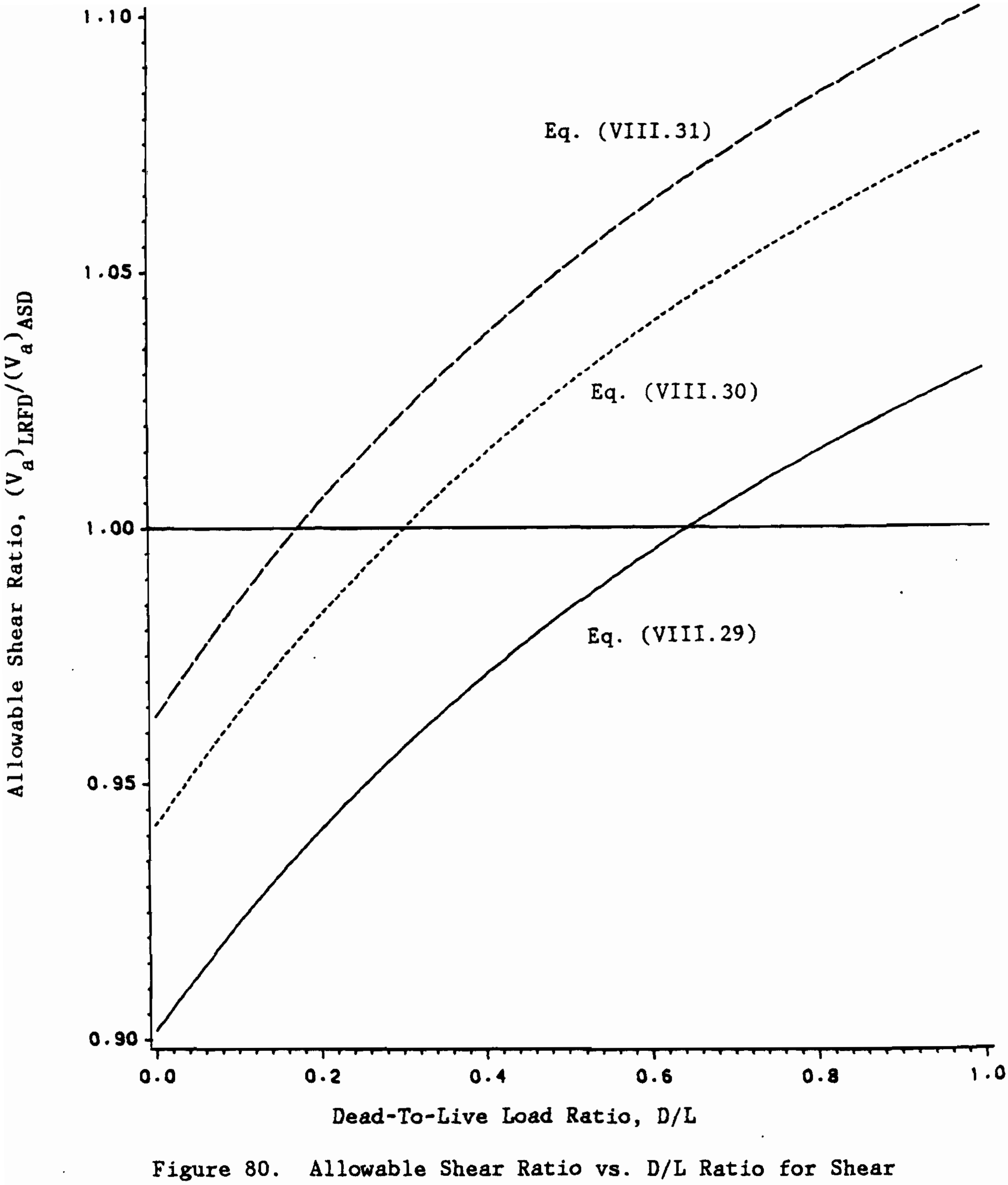

Figure 80. Allowable Shear Ratio vs. D/L Ratio for Shear Strength of Shear Stiffeners

IX. WALL STUDS AND WALL STUD ASSEMBLIES

A. WALL STUDS IN COMPRESSION

1. Allowable Stress Design. According to Section D4.1 of the AISI Specification, for studs having identical sheathing attached to both flanges, and neglecting any rotational restraint provided by the sheathing, the applied axial load, P, shall not exceed P_a calculated as follows:

$$P_a = A_e F_n / \Omega_c \qquad \text{(IX.1)}$$

where

A_e = Effective area determined at F_n

Ω_c = Factor of safety for axial compression

F_n = The lowest value determined by the following three conditions:

(a) To prevent column buckling between fasteners in the plane of the wall, F_n shall be calculated according to Section C4 of the AISI Specification 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 C4 of the AISI Specification 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

$$\sigma_{CR} = \sigma_{ey} + \bar{Q}_a \qquad \text{(IX.2)}$$

$$\sigma_{CR} = 1/(2\beta)[(\sigma_{ex}+\sigma_{tQ}) - \sqrt{(\sigma_{ex}+\sigma_{tQ})^2-(4\beta\sigma_{ex}\sigma_{tQ})}] \qquad \text{(IX.3)}$$

(2) Z-Sections

$$\sigma_{CR} = \sigma_t+\bar{Q}_t \qquad \text{(IX.4)}$$

$$\sigma_{CR} = 1/2\left\{(\sigma_{ex}+\sigma_{ey}+\bar{Q}_a)-[(\sigma_{ex}+\sigma_{ey}+\bar{Q}_a)^2-4(\sigma_{ex}\sigma_{ey}+\sigma_{ex}\bar{Q}_a-\sigma_{exy}^2)]^{1/2}\right\} \qquad \text{(IX.5)}$$

(3) I-Sections (doubly-symmetric)

$$\sigma_{CR} = \sigma_{ey}+\bar{Q}_a \qquad \text{(IX.6)}$$

$$\sigma_{CR} = \sigma_{ex} \qquad \text{(IX.7)}$$

In the above formulas

$$\sigma_{ex} = \pi^2E/(K_xL_x/r_x)^2 \qquad \text{(IX.8)}$$

$$\sigma_{exy} = (\pi^2EI_{xy})/(AL^2) \qquad \text{(IX.9)}$$

$$\sigma_{ey} = \pi^2E/(K_yL_y/r_y)^2 \qquad \text{(IX.10)}$$

$$\sigma_t = 1/(Ar_o^2)[GJ+\pi^2EC_w/(K_tL_t)^2] \qquad \text{(IX.11)}$$

$$\sigma_{tQ} = \sigma_t+\bar{Q}_t \qquad \text{(IX.12)}$$

$\bar{Q}$ = $\bar{q}B$ = Design shear rigidity for sheathing on both sides of the wall assembly (IX.13)

$\bar{q}$ = Design shear rigidity for sheathing per inch of stud spacing (see Table IX.1)

B = Stud spacing

$$\bar{Q}_a = \bar{Q}/A \qquad \text{(IX.14)}$$

A = Area of full unreduced cross section

L = Length of stud

$$\bar{Q}_t = (\bar{Q}d^2)/(4Ar_0^2) \qquad \text{(IX.15)}$$

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:

$$\gamma = (\pi/L)[C_1+(E_1 d/2)] \qquad \text{(IX.16)}$$

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

$$C_1 = (F_n C_o)/(\sigma_{ey}-F_n+\bar{Q}_a) \qquad \text{(IX.17)}$$

$$E_1 = \frac{F_n[(\sigma_{ex}-F_n)(r_o^2 E_o-x_o D_o)-F_n x_o(D_o-x_o E_o)]}{(\sigma_{ex}-F_n)r_o^2(\sigma_{tQ}-F_n)-(F_n x_o)^2} \qquad \text{(IX.18)}$$

(2) Z-Sections

$$C_1 = \frac{F_n[C_o(\sigma_{ex}-F_n)-D_o\sigma_{exy}]}{(\sigma_{ey}-F_n+\bar{Q}_a)(\sigma_{ex}-F_n)-\sigma_{exy}^2} \qquad \text{(IX.19)}$$

$$E_1 = (F_n E_o)/(\sigma_{tQ}-F_n) \qquad \text{(IX.20)}$$

(3) I-Sections

$$C_1 = (F_n C_o)/(\sigma_{ey}-F_n+\bar{Q}_a) \qquad \text{(IX.21)}$$

$$E_1 = 0$$

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 (IX.22)

D_o = L/700 in a direction perpendicular to the wall (IX.23)

E_o = L/(dx10,000), rad., a measure of the initial twist of the stud from the initial, ideal, unbuckled shape. (IX.24)

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

$$E' = 4EF_n(F_y - F_n)/F_y^2 \tag{IX.25}$$

$$G' = G(E'/E) \tag{IX.26}$$

Sheathing parameters $\bar{q}_o$ and $\bar{\gamma}$ may be determined from representative full-scale tests, conducted and evaluated as described by published documented methods, or from the small-scale-test values given in Table IX.1

TABLE IX.1
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) $$\bar{q} = \bar{q}_o(2 - s/12) \tag{IX.27}$$

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 as described by published documented methods.

2. LRFD Criteria. According to Section D4.1 of the LRFD Specification, for studs having identical sheathing attached to both flanges, and neglecting any rotational restraint provided by the sheathing, the factored nominal axial strength, ϕP_n, shall be calculated as follows:

$$\phi = 0.85$$

$$P_n = A_e F_n \qquad \text{(IX.28)}$$

P_n is the same as that specified in Section D4.1 of the AISI Specification.

