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Preliminary Design Guide for Cold-formed Steel C- And Z-members

Committee on Specifications for the Design of Cold-Formed Steel Structural Members

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PRELIMINARY DESIGN GUIDE FOR COLD-FORMED STEEL C- AND Z-MEMBERS

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Preface

The Preliminary Design Guide for Cold-Formed Steel C- and Z-Members provides a simplified method of preliminary design for these members. The Guide is based on the American Iron and Steel Institute's *Specification for the Design of Cold-Formed Steel Structural Members*, August 19, 1986 Edition with December 11, 1989 Addendum. The design equations have been calibrated to generally yield a conservative load or moment capacity, when compared to the Specification. The Guide can be as much as 25% conservative when compared to the Specification.

The Preliminary Design Guide has been developed in response to requests from users of the Specification for a simplified design procedure. While the AISI Specification is still the final authority, it is recognized that a simplified procedure is useful in performing preliminary design checks. In this context, the Guide can serve as a mechanism for the casual user or code official to quickly verify the adequacy of a cold-formed steel member.

AISI acknowledges the efforts of Dr. Roger A. LaBoube of the University of Missouri-Rolla in the development of this Guide. Dr. LaBoube has donated much time and effort towards the completion of this document.

Users of the Guide for Preliminary Design of Cold-Formed Steel C- and Z-Members are invited to offer comments and suggestions. User response will be critical in determining the direction of future simplification efforts.

Table of Contents

PRELIMINARY DESIGN GUIDE FOR COLD-FORMED STEEL C- AND Z-MEMBERS

INTRODUCTION

This is intended to serve as a guide for preliminary design of members cold-formed from sheet steels, and used in construction applications. Final design is to be based on the provisions of the 1986 Edition of the American Iron and Steel Institute's Specification for the Design of Cold-Formed Steel Structural Members, with 1989 Addendum.

Information herein is based on studies of the AISI Specification and has been calibrated to generally yield a conservative load capacity when compared with the Specification.

The guide differs from the AISI Specification on the following:

1. The design expressions apply only to C- and Z-sections, having either an edge-stiffened or an unstiffened compression flange, and subjected to bending about the major axis.
2. All section properties are the full, unreduced section properties.
3. To maintain simplicity, the range of applicability of the design expressions has been restricted in some instances.
4. The design expressions are extended to materials having yield strengths up to and including 55 ksi.
5. The flat width to thickness ratio of an edge-stiffened compression flange shall be less than or equal to 60.
6. The flat width to thickness ratio of an unstiffened compression flange shall be less than or equal to 30.
7. The depth to thickness ratio of the web shall be less than 200.
8. Edge stiffeners shall not be less than 45 degrees and not greater than 90 degrees to the compression flange.
9. Web elements shall be 90 degrees to the bearing surface, and free of holes or openings.

For situations not covered in this simplified Guide, or for more refined analysis, the designer is referred to the AISI Specification and Commentary.

The linear method of computing section properties is employed. For specifics, the user is referred to Part III of the AISI Manual.

SECTION 1 - GENERAL

1.1 Scope

This Guide is intended for preliminary design of structural members cold-formed from sheet and strip steels. The Guide considers only members with flat elements subject to static, gravity loads.

1.2 Material

The design expressions herein can be applied to any structural steel which has a yield strength not greater than 55 ksi, a proportional limit equal to or greater than 70 percent of the yield strength, a tensile to yield strength ratio not less than 1.08, and adequate ductility to form the member and serve the intended function.

SECTION 2 - DESIGN PROCEDURE

2.1 Procedure

This Guide is based on the allowable stress concept presented in terms of allowable moments and loads. The allowable moments and loads are determined by dividing the corresponding nominal capacities by an accepted factor of safety.

2.2 Definitions

A_g = gross cross-sectional area of the member, in^2.

b = width of compression flange (flat width plus radii and thickness), in.

d = overall depth of section, in.

d' = d - 2 t

D = length of edge stiffener, in.

K = effective length factor

E = modulus of elasticity of steel (29,500 ksi)

F_b = maxium bending stress, ksi

F_e = elastic buckling stress, ksi

F_y = yield point used for design, ksi

I_x, I_y = moment of inertia, in.^{4}

K_x, K_y = effective length factor

L = unbraced length of compression member, in.

M = applied moment, in-kips

M_a = allowable moment capacity, in-kips

M_n = nominal moment capacity, in-kips

N = actual length of bearing, in.

V = applied shear force, kips

V_a = allowable shear force, kips

S = $1.28 \sqrt{E/F_y}$ for stiffened elements and $0.42 \sqrt{E/F_y}$ for unstiffened elements

S_f = elastic section modulus for gross cross section, in^3.

R_c = axial load reduction factor

R_f = moment reduction factor

R_w = web reduction factor

r = inside bend radius, in.

r_o = polar radius of gyration, in.

r_y = radius of gyration of gross cross section about minor axis, in.

t = base steel thickness of any element or section, in.

w = flat width of compression flange, in.

SECTION 3 - MEMBER DESIGN

3.1 Properties of Sections

Properties of sections shall be determined in accordance with conventional methods of structural design. All properties shall be based on the gross cross section of the member.

3.2 Strength for Bending Only

Nominal Section Strength

For flexural members, the applied moment, assuming full lateral support, uncoupled from axial load, shear and local concentrated forces or reactions, shall not exceed the allowable moment, M_a, calculated as follows:

$M_a = M_n/1.67$ (Eq. 3.2-1)

where

$M_n = F_n S_f R_f$ (Eq. 3.2-2)
$F_n = [1.21 - 0.00034 (d'/t) \sqrt{F_y}] F_y \leq F_y$ (Eq. 3.2-3)
$d' = d - 2t$ (Eq. 3.2-4)
D = Overall depth of the section, in.
F_y = Yield point, ksi
t = Base steel thickness, in.
S_f = Elastic section modulus for the gross cross section with respect to a centroidal axis perpendicular to the web, in^3.
R_f = Reduction factor defined as follows:

1. For Edge-Stiffened Compression Flange, $w/t \leq 60$

$R_f = R_1 R_2$ (Eq. 3.2-5)
$R_1 = 1.227 - 0.284[(w/t)/S] \leq 1.0$ (Eq. 3.2-6)
w = flat width of compression flange, in.
$S = 1.28 \sqrt{E/F_y}$ (Eq. 3.2-7)
$R_2 = 1.5 (D/b) + 0.55$, when $F_n = F_y$ and $0.2 \leq D/b < 0.3$ (Eq. 3.2-8)
$= 1.0$, when $F_n = F_y$ and $0.3 \leq D/b \leq 0.4$
$= 1.0$, when $F_n \leq F_y$
D = Length of edge stiffener where $0.2 b \leq D \leq 0.4 b$, in.
b = Width of compression flange (flat width plus radii and thickness), in.

2. For Unstiffened Compression Flange, $w/t \leq 30$

$R_f = 1.190 - 0.190[(w/t)/S] \leq 1.0$ (Eq. 3.2-9)
$S = 0.42 \sqrt{E/F_y}$ (Eq. 3.2-10)

Lateral Buckling Strength

For flexural members, the applied moment, assuming discrete point bracing, uncoupled from axial load, shear, and local concentrated forces or reactions, shall not exceed the allowable moment, M_a, calculated as follows:

$M_a = M_n / 1.67$ (Eq. 3.2-11)

where

$M_n = R_f R_w M_c$ (Eq. 3.2-12)

$M_y = S_f F_y$ . . . . (Eq. 3.2-13)

L = Unbraced length of member, in.

$R_w = 1.21 - 0.00034\,(d'/t)\sqrt{F_y} \leq 1.0$ . . . . (Eq. 3.2-14)

S_f, F_y, F_n have been previously defined

M_c = Critical moment as calculated below:

For $M_e > 0.5\,M_y$

$M_c = M_y\,[1.0 - M_y/(4\,M_e)]$ . . . . (Eq. 3.2-15)

For $M_e \leq 0.5\,M_y$

$M_c = M_e$ . . . . (Eq. 3.2-16)

where

$M_e = 0.20\,\pi^2\,E\,C_b\,d\,I_y\,/\,L^2$ For Z-shaped sections . . . . (Eq. 3.2-17)

$M_e = 0.42\,\pi^2\,E\,C_b\,d\,I_y\,/\,L^2$ For C-shaped sections . . . . (Eq. 3.2-18)

1. For Edge-Stiffened Compression Flange, $w/t \leq 60$

   R_f has been previously defined.

2. For Unstiffened Compression Flange, $w/t \leq 30$

   R_f has been previously defined.

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

   $C_b = 1.75 + 1.05\,(M_1/M_2) + 0.3\,(M_1/M_2)^2 \leq 2.3$ . . . . (Eq. 3.2-19)

   Where

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

3.3 Strength for Shear Only

The allowable shear force, V_a, in a single web element is given by the following applicable equation:

a. For $d'/t \leq 3.19\,\sqrt{E/F_y}$

   $V_a = 0.88\,t^2\sqrt{F_y E} \leq 0.4\,F_y\,d'\,t$ . . . . (Eq. 3.3-1)

b. For $D'/t > 3.19\,\sqrt{E/F_y}$

$V_a = 2.83\ E\ t^3/d'$ . . . . (Eq. 3.3-2)

where

t = Web thickness, in.