3. Comparison. The unfactored load applied to the member can be computed for both design methods by using the following formula:

$$P_T = P_{DL}+P_{LL} \qquad \text{(IX.29)}$$

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

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

$$(P_a)_{ASD} = P_n/\Omega_c \qquad \text{(IX.30)}$$

For LRFD, the allowable axial load can be computed by using the following equation developed from Eq. (II.6):

$$(P_a)_{LRFD} = \phi_c P_n(D/L+1)/(1.2D/L+1.6) \qquad \text{(IX.31)}$$

Then, the allowable load ratio can be determined as follows:

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = \frac{\Phi_c P_n}{P_n/\Omega_c}\left[\frac{D/L+1}{1.2D/L+1.6}\right] = 0.85\Omega_c \frac{D/L+1}{1.2D/L+1.6} \qquad \text{(IX.32)}$$

For fully effective sections having wall thickness greater than 0.09 in. and $F_e > F_y/2$,

$$\Omega_c = 5/3+(3/8)R-(1/8)R^3$$

Therefore, the allowable load ratio is

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 0.85\left(\frac{5}{3} + \frac{3}{8}R - \frac{1}{8}R^3\right)\frac{D/L+1}{1.2D/L+1.6} \qquad \text{(IX.33)}$$

For all other cases, $\Omega_c = 1.92 = 23/12$, therefore the allowable load ratio is

$$\frac{(P_a)_{LRFD}}{(P_a)_{ASD}} = 0.85(23/12)\frac{D/L+1}{1.2D/L+1.6} = 1.629\frac{D/L+1}{1.2D/L+1.6} \qquad \text{(IX.34)}$$

Figure 81 shows the allowable load ratio versus dead-to-live load ratio for the wall studs used to develop Eq. (IX.34). 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.

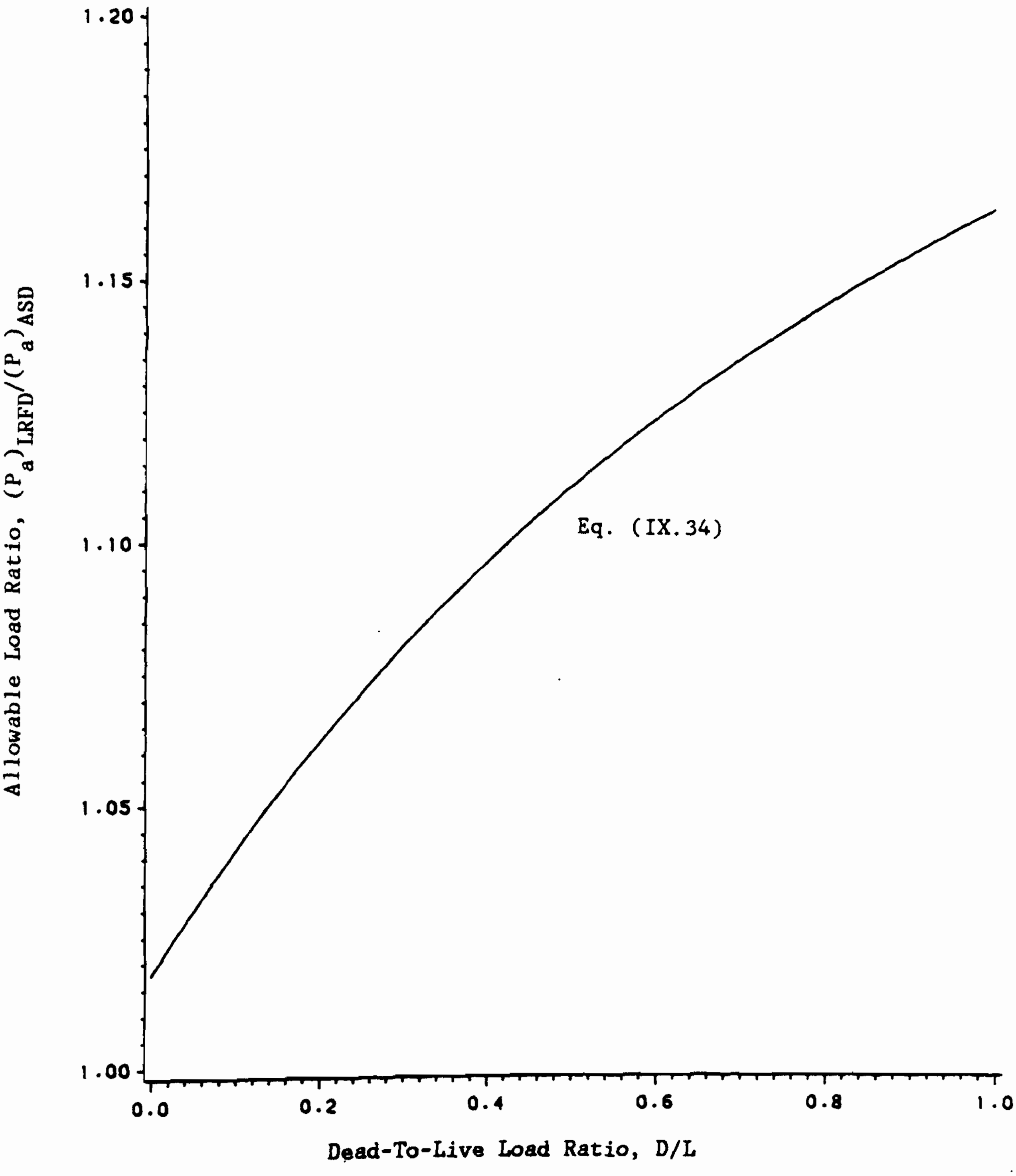

Figure 81. Allowable Load Ratio vs. D/L Ratio for Wall Studs in Compression-Case 1

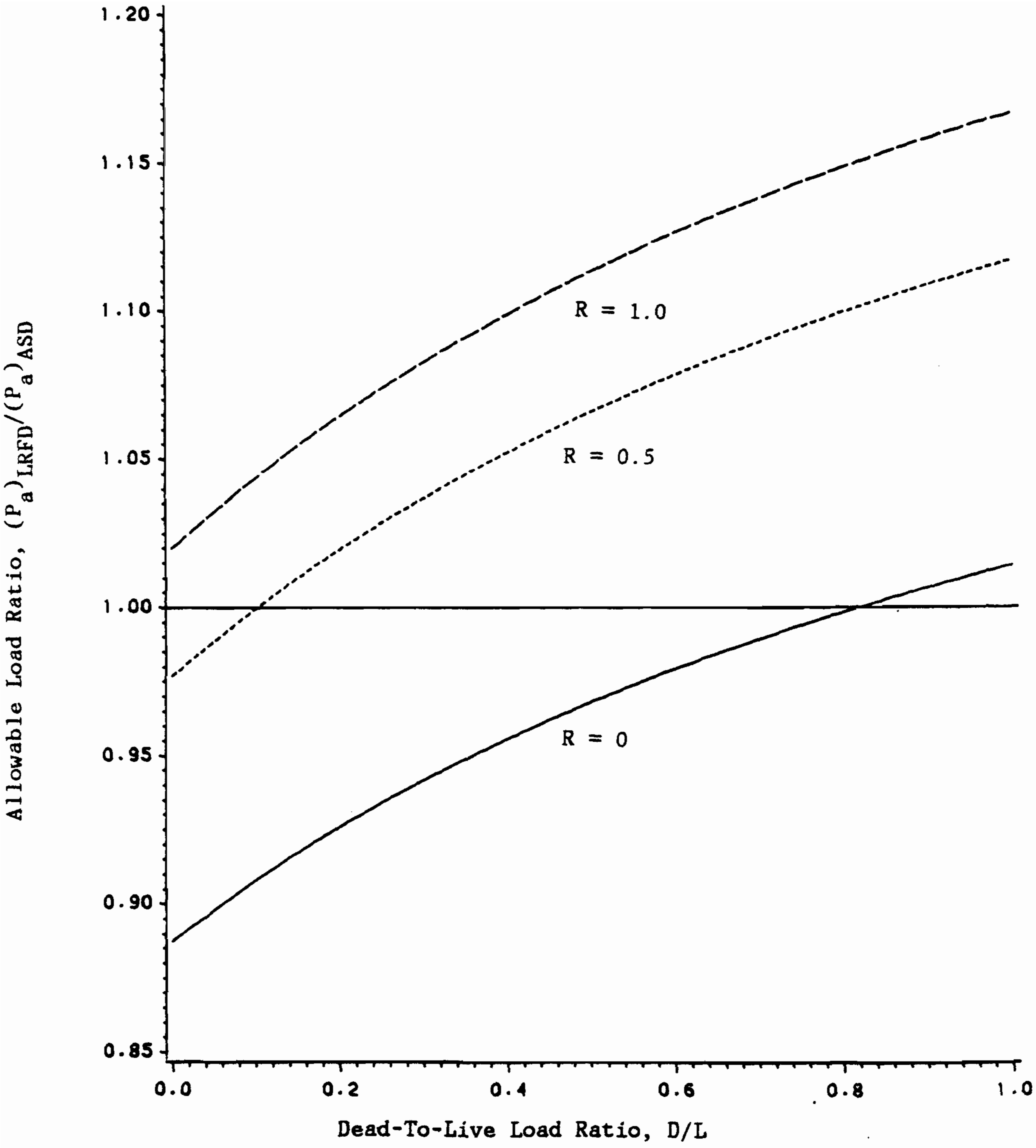

Figure 82. Allowable Load Ratio vs. D/L Ratio for Wall Studs in Compression-Case 2

Figure 82 shows the allowable load ratio versus dead-to-live load ratio for the wall studs used to develop Eq. (IX.33). 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.