$d' = d - 2t$ . . . . (Eq. 3.3-3)

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

3.4 Strength for Combined Bending and Shear

For beams with unreinforced web elements, the applied moment, M, and the applied shear, V, shall satisfy the following:

$(M/M_a)^2 + (V/V_a)^2 \leq 1.0$ . . . . (Eq. 3.4-1)

where

M_a = Allowable moment per Section 3.2

V_a = Allowable shear force per Section 3.3

3.5 Web Crippling Strength

The applied concentrated load or reaction on a single unreinforced flat web element shall not exceed the allowable load, P_a, obtained from the following applicable equation.

These equations are valid for webs of flexural members subject to compressive, concentrated loads or reactions, or their components, acting perpendicular to the longitudinal axis of the member.

The equations apply to beams when $d'/t \leq 200$, $r/t \leq 6$, $N/t \leq 210$, $N/d' \leq 3.5$ and the angle between the plane of the web and the plane of the bearing surface is 90 degrees.

The value of P_a is for a single web element. For two or more webs, P_a should be evaluated for each web element and added to obtain the total allowable load.

$P_a = P_n / 1.85$ . . . . (Eq. 3.5-1)

where P_n is the nominal load and is defined by the following load and geometry conditions:

1. Opposing loads spaced greater than 1.5 d'
   a. End reaction for shapes having edge-stiffened flanges

      $P_n = 13.32\ t^2\ F_y\ [1 - (F_y/133)]\ [1 - (d'/542t)]\ [1 + (N/100t)]$ . . . . . (Eq. 3.5-2)

   b. End reaction for shapes having unstiffened flanges

      $P_n = 8.70\ t^2\ F_y\ [1 - (F_y/133)]\ [1 - (d'/780t)]\ [1 + (N/100t)]$ . . . . . . (Eq. 3.5-3)

If r/t > 1.0, then the nominal load is reduced by multiplying by the factor [1.15(1 - (r/8t))], which shall be taken not less than 0.5. If N/t > 60, then the factor [1+(N/100t)] may be replaced by [0.71 (1 + (N/47t))].

c. Interior reactions for shapes having stiffened or unstiffened flanges

$P_n = 19.88\, t^2 F_y [1 - (F_y/183)] [1 - (d'/270t)] [1+(N/143t)]$ . . . . . . (Eq. 3.5-4)

If r/t > 1.0, then the nominal load is reduced by multiplying by the factor [1.06 (1 - (r/17t))]. If N/t > 60, then the factor [1 + (N/143t)] may be replaced by [0.75 (1 + (N/68t))].

2. Opposing loads spaced closer than 1.5 d'

a. End reactions for shapes having stiffened or unstiffened flanges

$P_n = 9.80\, t^2 F_y [1 - (F_y/133)] [1 - (d'/425t)] [1 + (N/100t)]$ . . . . . . (Eq. 3.5-5)

If r/t > 1.0, then the nominal load is reduced by multiplying by the factor [1.15 (1 - (r/8t))], which shall be taken not less than 0.5.

b. Interior reactions for shapes having stiffened or unstiffened flanges

$P_n = 28.49\, t^2 F_y [1 - (F_y/183)] [1 - (d'/341t)] [1 + (N/769t)]$ . . . . . (Eq. 3.5-6)

If r/t > 1.0, then the nominal load is reduced by multiplying by the factor [1.06 (1 - (r/18t))]

where

F_y = Yield point used for design, ksi

d' = $d - 2t$ . . . . (Eq. 3.5-7)

t = Web thickness, in.

N = Actual length of bearing, in. For the case of two equal and opposite concentrated loads distributed over unequal bearing lengths, the smaller value of N should be used when calculating P_n.

r = Inside bend radius, in.

3.6 Combined Bending and Web Crippling

Unreinforced flat webs of shapes subject to a combination of bending and concentrated load or reaction shall be designed to meet the following:

$1.2 (P/P_a) + (M/M_a) \leq 1.5$ . . . . (Eq. 3.6-1)

where

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

P_a = Allowable concentrated load or reaction per Section 3.5

M = Bending moment at, or adjacent to, the point of application of the concentrated load or reaction

M_a = Allowable bending moment per Section 3.2

3.7 Concentrically Loaded Compression Members Having Edge-Stiffened Flanges

The applied concentric axial compression load on a member shall not exceed the allowable load, P_a, computed from the following equations. These equations are valid for members having the resultant of all axial loads passing through the centroid of the gross section, and having a slenderness ratio less than 200.

3.71 Sections Subject to Flexural Buckling

These equations are valid for members having the resultant of all axial loads passing through the centroid of the gross section; having a slenderness ratio less than 200; and which are braced such that the member is subject to flexural buckling. For such members the unbraced length is limited by the following minimum unbraced length, L_{min} (for shorter unbraced lengths, torsional-flexural buckling may govern):

For $b/d \le 0.4$, L_{min} has no lower limit . . . . (Eq. 3.7-1)
For $b/d > 0.4$, $L_{min} = (d^2 / t)\,[3.72 - 19.53\,(b/d) + 25.91\,(b/d)^2]$ . . . . . (Eq. 3.7-2)

The applied axial load shall not exceed P_a calculated as follows:

$P_a = P_n/1.92$ . . . . (Eq. 3.7-3)

where

$P_n = F_n A_g R_c$ . . . . (Eq. 3.7-4)
A_g = gross cross-sectional area of the member, in^2.
F_n is determined as follows:

For $F_e > F_y/2$ $F_n = F_y\,(1 - F_y/4F_e)$ . . . . (Eq. 3.7-5)
For $F_e \le F_y/2$ $F_n = F_e$ . . . . (Eq. 3.7-6)

$F_e = \pi^2 E/(K L/ r_y)^2$ . . . . (Eq. 3.7-7)

E = modulus of elasticity, ksi

K = effective length factor

L = unbraced length of member, in.

r_y = radius of gyration of the gross cross section about the minor axis, in.

KL/r = maximum effective slenderness ratio.

R_c is a reduction factor defined as follows:

$R_c = R_3 R_4$ . . . . (Eq. 3.7-8)

$R_3 = 1.50 - 0.21\,[(\Sigma w/t) / S] + 0.01\,[(\Sigma w/t) / S]^2 \le 1.0$ . . . . (Eq. 3.7-9)

$R_4 = 0.175\,[(L/r_y)C_c] + 0.825 \leq 1.0$ . . . . (Eq. 3.7-10)

$S = 1.28\sqrt{E/F_y}$ . . . . (Eq. 3.7-11)

$C_c = \sqrt{2\pi^2 E/F_y}$ . . . . (Eq. 3.7-12)

$\Sigma w =$ the sum of all flat widths of the cross section (i.e., flanges + web + stiffeners), in.

3.72 Sections Subject to Torsional-Flexural Buckling

The applied concentric load passing through the centroid of a member shall not exceed the allowable load, P_a, calculated as follows:

$P_a = P_n/1.92$ . . . . (Eq. 3.7-13)

where

$P_n = F_n\,A_g\,R_c$ . . . . (Eq. 3.7-14)

$A_g =$ gross cross-sectional area of the member, in.^{2}

F_n is determined as follows:

For $F_e > F_y/2$

$F_n = F_y\,(1 - F_y/4F_e)$ . . . . (Eq. 3.7-15)

For $F_e \leq F_y/2$

$F_n = F_e$ . . . . (Eq. 3.7-16)

where F_e is the lesser value of the following two equations:

$F_e = \pi^2 E/\,(K_y\,L_y\,/\,r_y)^2$ . . . . (Eq. 3.7-17)

$F_e = \sigma_T\,\sigma_{ex}\,/\,(\sigma_T - \sigma_{ex})$ . . . . (Eq. 3.7-18)

where

$\sigma_{ex} = \pi^2\,E\,I_x\,/\,A_g(K_x L_x)^2$ . . . . (Eq. 3.7-19)

$\sigma_T = 0.17\,\pi^2\,E\,d^2\,I_y\,/\,[(L_T)^2\,(I_x + I_y)]$, for Z-shaped sections . . . . . . (Eq. 3.7-20)

$\sigma_T = 0.031\,\pi^2\,E\,d^2\,I_y\,/\,[(L_T)^2\,(I_x + I_y)]$, for C-shaped sections
back to back . . . . (Eq. 3.7-21)

$\sigma_T = 0.17\,\pi^2\,E\,d^2\,I_y\,/\,[(L_T)^2\,A_g r_o{}^2]$, for C-shaped sections . . . . (Eq. 3.7-22)

where

$r_o{}^2 = [(Ix + Iy)/A_g] + 1.9\,\{[3b^2\,/\,(6\,b{+}d)] + [b^2/\,(2b + d)]\}^2$ . . . . (Eq. 3.7-23)

All other parameters have been previously defined.

3.8 Combined Axial Load and Bending

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

$$\frac{P}{P_a}+\frac{C_{mx}M_x}{M_{ax}\alpha_x}+\frac{C_{my}M_y}{M_{ay}\alpha_y}\le 1.0 \qquad \text{(Eq. 3.8-1)}$$

$$\frac{P}{P_{ao}}+\frac{M_x}{M_{ax}}+\frac{M_y}{M_{ay}}\le 1.0 \qquad \text{(Eq. 3.8-2)}$$

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

$$\frac{P}{P_a}+\frac{M_x}{M_{ax}}+\frac{M_y}{M_{ay}}\le 1.0 \qquad \text{(Eq. 3.8-3)}$$

where

P = Applied axial load

M_x, M_y = Applied moments

P_a = Allowable axial load determined in accordance with Section 3.7

P_{ao} = Allowable axial load determined in accordance with Section 3.7 with $F_n = F_y$

M_{ax}, M_{ay} = Allowable moments about the centroidal axes determined in accordance with Section 3.2

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

$$= 1/[1-(1.92\,P/P_{cr})] \qquad \text{(Eq. 3.8-4)}$$

I_b = Moment of inertia of the full, unreduced cross section about the axis of bending, in.^{4}

$$P_{cr} = \frac{\pi^2 E I_b}{(K_b L_b)^2} \qquad \text{(Eq. 3.8-5)}$$

L_b = Actual unbraced length in the plane of bending, in.

K_b = Effective length factor in the plane of bending

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

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

   $C_m = 0.85$

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

   $$C_m = 0.6 - 0.4\,(M_1/M_2) \qquad \text{(Eq. 3.8-6)}$$

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, $C_m = 1.0$.

Commentary on Preliminary Design Guide for Cold-Formed C- and Z- Members

INTRODUCTION

The intent of this commentary is to provide some insight into the development, and scope of application of the Design Guide. The initial motivation for undertaking this development was to provide a tool for preliminary design of C- and Z-sections. However, a broader application may be in the confirmation of detailed computer design calculations. This confirmation may be undertaken by a consulting engineer or code official who is unfamiliar with the 1986 edition of the Specification for the Design of Cold-Formed Steel Structural Members (AISI 1986), with 1989 Addendum. Recognizing the conservative nature of the provisions of the Design Guide, a code official may opt to accept a final design that complies with the requirements set forth in the Guide. In this context, the Design Guide could serve as a mechanism for the casual user to employ a more economical cold-formed shape in lieu of an alternate structural component.

The information contained in the Guide is based on parametric studies of specific paragraphs of the 1986 edition of the Specification that apply to the design of C- and Z-shapes. The design equations of the Guide have been calibrated with the Specification to generally yield a conservative load, or moment capacity, when compared with the Specification. The Guide could, in some situations, be as much as 25% conservative when compared to the Specification.

The Specification provides general provisions for virtually any cold-formed shape. The general nature of the Specification naturally results in rigorous load capacity calculations, and the need to consider complex limit states. Because the Design Guide applies only to C- or Z-shaped sections, and ensures limits to preclude certain limit states from occurring, the load capacity calculations have been simplified.

The concept adopted for the Guide, and followed throughout the document, is to use only full section properties. The negative influence that local buckling has on the load capacity is accounted for by reduction factors. Use of full section properties is contrary to the Specification's design philosophy, but is consistent with the use of full section properties for hot-rolled design; it is a concept with which all structural engineers are familiar.

Evaluation of section properties can be accomplished using the recognized methods of structural mechanics. The Guide permits the continued use of the linear method of computing section properties, as is recognized by the Specification. Part III of the Cold-Formed Steel Design Manual (AISI 1986) gives details regarding the linear method.

For situations specifically not addressed in the Guide, the user is referred to the Specification.

SCOPE

The Guide, as with the Specification, is limited to steel structural members cold-formed from carbon or low-alloy sheet, strip, plate or bar. The forming process is carried out at, or near, room temperature by the use of bending brakes, press brakes or roll-forming machines.

TERMS

Terminology used in the Guide is consistent with the Specification, and the user is referred to the Specification.

MATERIAL

The Specification provides a comprehensive list of ASTM Standards for steels that are acceptable for fabrication and design of cold-formed steel members. However, because the parametric studies that serve as the basis of the Guide provisions focused on the commonly used commercial applications of C- and Z-shaped members, the design expressions in the Guide only apply to structural steels which have a yield strength not greater than 55 ksi, a proportional limit equal to or greater than 70 percent of the yield strength, and adequate ductility to form the member and serve the intended use.

DESIGN PROCEDURE

The Guide is consistent with the Specification in that it is based on the allowable load and moment concept, wherein the allowable capacity is arrived at by applying a factor of safety to the nominal capacity. The factors of safety employed by the Guide are consistent with the values stipulated in the Specification.

STRENGTH FOR BENDING

The Guide may be used for members with full lateral support and discretely braced members. Reduction in load capacity as a result of local instability is recognized in the Guide, and accounted for by using reduction factors applied to the nominal yield moment.

Web local buckling effects are accounted for by applying the following reduction factor to the nominal stress, F_n,

$$[\, 1.21 - 0.00034\, (d'/t) \sqrt{F_y}\,]$$

where $d' = d - 2t$, d = depth of the member, t = base thickness of material, and F_y = yield point of the steel. This reduction reflects the decrease in compression flange capacity resulting from the interaction of flange and local web buckling, and is based on research conducted at the University of Missouri-Rolla (LaBoube and Yu 1978). The reduction was previously used in the 1980 edition of the Specification (1980).

The influence of flange local buckling is quantified by the relationships R_1 and R_f, for both stiffened and unstiffened compression elements. These expressions recognize the degree of slenderness of the compression element, $(w/t)/S$. The parameter S represents the maximum width to thickness ratio, $(w/t)_{lim}$, for which the compression element is fully effective. This concept has been carried forward from the 1980 edition of the AISI Specification. Figures 1 and 2 show the general trend in moment reduction as a result of local buckling.

For stiffened compression elements, an additional modification, R_2, is given to account for the stiffening influence of the edge stiffener. The findings of the parametric studies revealed that the reduction in moment capacity due to flange local buckling can be conservatively estimated by linear relationships, as given in Section 3.2 of the Guide.

The elastic buckling moment (M_e) for discretely braced members is derived from a simplification of the lateral buckling provisions of Part 3.1.2 (a) of the 1989 Addendum. This simplification assumes that the value of GJ may be reasonably neglected and that the unbraced lengths for torsion and weak axis buckling are equal. A parametric study indicated that for reasonably proportioned members, $(C_w/I_y)^{1/2}$ is approximately equal to 0.42d for cees and 0.41d for zees. The result of this parametric study has been included in the equations. Calculation of the critical moment is taken directly from the 1989 Addendum. Local buckling effects are handled in the same manner as for fully braced members.

The accuracy of the equations in Section 3.2 and their tendency to yield conservative strength estimates is demonstrated by the ratio of Guide/Specification values given in Tables 1, 2 and 3.

STRENGTH FOR SHEAR

The Guide provisions for shear apply to unreinforced web elements, and are taken from the Specification. The Guide equations appear to differ from the Specification because the buckling coefficient $k = 5.34$ for unreinforced web elements has been incorporated in both the elastic buckling and inelastic buckling equations. This also impacts on the stated limits for the equations.

STRENGTH FOR COMBINED BENDING AND SHEAR

The Guide provision for combined bending and shear is taken directly from the Specification.

STRENGTH FOR WEB CRIPPLING

Because the intent of the Guide is to address the design for common commercial applications, the scope of the Guide's design provisions for web crippling has been significantly limited, as compared to the Specification. The equations in the Guide have been taken directly from the Specification, and reformatted for ease of use.

Figures 3 through 8 provide an indication of the web strength with variations in the key design parameters, d'/t, N, and t for the two most common load applications.

COMBINED BENDING AND WEB CRIPPLING

The Guide provisions for combined bending and web crippling have been taken directly from the Specification.

CONCENTRICALLY LOADED COMPRESSION MEMBERS

The guide provides methods for calculating the strength of columns considering flexural, torsional, and torsional-flexural buckling. Column unbraced lengths below L_{min} are subject to torsional-flexural buckling. The equation for L_{min} is based on Figure 9 which is taken from the research of Chajes, Fang, and Winter (1966).

The nominal stress equations for evaluating F_n are taken directly from the Specification for both elastic and inelastic flexural buckling.

The influence of local buckling on the nominal load capacity is accounted for by reduction factors that recognize the degree of slenderness of the compression elements, $(\Sigma w/t)/S$, and the overall slenderness of the member, $(L/r_y)/C_c$.

The element slenderness factor, R_3, accounts for the instability of all compression elements by using for w the sum of the flat widths of all compression elements, Σw, in the cross section: flanges, web and edge stiffeners. The influence of local buckling is demonstrated by the plot of $(\Sigma w/t)/S$ versus R_3 given by Figure 10.

The element slenderness factor, R_3, was developed assuming the cross section had yielded. However, as the overall column slenderness increases, the effective area will also increase. Thus, the overall slenderness factor, R_4, accounts for the increase in the cross-section's effective area as the overall column slenderness increases.

Torsional and torsional-flexural buckling is handled using the simplified method of Section C4.2 of the 1989 Addendum. σ_T equations were derived using the same parametric studies as those for discretely braced flexural members.

The accuracy of the equations in Section 3.7, and their tendency to yield conservative strength estimates is demonstrated by the ratio of Guide/Specification values give in Table 4.

COMBINED AXIAL LOAD AND BENDING

The guide provision for combined axial load and bending is taken directly from the specification.