B. WALL STUDS IN BENDING

1. Allowable Stress Design. According to Section D4.2 of the AISI Specification, for studs having identical sheathing attached to both flanges, and neglecting any rotational restraint provided by the sheathing, the allowable moments are M_{axo} and M_{ayo}, where

M_{axo} and M_{ayo} = Allowable moments about the centroidal axes determined in accordance with Section C3.1 of the AISI Specification, excluding the provisions of Section C3.1.2 (lateral buckling)

2. LRFD Criteria. According to Section D4.2 of the LRFD Specification, for studs having identical sheathing attached to both flanges, and neglecting any rotational restraint provided by the sheathing, the factored nominal moments are ϕM_{nxo} and ϕM_{nyo} as follows:

where

ϕ = 0.95 for sections with stiffened compression flanges

= 0.90 for sections with unstiffened compression flanges

M_{nxo} and M_{nyo} = Nominal moments about the centroidal axes determined in accordance with Section C3.1, excluding the provisions of Section C3.1.2 (lateral buckling)

3. <u>Comparison.</u> The unfactored moment can be calculated by using Eq. (IX.35) for both methods (ASD and LRFD) for comparison.

$$M_{TL} = M_{DL}+M_{LL} \tag{IX.35}$$

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 nominal section strength with a factor of safety of 1.67. Therefore, the allowable moment for beams is

$$(M_a)_{ASD} = M_n/\Omega_f = M_n/1.67 \tag{IX.36}$$

For LRFD, the allowable moment can be computed by using the following equation developed from Eq. (II.6).

$$(M_a)_{LRFD} = \phi M_n(D/L+1)/(1.2D/L+1.6) \tag{IX.37}$$

The ratio of the allowable moments is

$$\frac{(M_a)_{LRFD}}{(M_a)_{ASD}} = 1.67\phi \frac{D/L+1}{1.2D/L+1.6} \tag{IX.38}$$

For sections with stiffened compression flanges, $\phi = 0.95$

$$\frac{(M_a)_{LRFD}}{(M_a)_{ASD}} = 1.58 \frac{D/L+1}{1.2D/L+1.6} \tag{IX.39}$$

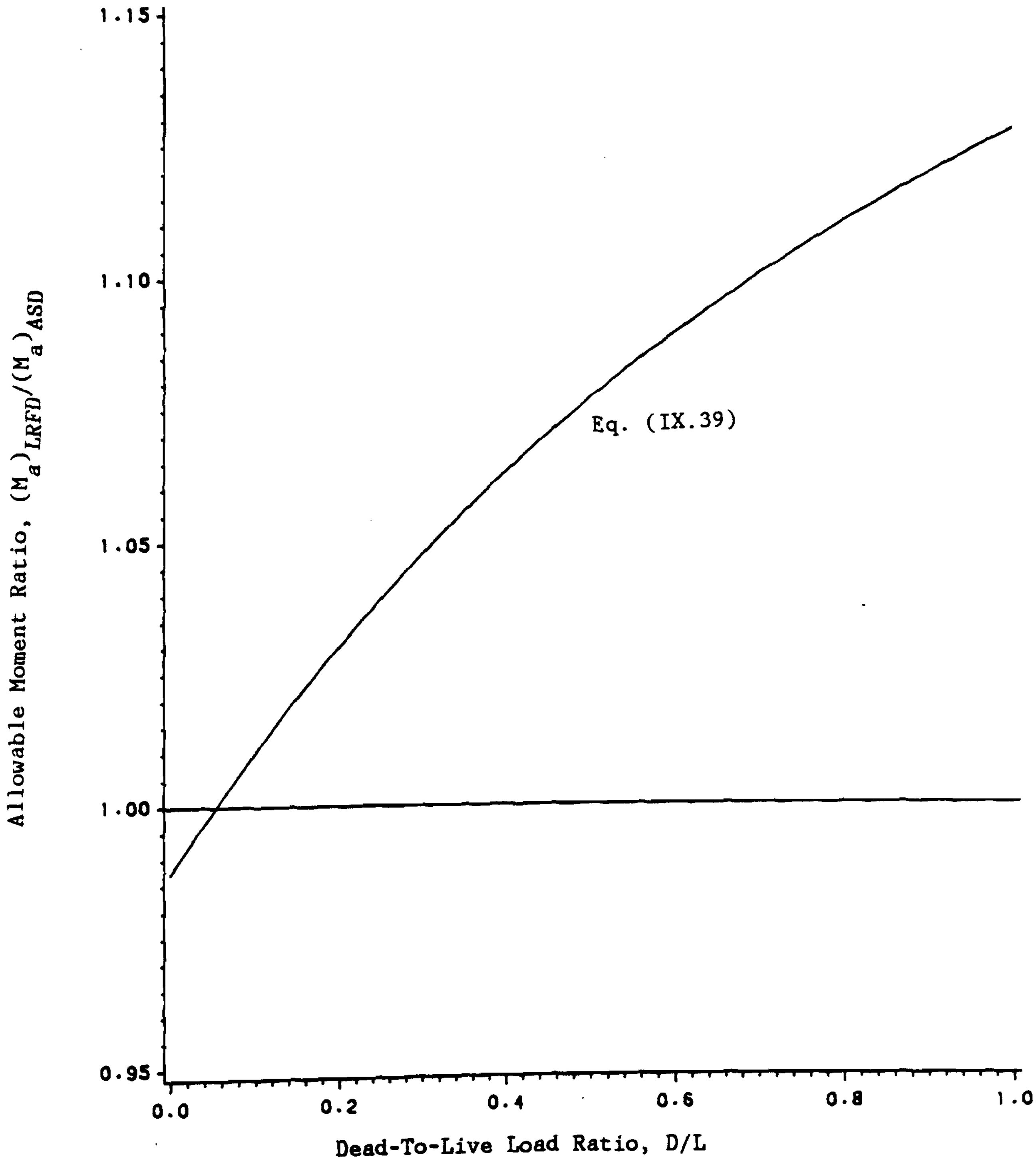

Figure 83. Allowable Moment Ratio vs. D/L Ratio for Wall Studs in Bending-Case 1

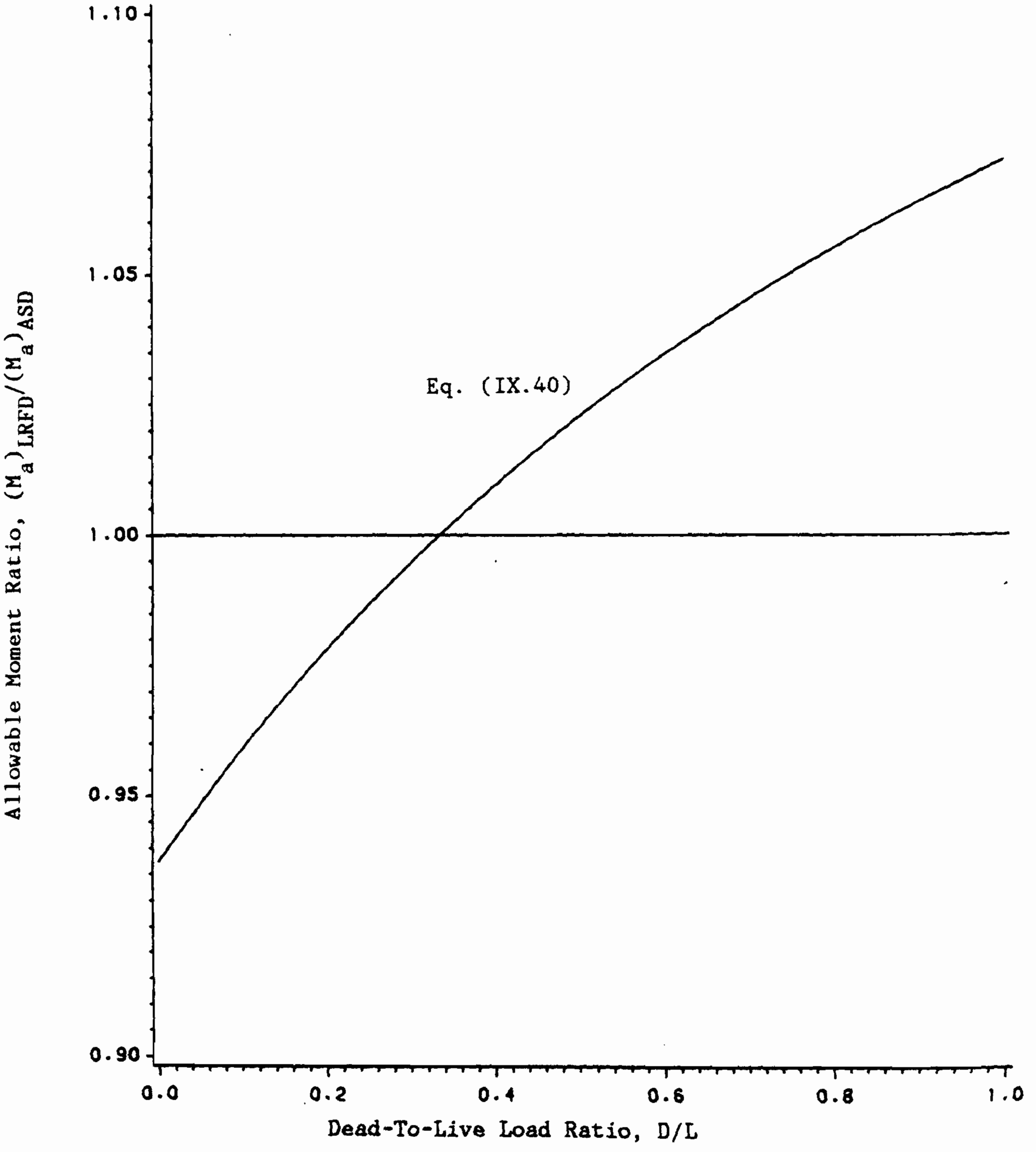

Figure 84. Allowable Moment Ratio vs. D/L Ratio for Wall Studs in Bending-Case 2

Figure 83 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, $\phi = 0.90$

$$\frac{(M_a)_{LRFD}}{(M_a)_{ASD}} = 1.50 \frac{D/L+1}{1.2D/L+1.6} \tag{IX.40}$$

Figure 84 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.

C. WALL STUDS WITH COMBINED AXIAL LOAD AND BENDING

1. Allowable Stress Design. According to Section D4.3 of the AISI Specification, the axial load and bending moments shall satisfy the interaction equations of Section C5 of the AISI Specification with the following redefined terms:

P_a = Allowable axial load determined according to Section D4.1 of the AISI Specification

M_{ax} and M_{ay} in Equations C5-1 and C5-3 shall be replaced by allowable moments, M_{axo} and M_{ayo}, respectively.

2. <u>LRFD Criteria.</u> According to Section D4.3 of the LRFD Specification, the factored axial load and bending moments shall satisfy the interaction equations of Section C5 of the LRFD Specification with the following redefined terms:

P_n = Nominal axial strength determined according to Section D4.1 of the LRFD Specification

M_{nx} and M_{ny} in Equations C5-1 and C5-3 shall be replaced by nominal moments, M_{nxo} and M_{nyo}, respectively.