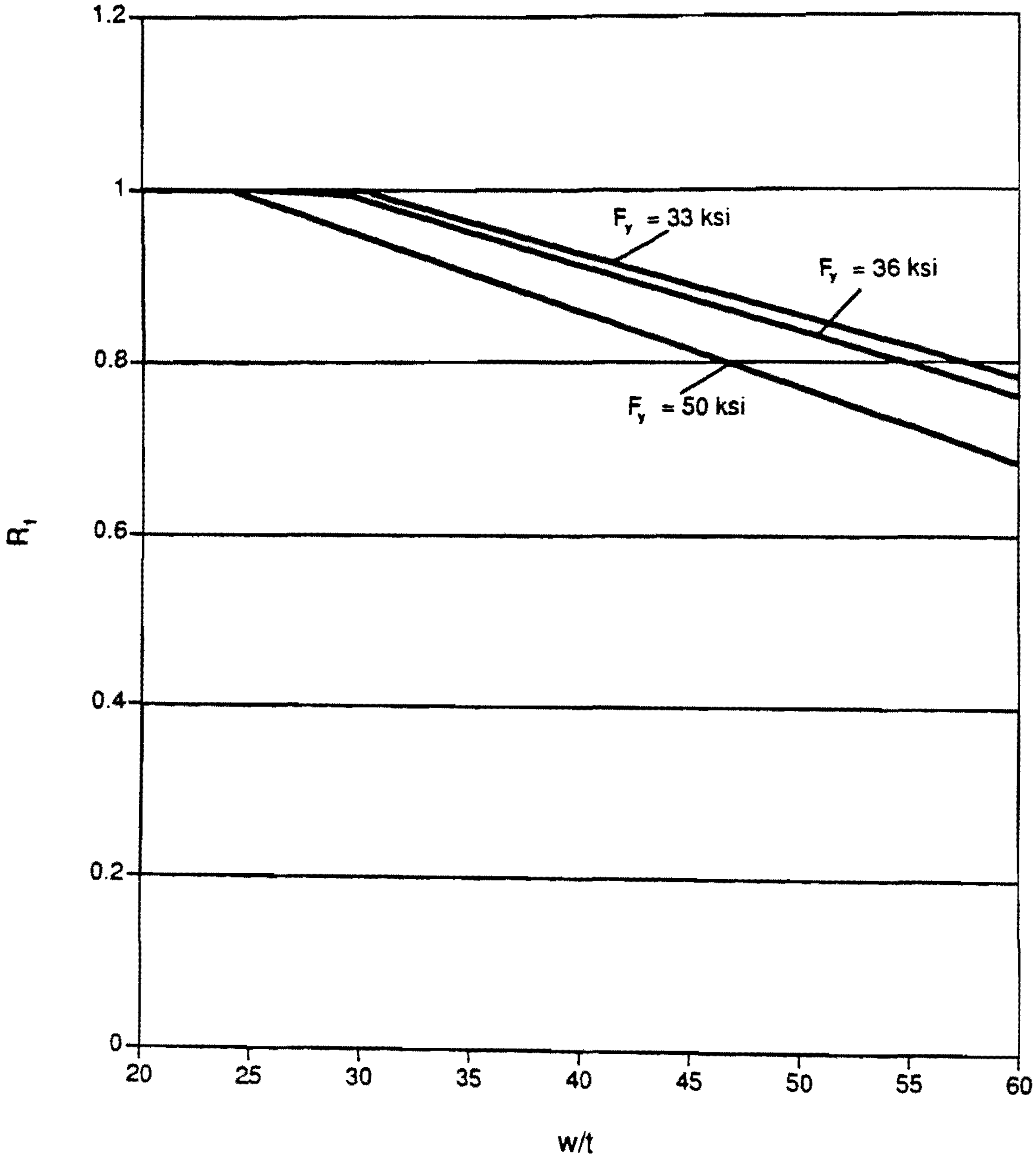


Fig. 1 – Relationship Between Bending Strength Reduction Factor R_1 and (w/t).

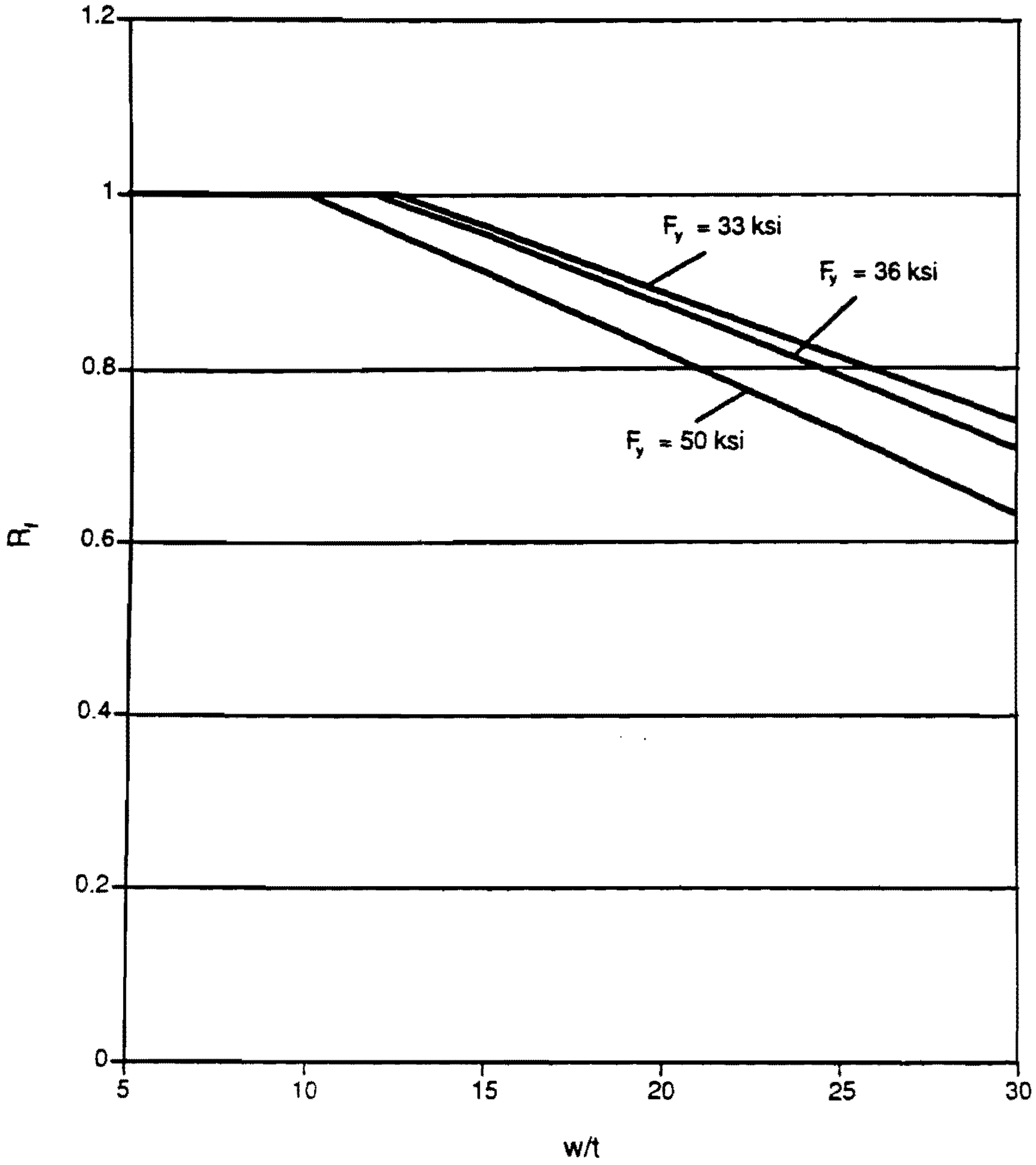


Fig. 2 – Relationship Between Bending Strength Reduction Factor R_f (Eq. 3.2-9) and (w/t).

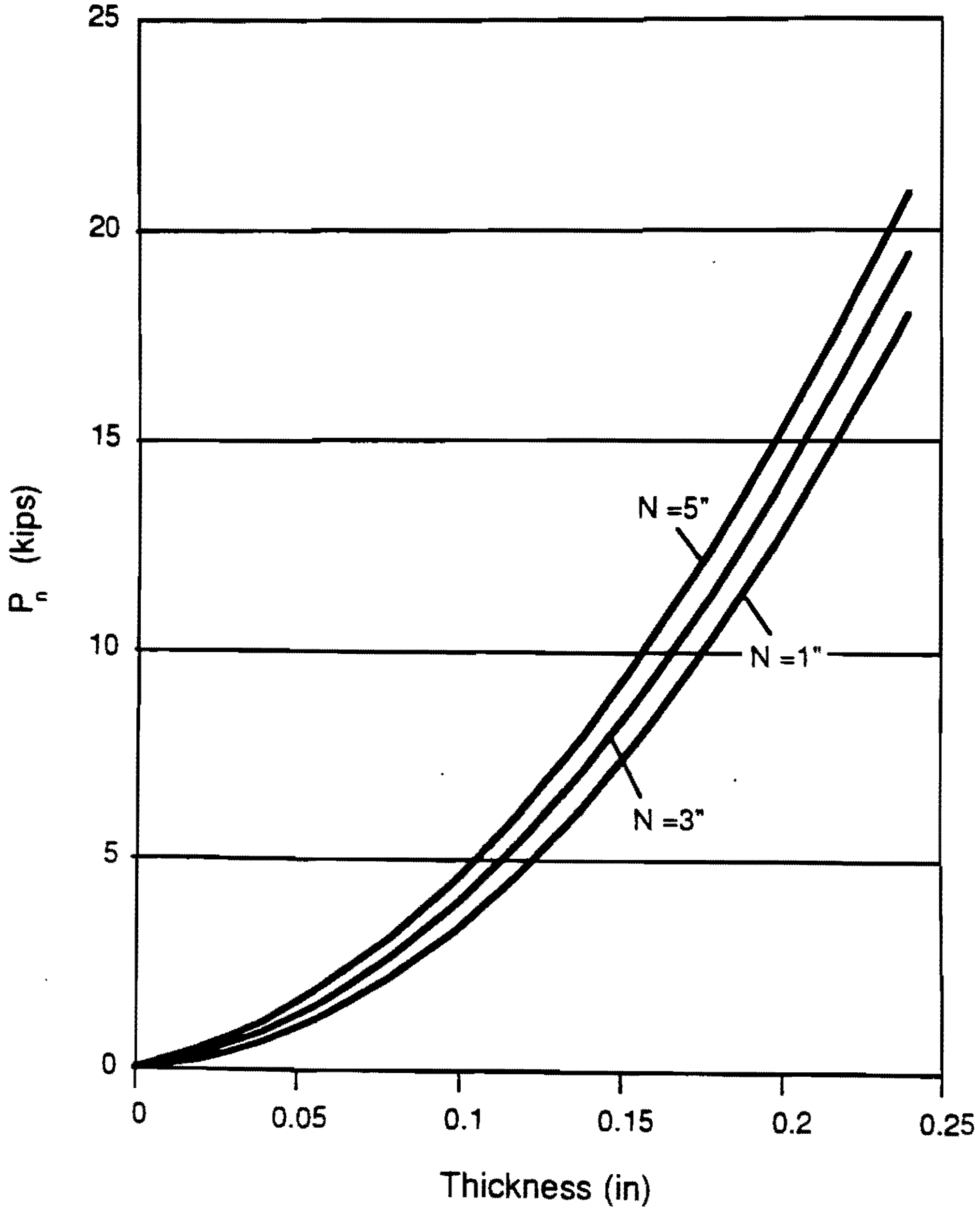


Fig. 3 – Web Crippling Using Eq. 3.5-2 for $d'/t = 50$ and $F_y = 33$ ksi.