3. <u>Comparison.</u> 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 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:

$$\frac{P}{P_a} = \frac{P_T}{P_n/\Omega_c} = \frac{\Omega_c P_T}{P_n} \qquad \text{(IX.41)}$$

$$\frac{M}{M_{ao}} = \frac{M_T}{0.6M_{no}} = \frac{M_T/M_{no}}{0.6} \qquad \text{(IX.42)}$$

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 which is defined in Article V

Substitution of Eqs. (IX.41) and (VI.42) into Eq. (VI.1) results in the following expression :

$$\frac{\Omega_c P_T}{P_n} + \frac{C_m(M_T/M_{no})}{0.6(1-\Omega_c P_T/P_{cr})} = 1.0 \qquad \text{(IX.43)}$$

By solving for P_T in the first term of Eq. (VI.43), the following equation for allowable load is obtained :

$$(P_T)_{ASD1} = \left[1- \frac{C_m(M_T/M_{no})}{0.6(1-\Omega_c P_T/P_{cr})}\right]\frac{P_n}{\Omega_c} \qquad \text{(IX.44)}$$

Equation (IX.44) is based on Eq. (VI.1) for 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 Eq. (VI.2) :

$$\frac{P}{P_{ao}} = \frac{P_T}{P_{no}/\Omega_c} = \frac{\Omega_c P_T}{P_{no}} \qquad \text{(IX.45)}$$

Substitution of Eqs. (IX.45) and (IX.42) into Eq. (VI.2) results in the following expression :

$$\frac{\Omega_c P_T}{P_{no}} + \frac{(M_T/M_{no})}{0.6} = 1.0 \qquad \text{(IX.46)}$$

By solving for P_T in Eq. (IX.46), the following equation for allowable load is obtained :

$$(P_T)_{ASD2} = \left[1 - \frac{(M_T/M_{no})}{0.6}\right] \frac{P_{no}}{\Omega_c} \qquad \text{(IX.47)}$$

Equation (IX.47) is based on Eq. (VI.2) for failure at the braced points.

When $P/P_a \leq 0.15$, Eq. (VI.3) can be used in lieu of Eqs. (VI.1) and (VI.2). Equation (VI.3) can be written in the following form by using Eqs. (IX.41) and (IX.42) :

$$\frac{\Omega_c P_T}{P_n} + \frac{M_T/M_{no}}{0.6} = 1.0 \qquad \text{(IX.48)}$$

By solving for P_T in Eq. (IX.48), the following equation for allowable load is obtained :

$$(P_T)_{ASD3} = \left[1 - \frac{M_T/M_{no}}{0.6}\right] \frac{P_n}{\Omega_c} \qquad \text{(IX.49)}$$

Equation (IX.49) is based on Eq. (VI.3) for flexural failure when the effect of the secondary moment is neglected.

For LRFD, the allowable axial loads were computed in accordance with Eq. (II.6) as follows :

$$\frac{P_D}{\phi_c P_n} = \frac{1.2D/L+1.6}{D/L+1}\left[\frac{P_T}{\phi_c P_n}\right] \tag{IX.50}$$

$$\frac{M_D}{\phi M_{no}} = \frac{1.2D/L+1.6}{D/L+1}\left[\frac{M_T/M_{no}}{\phi}\right] \tag{IX.51}$$

$$\frac{P_D}{\phi_c P_E} = \frac{1.2D/L+1.6}{D/L+1}\left[\frac{P_T}{\phi_c P_E}\right] \tag{IX.52}$$

Substitution of Eqs. (IX.50), (IX.51), and (IX.52) into Eq. (VI.7) results in the following expression :

$$\frac{1.2D/L+1.6}{D/L+1}\left\{\frac{P_T}{\phi_c P_n} + \frac{C_m(M_T/M_{no})}{\phi[1-(1.2D/L+1.6)P_T/(D/L+1)\phi_c P_E]}\right\} = 1.0 \tag{IX.53}$$

By solving for P_T in the first term of Eq. (IX.53), the following equation for allowable load is obtained :

$$(P_T)_{LRFD1} = \left\{\frac{D/L+1}{1.2D/L+1.6} - \frac{C_m(M_T/M_{no})}{\phi[1-(1.2D/L+1.6)P_T/(D/L+1)\phi_c P_E]}\right\}\phi_c P_n \tag{IX.54}$$

Equation (IX.54) is based on Eq. (VI.7) for 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 Eq. (VI.8) :

$$\frac{P_D}{\phi_c P_{no}} = \frac{1.2D/L+1.6}{D/L+1} \left[\frac{P_T}{\phi_c P_{no}} \right] \tag{IX.55}$$

Substitution of Eqs. (IX.55) and (IX.51) into Eq. (VI.8) results in the following expression :

$$\frac{1.2D/L+1.6}{D/L+1} \left[\frac{P_T}{\phi_c P_{no}} + \frac{(M_T/M_{no})}{\phi} \right] = 1.0 \tag{IX.56}$$

By solving for P_T in Eq. (IX.56), the following equation for allowable load is obtained :

$$(P_T)_{LRFD2} = \left[\frac{D/L+1}{1.2D/L+1.6} - \frac{(M_T/M_{no})}{\phi} \right] \phi_c P_{no} \tag{IX.57}$$

Equation (IX.57) is based on Eq. (VI.8) for failure at the braced points.

When $P_D/(\phi_c P_n) \leq 0.15$, Eq. (VI.9) can be used in lieu of Eqs. (VI.7) and (VI.8). Equation (VI.9) can be written in the following form by using Eqs. (IX.50) and (IX.51) :

$$\frac{1.2D/L+1.6}{D/L+1}\left[\frac{P_T}{\phi_c P_n}+\frac{(M_T/M_{no})}{\phi}\right]=1.0 \qquad \text{(IX.58)}$$

By solving for P_T in Eq. (IX.58), the following equation for allowable load is obtained :

$$(P_T)_{LRFD3}=\left[\frac{D/L+1}{1.2D/L+1.6}-\frac{(M_T/M_{no})}{\phi}\right]\phi_c P_n \qquad \text{(IX.59)}$$

Equation (IX.59) is based on (VI.9) for flexural failure when the effect of the secondary moment is neglected.

Equations (IX.44), (IX.47), and (IX.49) for determining the allowable axial load based on allowable stress design and Eqs. (IX.54), (IX.57), and (IX.59) 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 85 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 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 85. 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 86 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 87 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 85 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 without and with star symbols in Figure 88 are for $C_m = 1.0$ and 0.85, 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 88. 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 $C_m = 1.0$.

The curves without and with star symbols in Figure 89 are for $C_m = 1.0$ and 0.85, respectively, and for $F_y = 50$ ksi. 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 88. 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 $C_m = 1.0$.

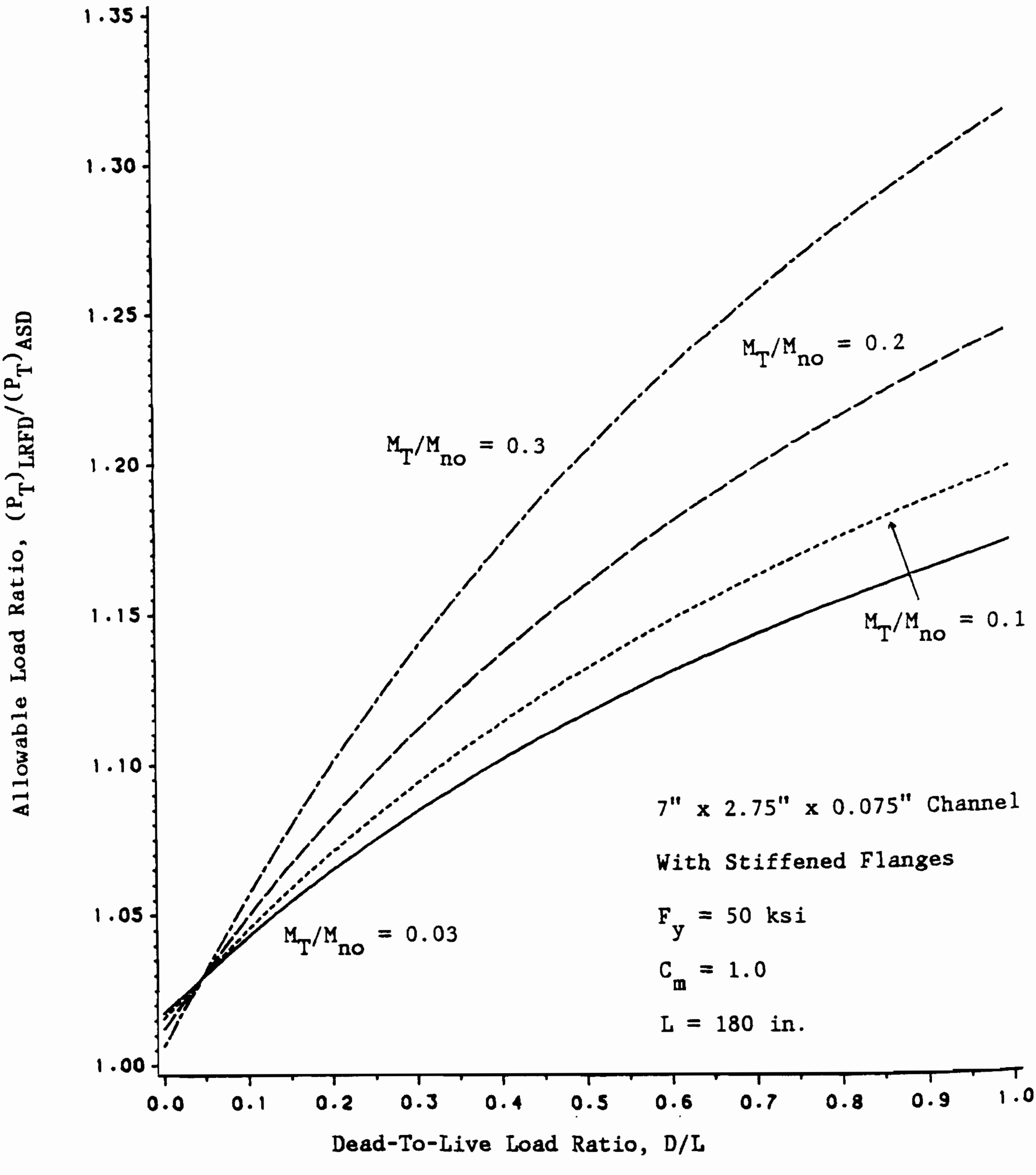

Figure 85. Allowable Load Ratio vs. D/L Ratio for Wall Studs With Combined Axial Load and Bending-Case A