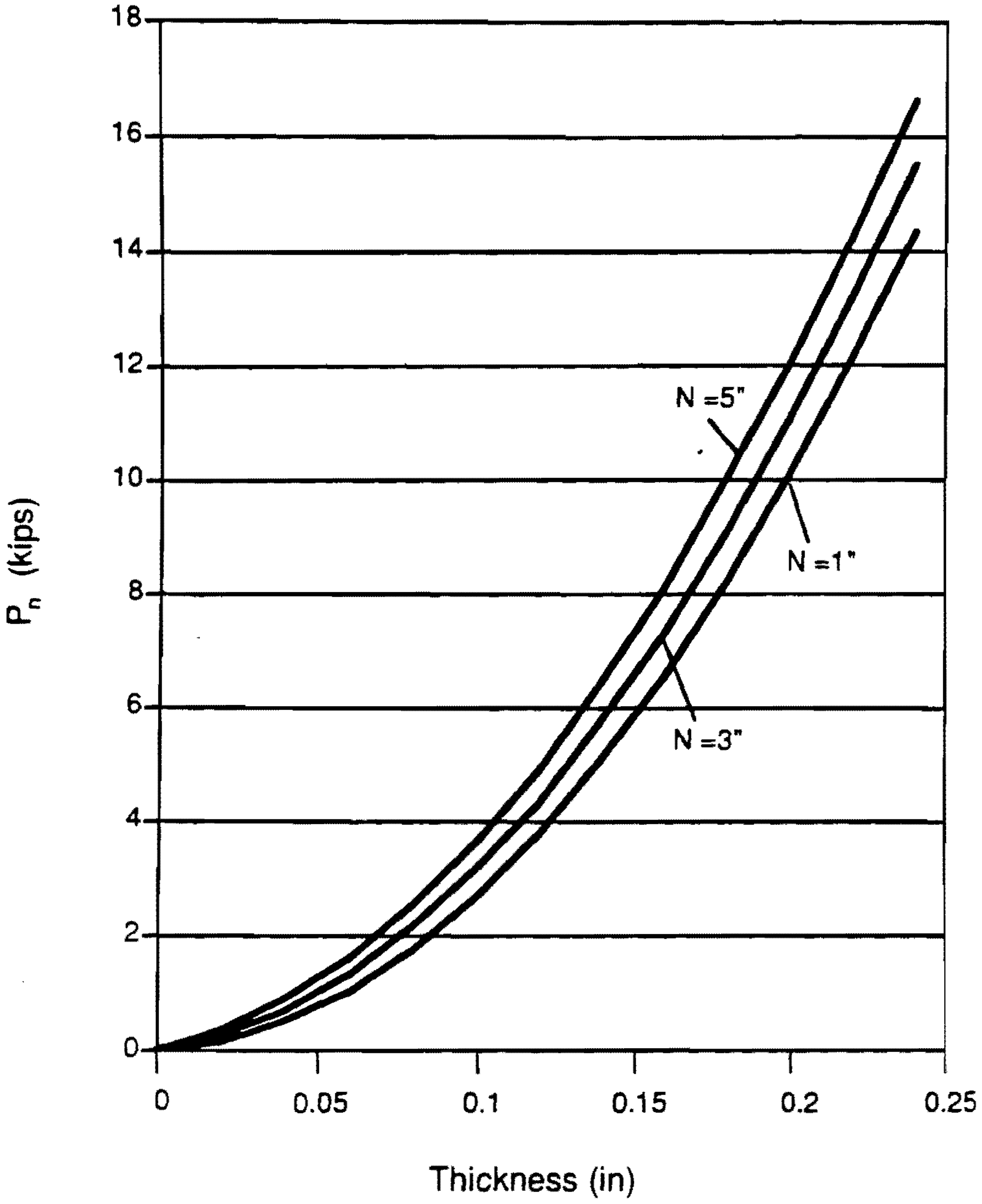


Fig. 4 – Web Crippling Using Eq. 3.5-2 for d'/t = 150 and F_y = 33 ksi.

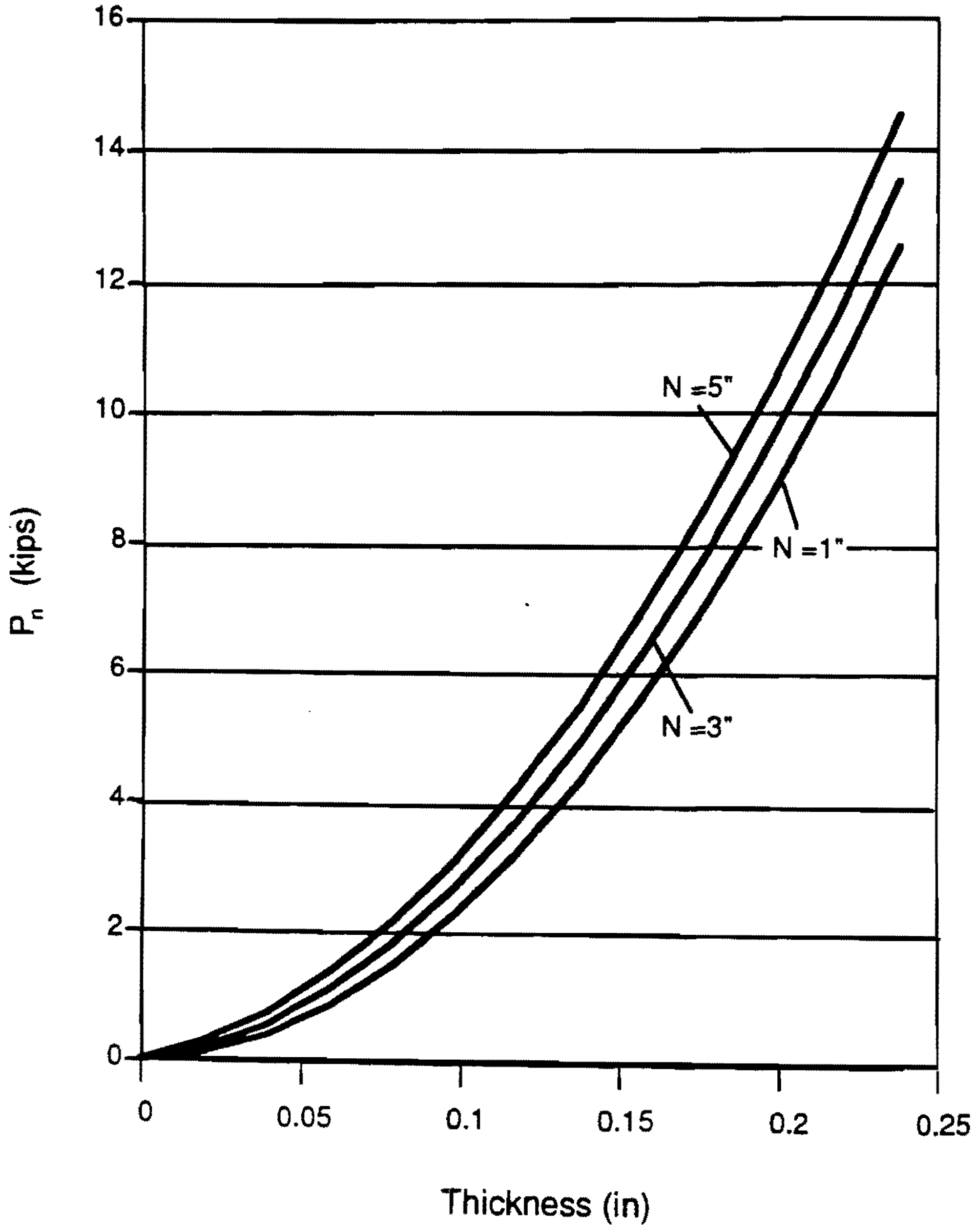


Fig. 5 – Web Crippling Using Eq. 3.5-2 for d'/t = 200 and F_y = 33 ksi.

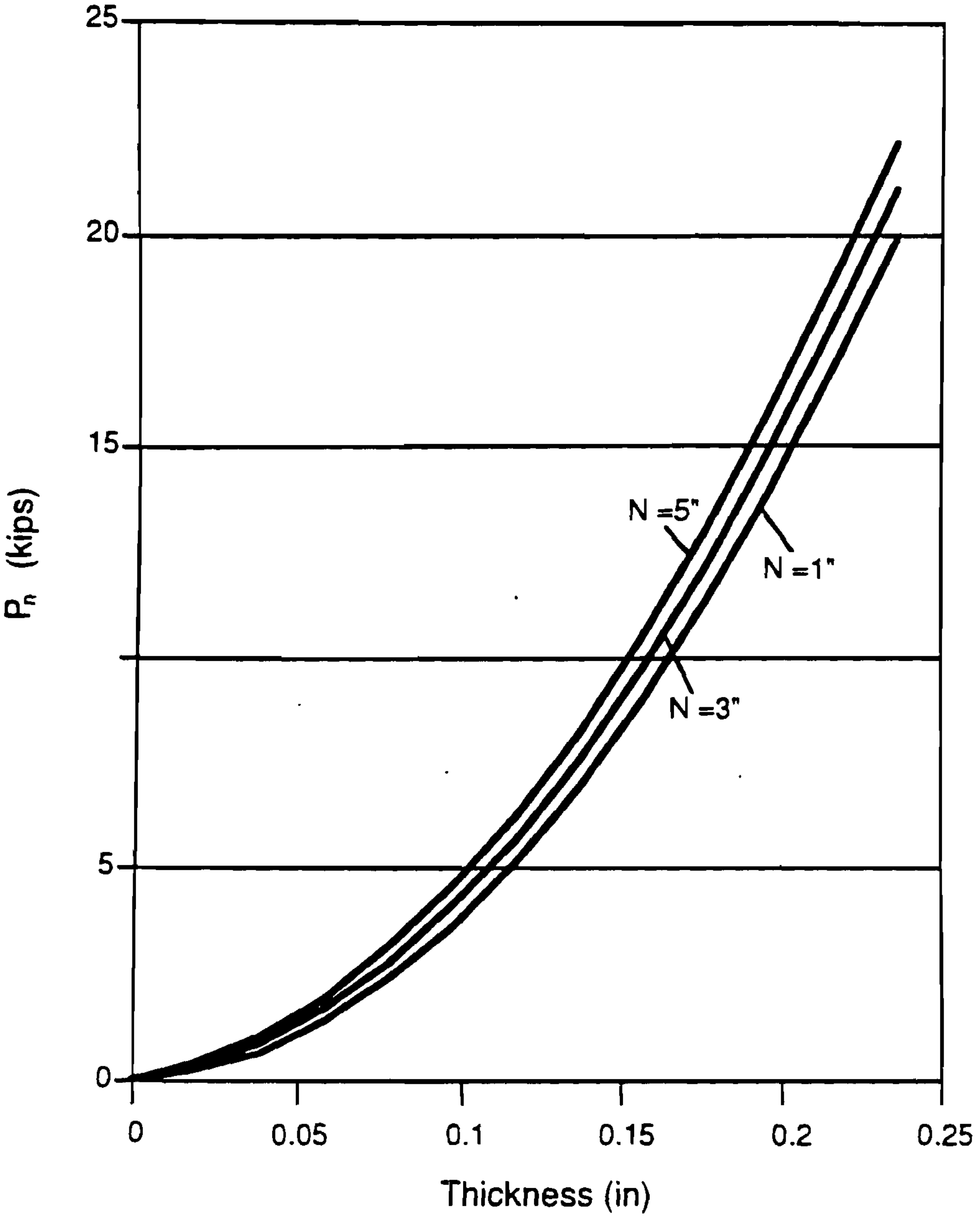


Fig. 6 – Web Crippling Using Eq. 3.5-4 for d'/t = 50 and F_y = 33 ksi.

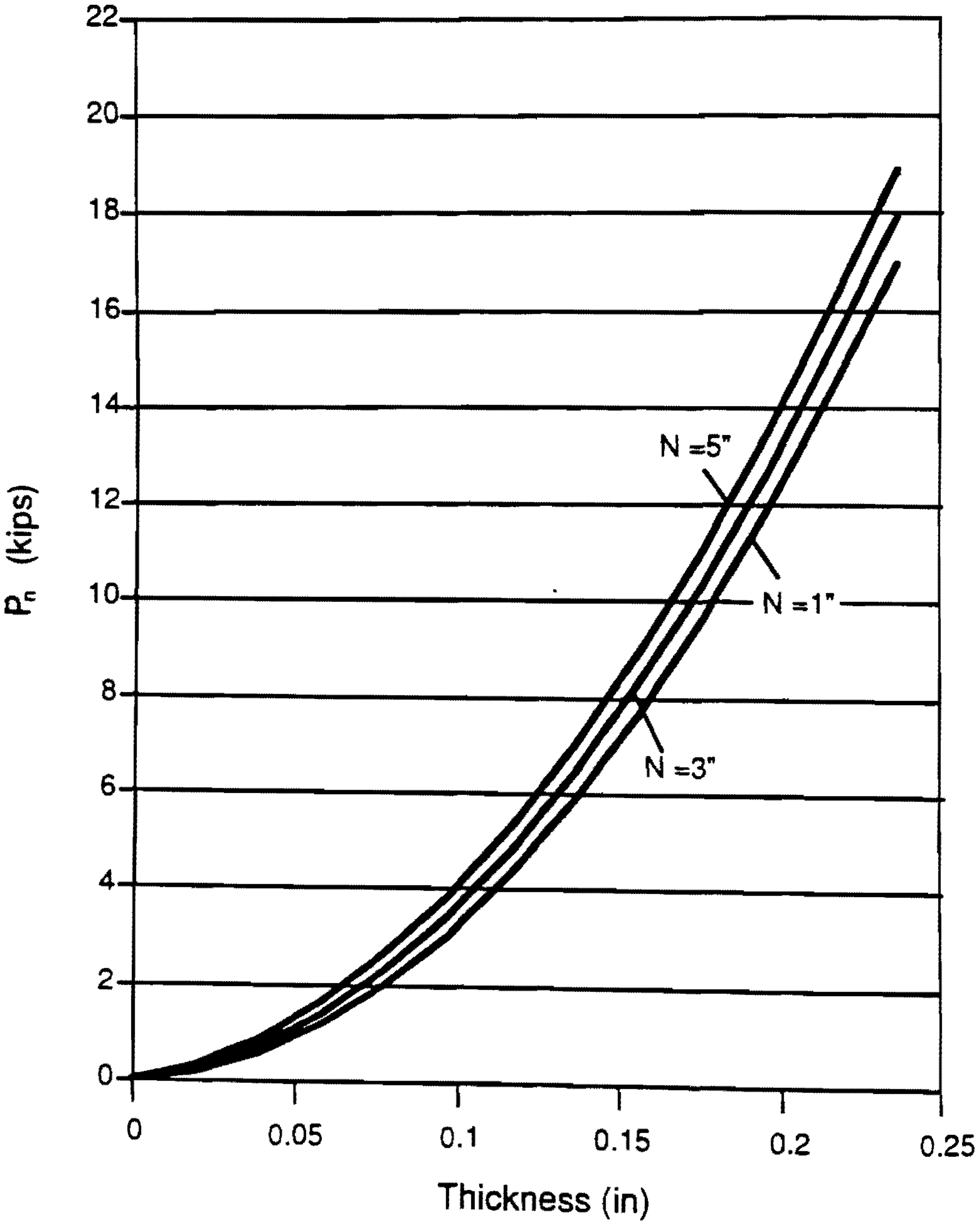


Fig. 7 – Web Crippling Using Eq. 3.5-4 for d'/t = 150 and F_y = 33 ksi.