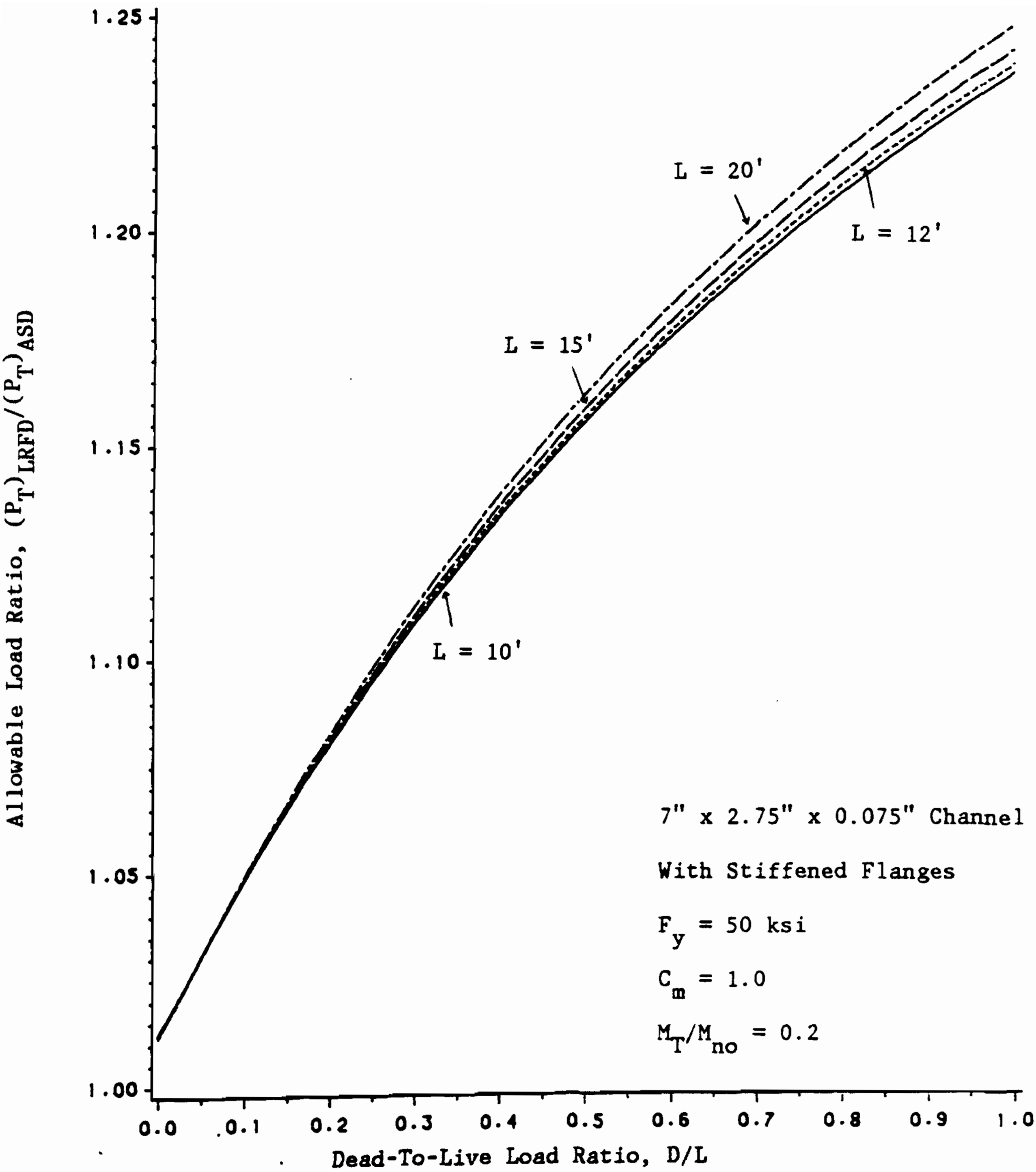

Figure 86. Allowable Load Ratio vs. D/L Ratio for Wall Studs With Combined Axial Load and Bending-Case B

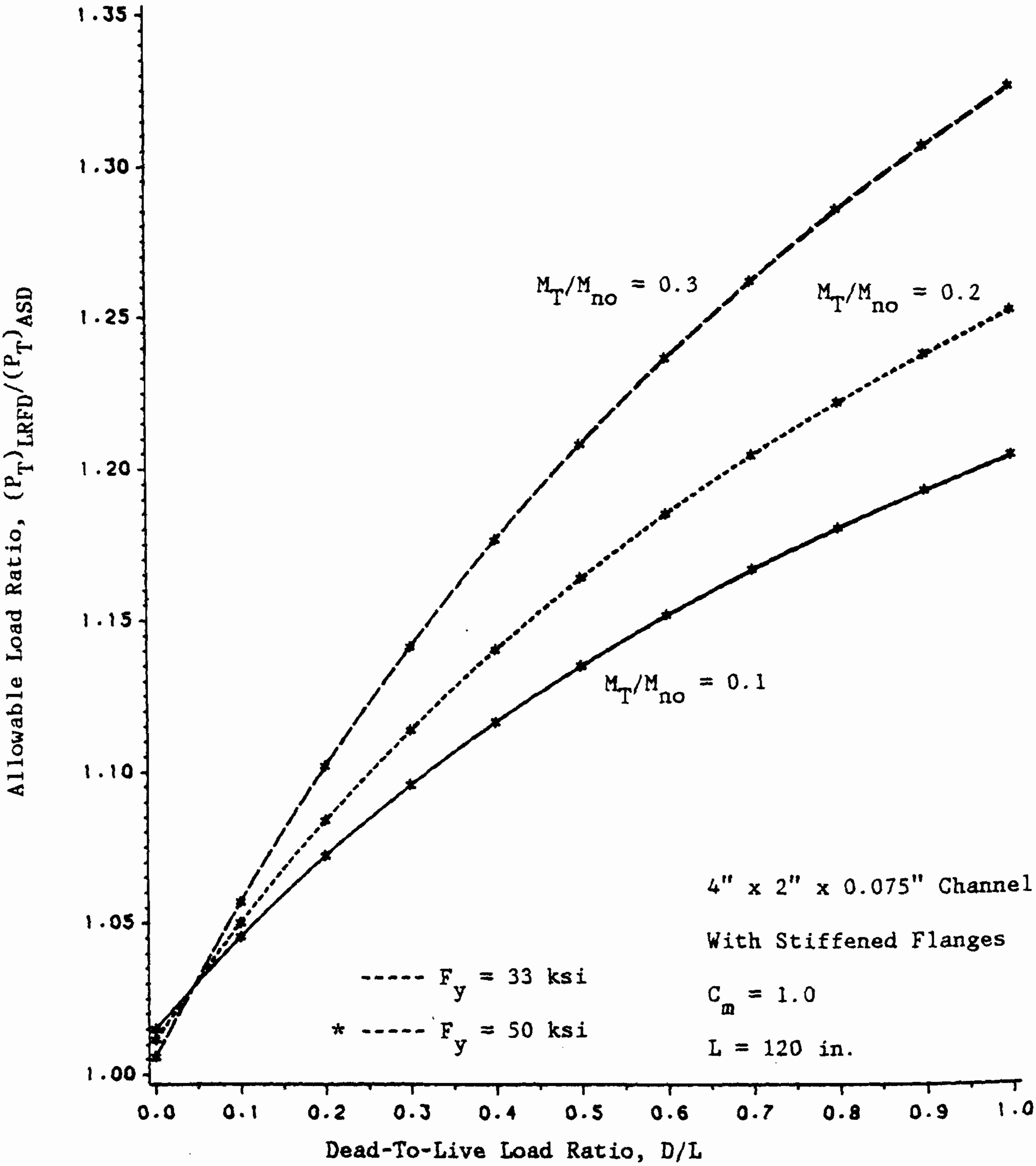

Figure 87. Allowable Load Ratio vs. D/L Ratio for Wall Studs With Combined Axial Load and Bending-Case C