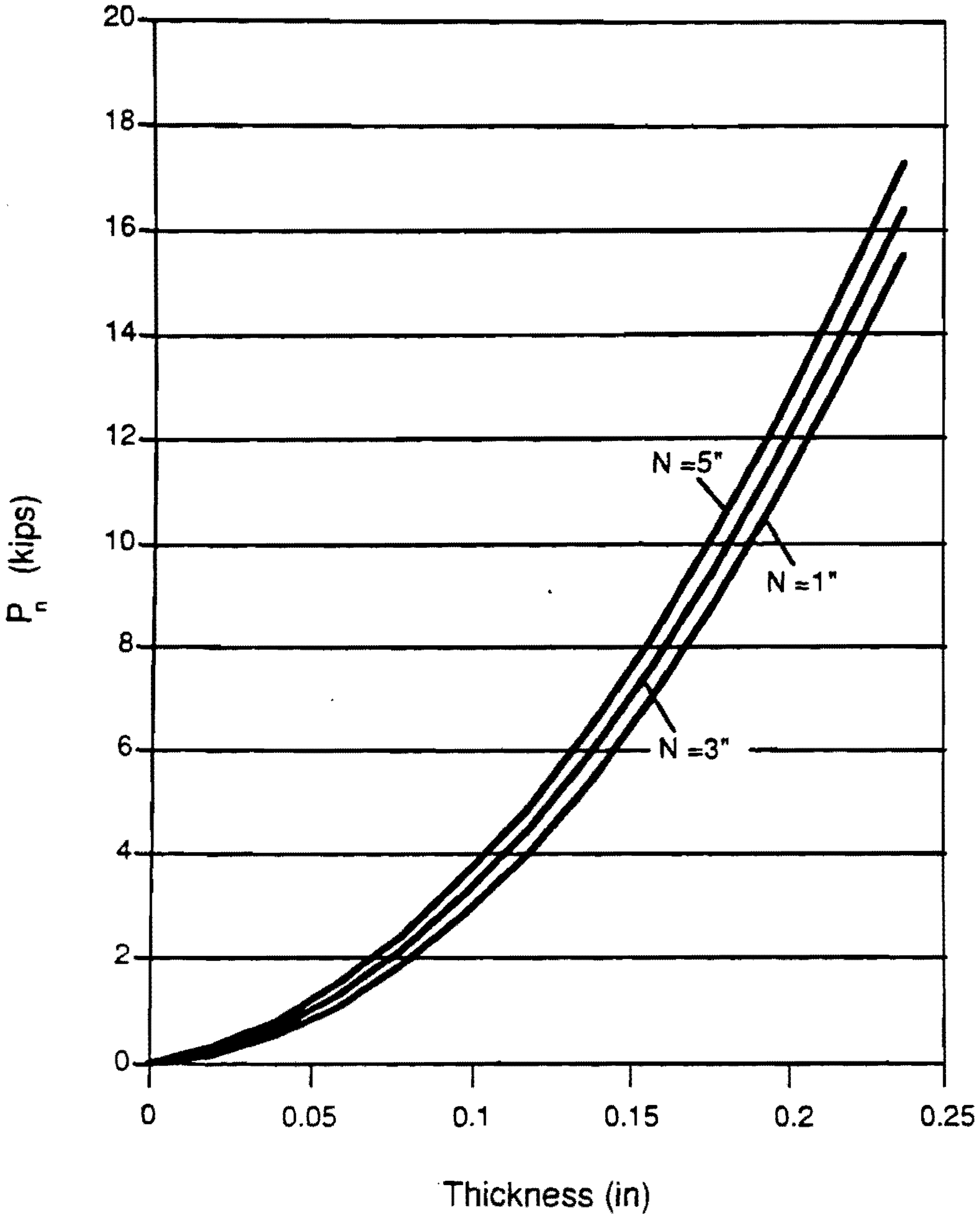
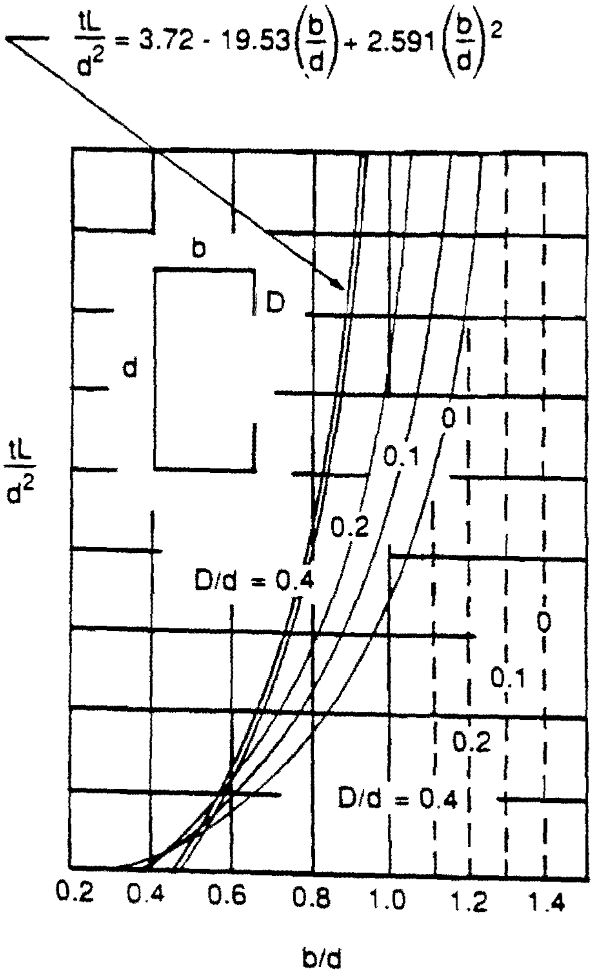


Fig. 8 – Web Crippling Using Eq. 3.5-4 for d'/t = 200 and F_y = 33 ksi.



$$\frac{tL}{d^2} = 3.72 - 19.53\left(\frac{b}{d}\right) + 2.591\left(\frac{b}{d}\right)^2$$

Fig. 9 – Influence of Section Geometry on Flexural Buckling of Axially Loaded Columns.

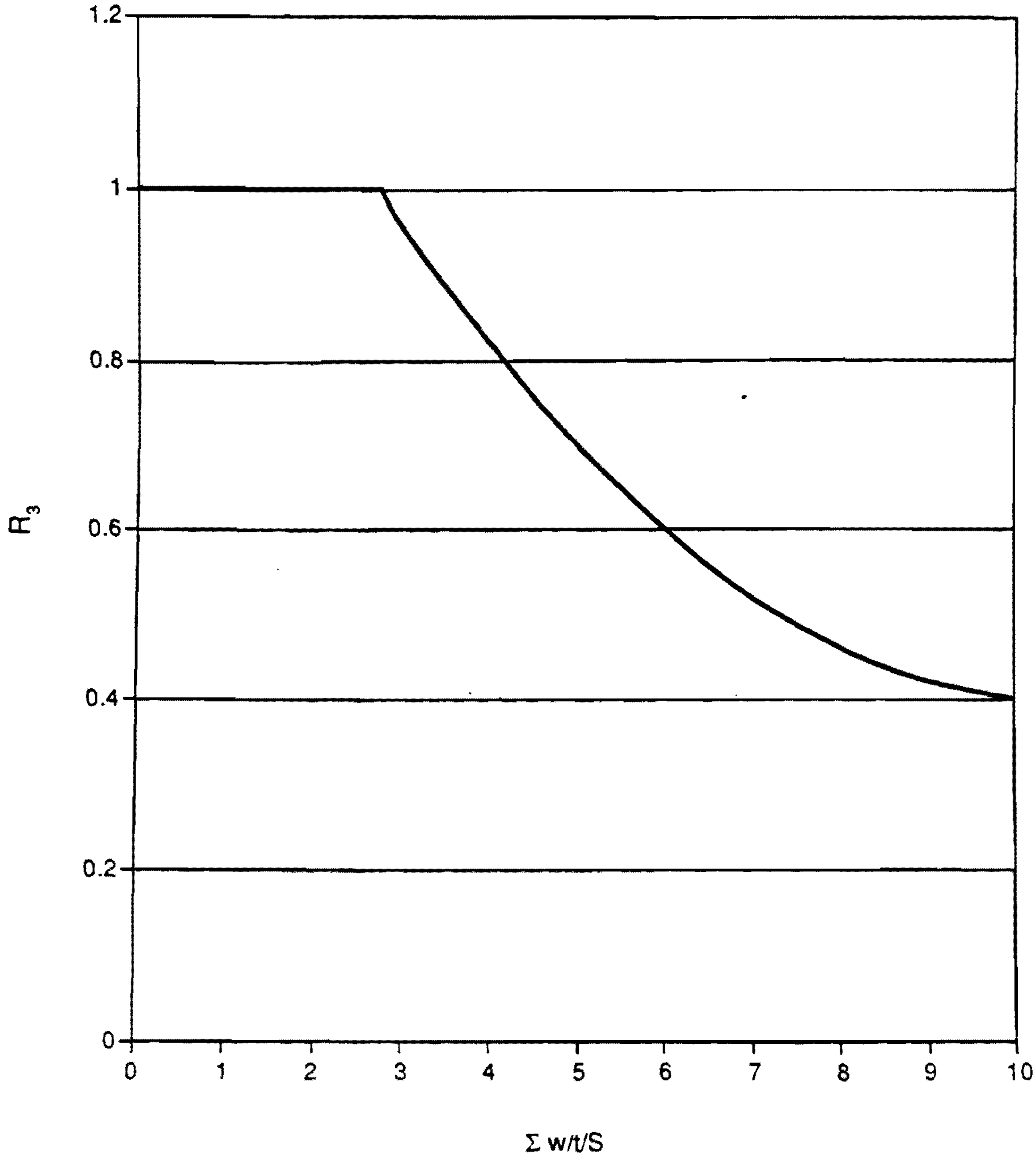


Fig. 10 – Reduction in Flexural Column Buckling Resulting from Local Buckling.