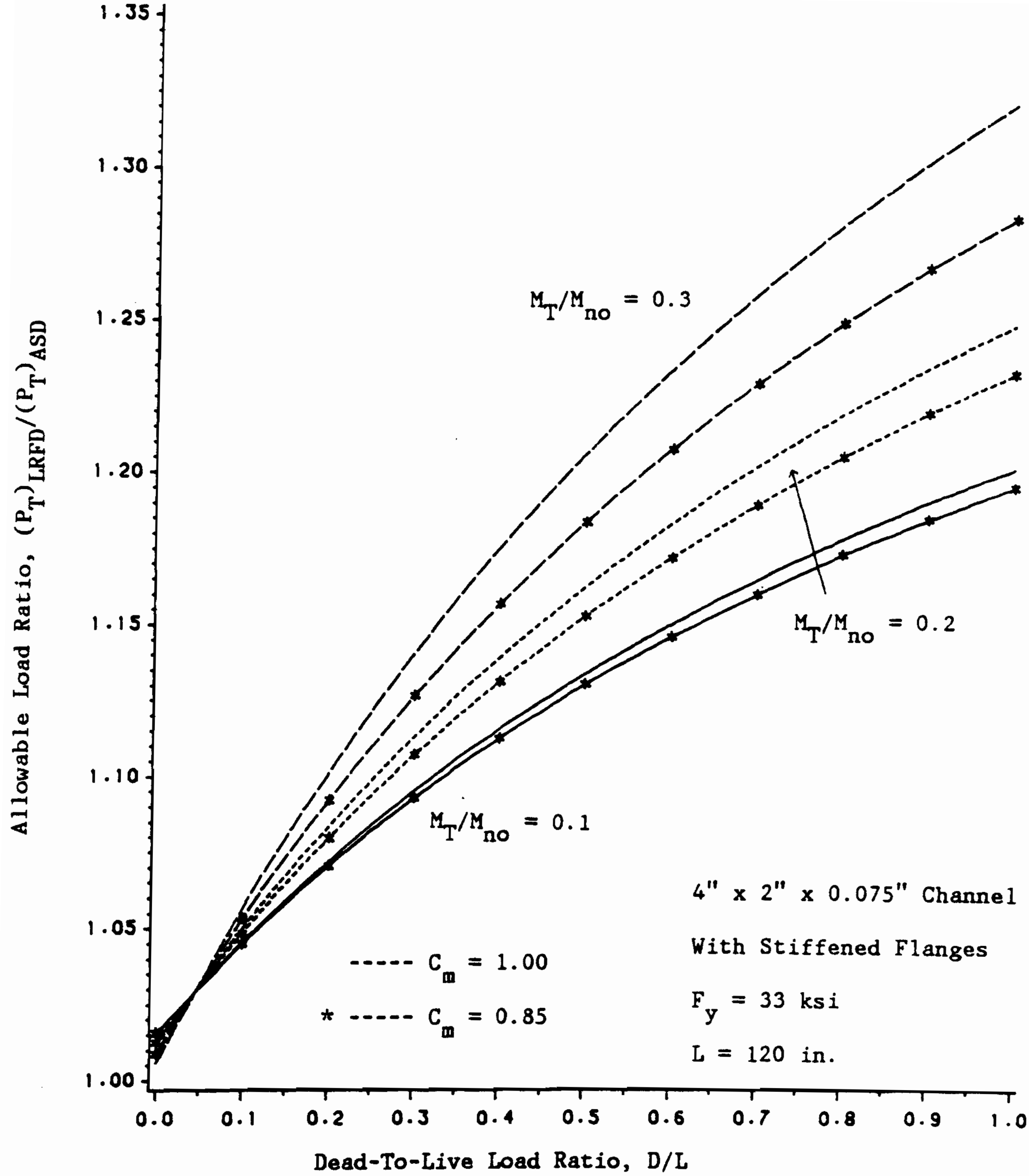

Figure 88. Allowable Load Ratio vs. D/L Ratio for Wall Studs With Combined Axial Load and Bending-Case D

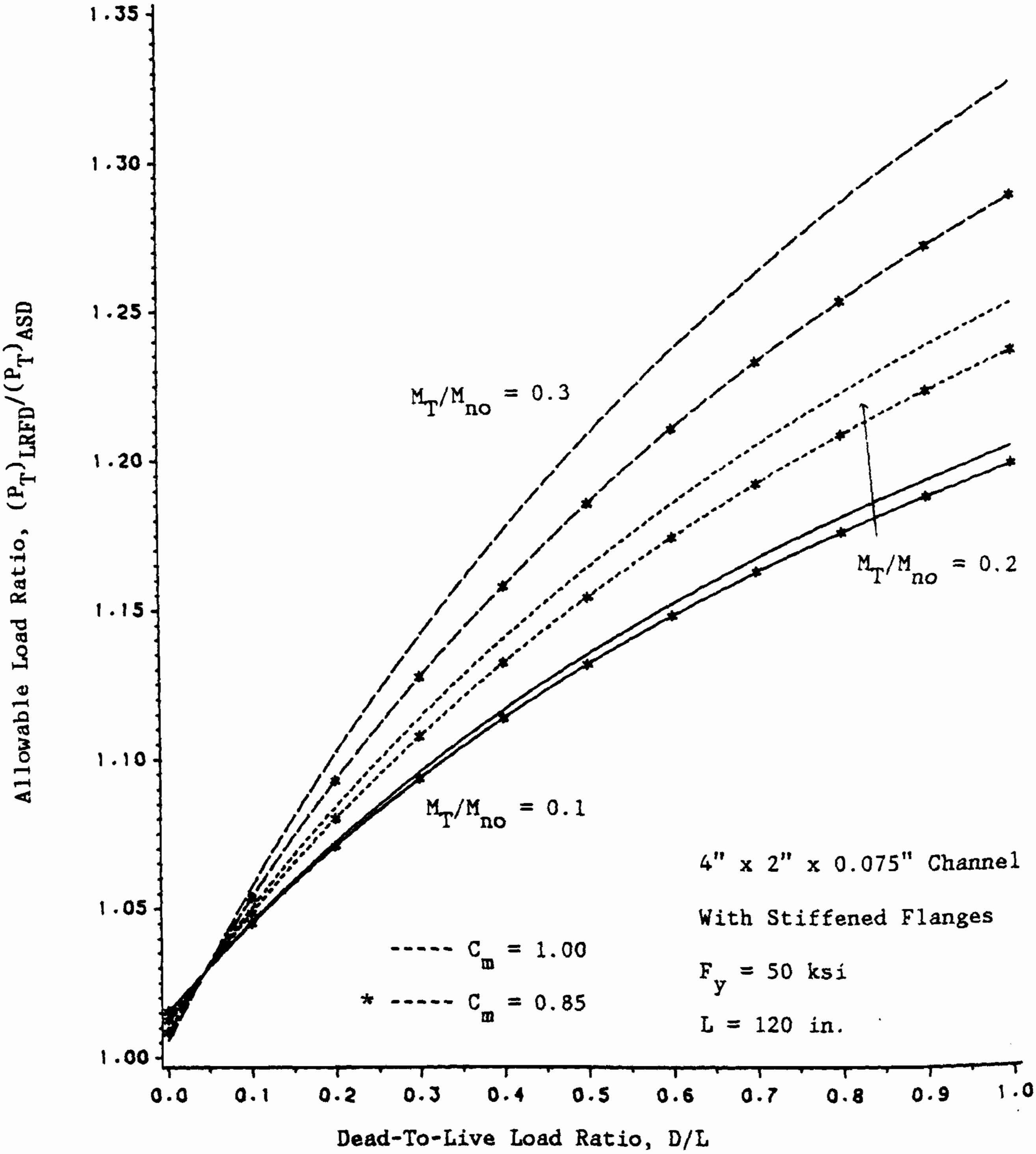

Figure 89. Allowable Load Ratio vs. D/L Ratio for Wall Studs With Combined Axial Load and Bending-Case E

Channel sections with unstiffened flanges were studied in a similar manner. Figure 90 shows the allowable load ratio versus dead-to-live load ratio for a channel section (7 in. x 1.5 in. x 0.075 in.) having unstiffened flanges with F_y = 50 ksi and an effective length of 15 ft. For D/L = 0.5 and M_T/M_{no} = 0.1, the allowable load obtained from LRFD is 11.7% larger than the value obtained from allowable stress design. For D/L = 0.5 and M_T/M_{no} = 0.3, LRFD would result in an allowable load 13.9% higher than the value determined from allowable stress design.

Figure 91 shows the allowable load ratio versus dead-to-live load ratio for end moment ratio of 0.2. Different curves represent different lengths of the channel section (7 in. x 1.5 in. x 0.075 in.) with F_y = 50 ksi. For M_T/M_{no} = 0.2 and D/L = 0.5, the allowable load values obtained from LRFD vary from 12.4% to 12.7% larger than the values obtained from the allowable stress design 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 1.125 in. x 0.075 in.) with unstiffened flanges was also included in this study for an effective length of 10 ft. The relationship between allowable load ratio and dead-to-live load ratio for the channel section is shown in Figure 92 for various end moment ratios and F_y values. The curves computed for F_y = 33 ksi are similar to the curves shown in Figure 90 which was obtained for 7 in. deep channel channel. For D/L = 0.5, the allowable load ratios vary from 1.12 to 1.14 for M_T/M_{no} ratios ranging from 0.1 to 0.3.

The curves with star symbols in Figure 92 represent the allowable load ratios determined for the same channel section by using $F_y = 50$ ksi. It can be seen that the allowable load ratios computed for $F_y = 50$ ksi are the same as those computed for $F_y = 33$ ksi. From this figure it can also be seen that the yield point has no significant effect on the allowable load ratio for the channel section with unstiffened flanges.

Figures 93 and 94 show how the C_m coefficient affects the allowable load ratio for the channel section having unstiffened flanges for $F_y = 33$ ksi and 50 ksi, respectively. The curves without star symbols are plotted for $C_m = 1.0$. The curves with star symbols represent the allowable load ratios calculated by using $C_m = 0.85$. For $D/L < 1/3$, the allowable load ratios are larger for $C_m = 0.85$ as compared to the allowable load ratios computed with $C_m = 1.0$. In general, the effect of the C_m value on the allowable load ratio is more important for wall studs with large end moment ratios.

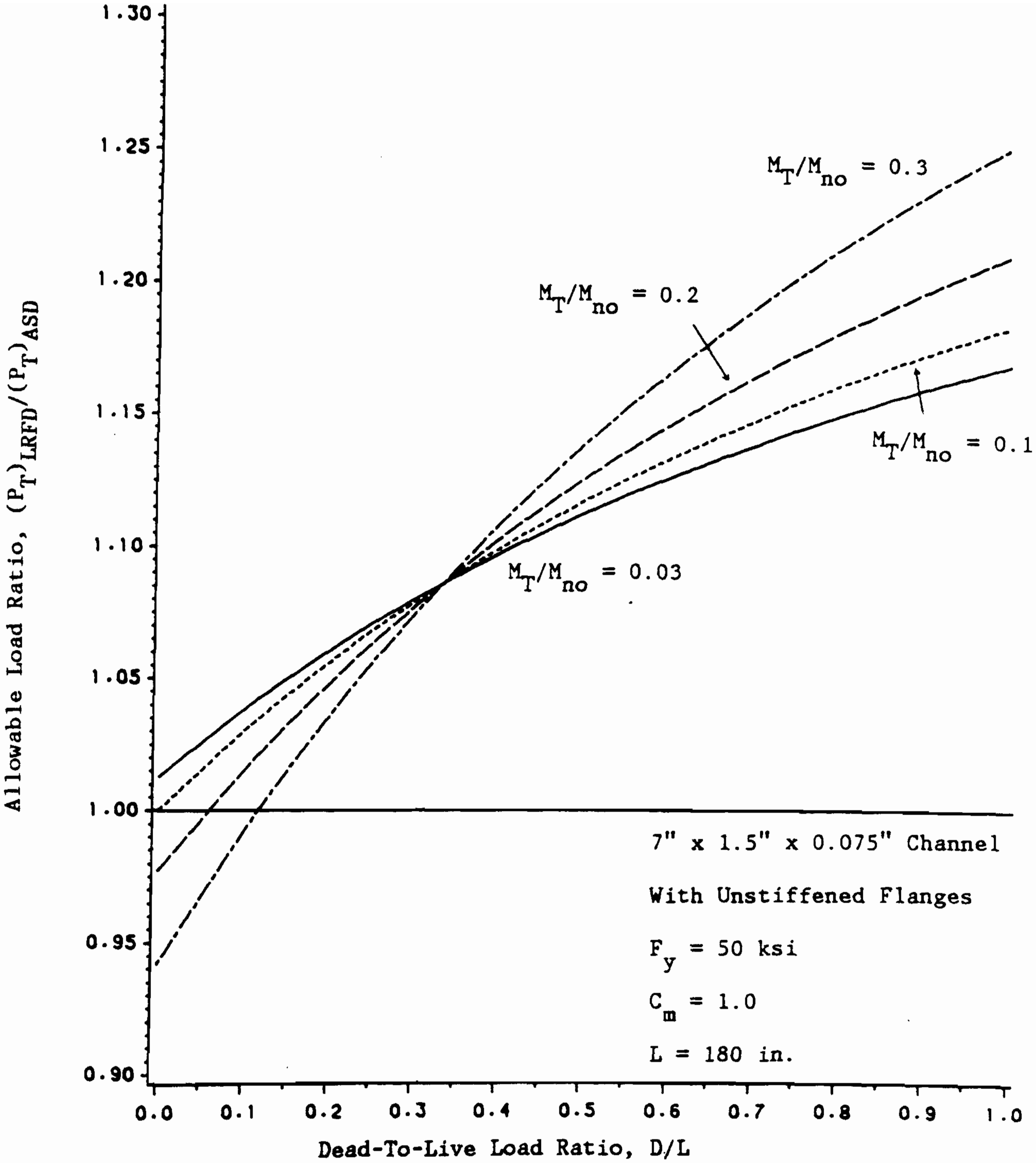

Figure 90. Allowable Load Ratio vs. D/L Ratio for Wall Studs With Combined Axial Load and Bending-Case F

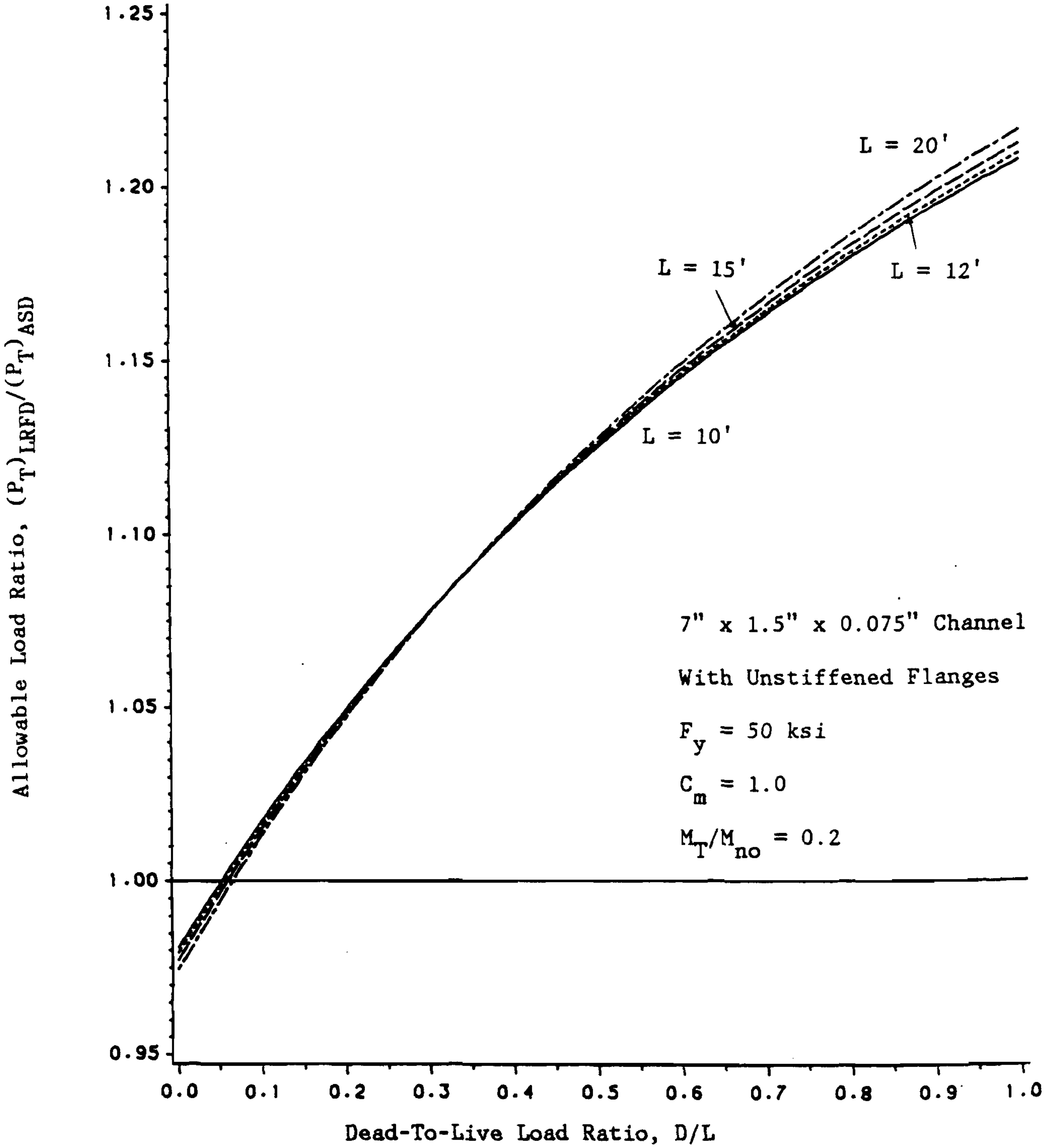

Figure 91. Allowable Load Ratio vs. D/L Ratio for Wall Studs With Combined Axial Load and Bending-Case G

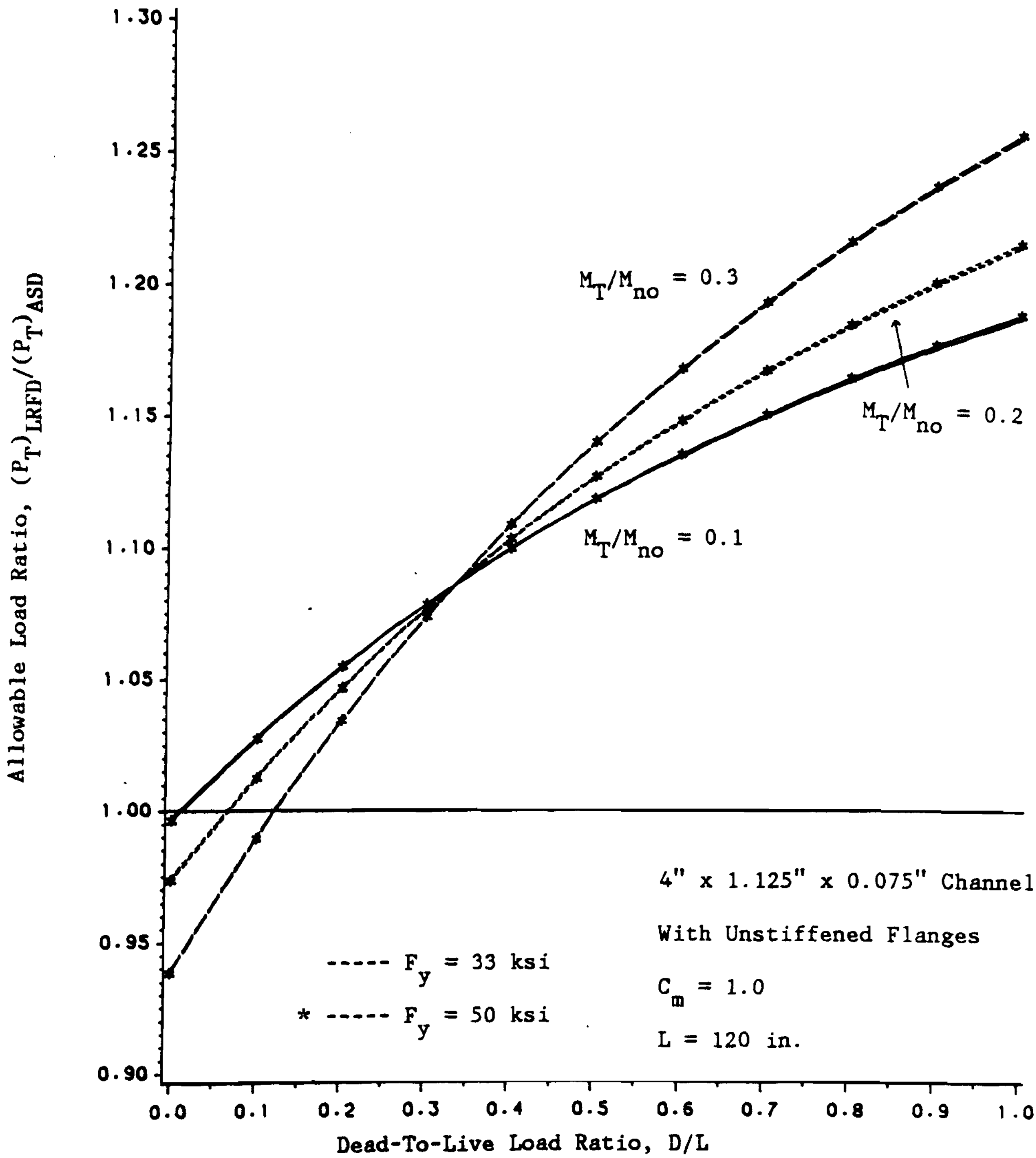

Figure 92. Allowable Load Ratio vs. D/L Ratio for Wall Studs With Combined Axial Load and Bending-Case H

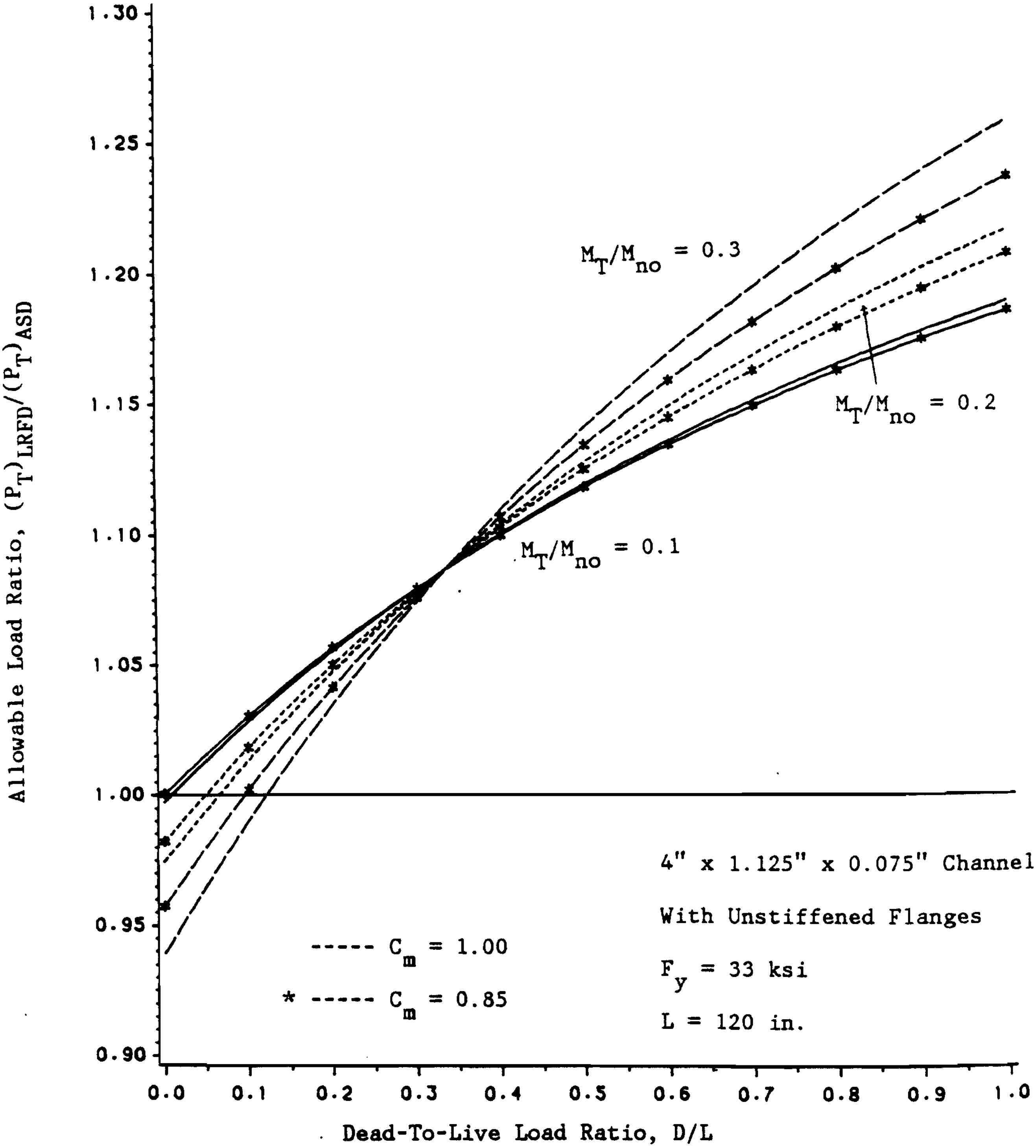

Figure 93. Allowable Load Ratio vs. D/L Ratio for Wall Studs With Combined Axial Load and Bending-Case I

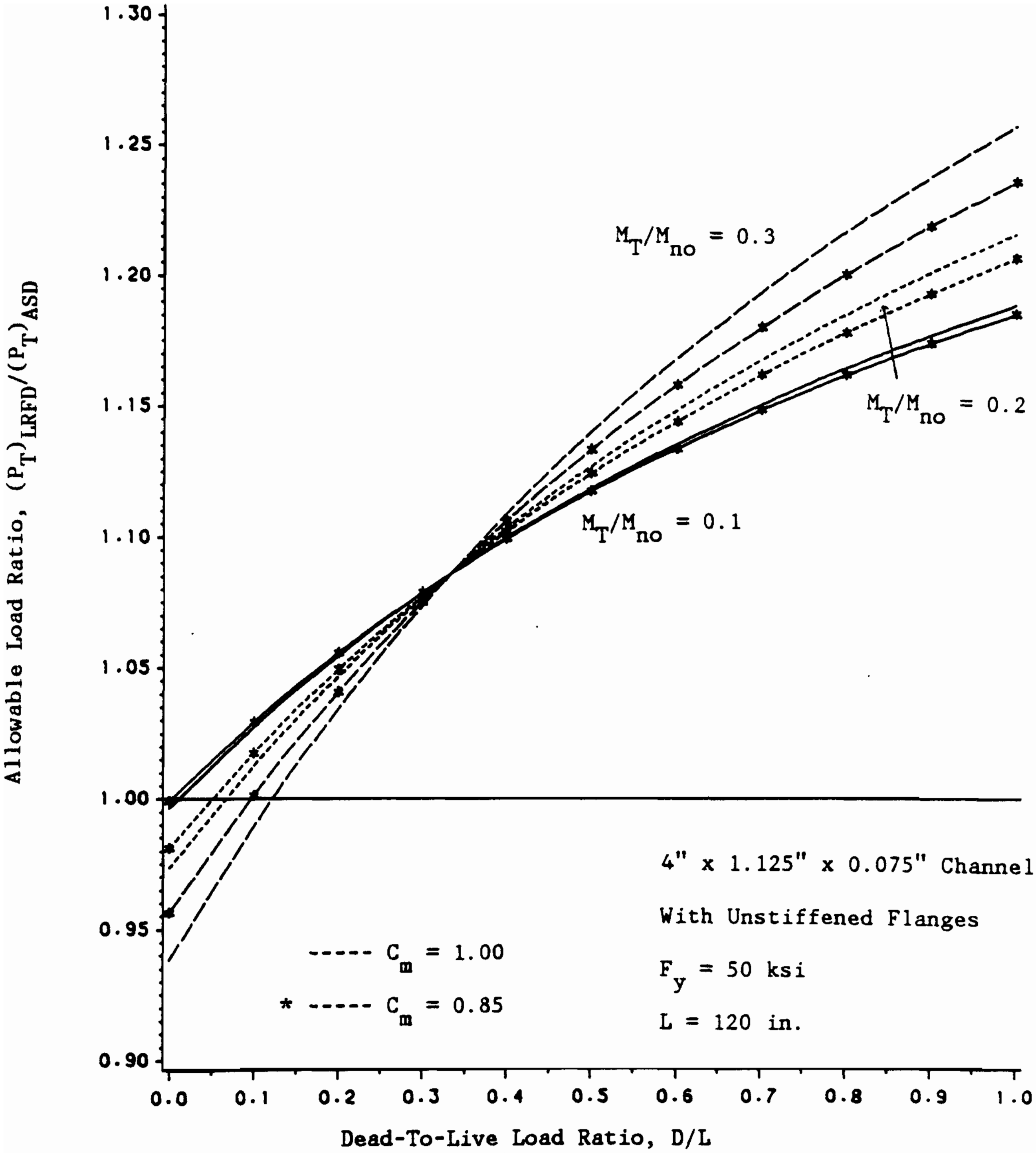

Figure 94. Allowable Load Ratio vs. D/L Ratio for Wall Studs With Combined Axial Load and Bending-Case J

X. CONCLUSIONS

Currently, the 1986 Edition of the Specification for the Design of Cold-Formed Steel Structural Members published by the American Iron and Steel Institute applies to the design of cold-formed steel members and connections for load-carrying purposes in buildings. This specification provides design formulas for determining allowable load carrying capacities for tension members, compression members, flexural members, and connections based on appropriate factors of safety recommended by AISI for different types of structural members.

The Load and Resistance Factor Design method for cold-formed steel members and connections has recently been studied by using probabilistic and statistical techniques to account for the uncertainties in design, fabrication, material properties, and applied loads. The Load and Resistance Factor Design Specification was developed from a joint research project conducted at the University of Missouri-Rolla, Washington University, and the University of Minessota.

This report compares these two methods for the design of cold-formed steel structural members using the proposed load and resistance factor design criteria and the allowable stress design criteria being used in the AISI Specification. Following a review of literature and discussion of different design variables used in both criteria, allowable loads using each design method were calculated for tension members, flexural members, compression members, beam-columns, connections, stiffeners, and wall studs. These allowable loads were then compared in Chapters III through IX for different types of structural members and connections. For some cases, specific examples were used in this study due to the complexity of the analysis.

For all types of structural members only the dead and live load combination was studied in this investigation. It was found that the D/L ratio has a significant effect on the allowable load ratio. In general, the allowable load ratio, $(P_a)_{LRFD}/(P_a)_{ASD}$, increases as the dead-to-live load ratio increases. Because cold-formed steel members are usually thin, the dead-to-live load ratios of such light weight members are expected to be lower than the ratios used for other building materials. In general practice, the dead-to-live load ratios used in building design of cold-formed steel members are less than 1/3. In view of the fact that the load factor used for live load is 1.6 which is larger than the load factor of 1.2 used for dead load, the LRFD criteria were found to be conservative for unusually small D/L ratios.

In addition to the effect of the dead-to-live load ratio, the resistance factors used in the LRFD criteria and the factors of safety used in allowable stress design also contribute to the differences between the allowable loads computed from two different methods. As the safety factor or resistance factor increases, the ratio of $(P_a)_{LRFD}/(P_a)_{ASD}$ also increases. For a given set of statistical data and a selected safety index, the resistance factor can be determined by Eq. (II.5). This equation is a function of the mean value and coefficient of variation of the professional factor which is the ratio of the tested load to the predicted load. A low value of the resistance factor is resulted from a low value of P_m and a large value of V_P which represents a big scatter of test results. This was the case for welded connections and plate failure of bolted connections.

The load and resistance factor design method is a rational approach for structural design. The research findings obtained from this comparative study of the current method based on allowable stress design and the proposed LRFD criteria can provide a useful reference for future revision of the current AISI Specification and the proposed LRFD Specification.

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