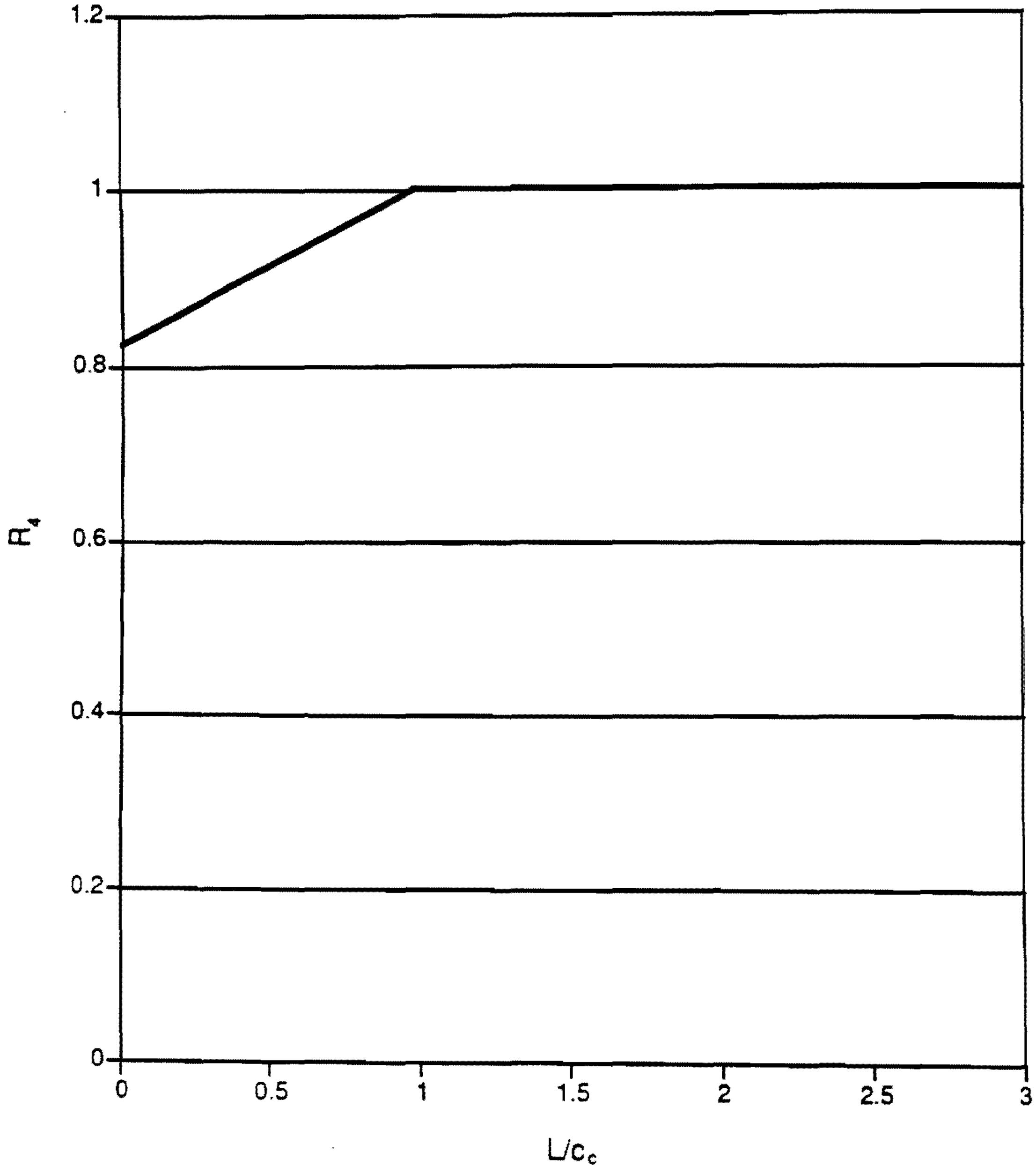


Fig. 11 – Influence of Column Slenderness on Local Buckling Behavior of Axially Loaded Columns.

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Chajes, A., Fang, P.J., and Winter, G. (1966), "Torsional Flexural Buckling, Elastic and Inelastic, of Cold Formed Thin Walled Columns," Cornell Engineering Research Bulletin 66-1, Cornell University.

LaBoube, R. A., and Yu, W. W., "Bending Strength of Webs of Cold-Formed Steel Beams," Journal of the Structural Division, Vol. 108, No. ST 7, July 1982, American Society of Engineers, pp. 1589-1604.

TABLE 1
COMPARISON BETWEEN AISI SPECIFICATION AND DESIGN GUIDE FOR SECTIONS WITH EDGE-STIFFENED COMPRESSION FLANGE
(Section 3.2)

Section (dxbxt)	Nominal Capacity (in-Kips) Specification	Guide	Guide / Specification
Channel Sections, $F_y = 33$ksi			
8 x 2.5 x .04	32.77	29.30	0.895
8 x 2.0 x .04	31.75	28.40	0.898
8 x 1.625 x .04	31.71	27.30	0.865
6 x 1.625 x .04	21.91	20.30	0.930
6 x 2.0 x .04	22.47	21.40	0.956
6 x 2.5 x .04	23.73	22.40	0.945
4 x 2.0 x .04	13.00	12.60	0.977
8 x 1.625 x .058	47.49	44.58	0.944
8 x 2.0 x .058	51.61	46.70	0.905
8 x 2.5 x .085	52.80	47.00	0.890
Z-Sections, $F_y = 33$ksi			
8 x 2.625 x .06	62.90	52.29	0.943
8 x 2.625 x .08	88.61	88.61	1.000
8 x 2.65 x .107	119.30	119.30	1.000
9.5 x 2.75 x .06	79.50	74.74	0.940
9.5 x 2.75 x .071	103.03	98.12	0.952
Z-Sections, $F_y = 55$ksi			
9.5 x 2.75 x .061	113.80	103.51	0.910
9.5 x 2.75 x .061	154.11	142.11	0.922
9.5 x 2.75 x .079	174.19	168.37	0.967
9.5 x 2.75 x .084	194.32	187.07	0.963
9.5 x 2.75 x .098	237.11	230.12	0.971

TABLE 2
COMPARISON BETWEEN AISI SPECIFICATION AND DESIGN GUIDE FOR SECTIONS WITH UNSTIFFENED COMPRESSION FLANGE
(SECTION 3.2)

Section (dxbxt)	Nominal Capacity (in-Kips) Specification	Guide	$\frac{\text{Guide}}{\text{Specification}}$
Channel Sections, F_y = 33ksi			
2.5 x 1.375 x .04	3.96	3.99	1.008
3.5 x 1.0 x .04	5.93	5.26	0.887
4.0 x 1.0 x .04	7.25	7.16	0.988
5.0 x 1.0 x .04	10.20	9.65	0.946
6.0 x 1.0 x .04	13.60	12.09	0.889
7.0 x 1.0 x .04	17.43	14.56	0.835
8.0 x 1.0 x .04	19.98	17.00	0.851
8.0 x 1.375 x .04	20.09	16.68	0.830
4.0 x 1.0 x .07	13.69	13.69	1.000
Channel Sections, F_y = 55ksi			
4.0 x 1.0 x .07	22.21	22.23	1.001
4.0 x 1.5 x .07	25.03	24.52	0.980

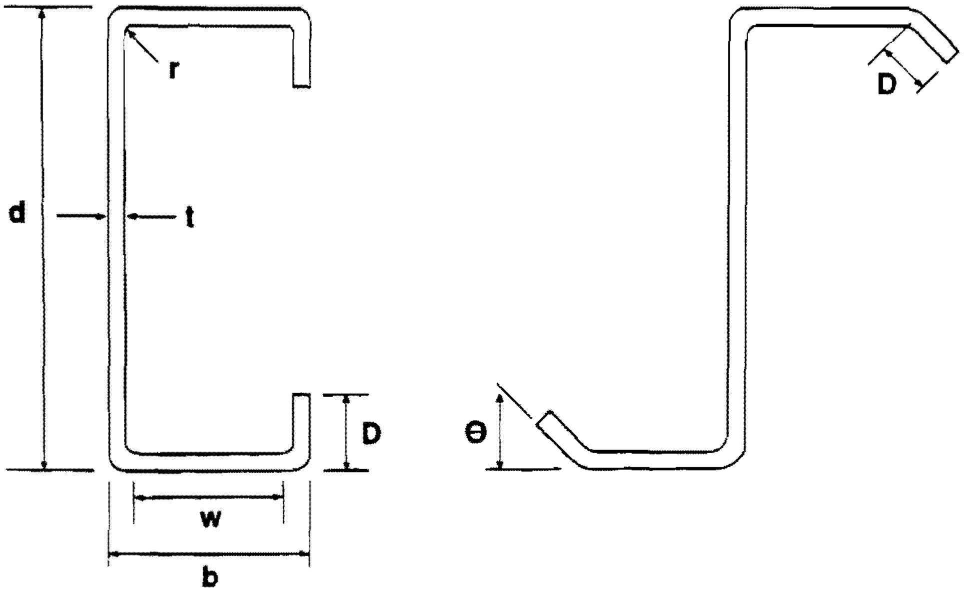
TABLE 3
COMPARISON BETWEEN AISI SPECIFICATION AND DESIGN GUIDE FOR DISCRETELY BRACED FLEXURAL MEMBERS
(Section 3.2)

Section (dxbxDxt)	L_b (ft)	M_c Specification	M_c Guide	Guide / Specification
		(in kips)		
Channel Sections				
12 x 3 . 5 x 0 . 9 x 0 . 105 , F_y = 55 ksi				
	1.0	400.6	400.5	0.999
	5.0	371.8	369.8	0.995
	9.0	305.9	298.0	0.974
	12.0	233.8	217.2	0.929
	20.0	91.4	78.2	0.855
12 x 3 . 5 x 0 . 9 x 0 . 105 , F_y = 33 ksi				
	1.0	240.7	240.4	0.999
	5.0	230.3	229.6	0.997
	9.0	206.6	203.7	0.986
	12.0	180.6	174.6	0.987
	20.0	91.4	78.2	0.855
8 x 3 . 0 x 0 . 7 x 0 . 075 , Fy = 50 ksi				
	1.0	137.5	137.5	1.000
	5.0	125.3	124.5	0.994
	9.0	97.6	94.0	0.969
	12.0	67.7	60.9	0.899
Zee Sections				
12 x 3 . 5 x 0 . 9 x 0 . 105 , Fy = 55 ksi				
	1.0	400.1	399.9	0.999
	5.0	359.8	356.0	0.989
	9.0	266.9	253.2	0.949
	12.0	170.1	152.8	0.898
	20.0	63.8	50.0	0.783
5 x 2 . 0 x 0 . 5 x 0 . 048, Fy = 55 ksi				
	1.0	39.2	39.2	1.000
	5.0	28.0	27.3	0.975
	9.0	10.7	9.8	0.916
	12.0	6.2	5.5	0.887
5 x 2 . 0 x 0 . 5 x 0 . 048 , Fy = 33 ksi				
	1.0	23.7	23.6	0.996
	5.0	19.6	19.3	0.985
	9.0	10.7	9.8	0.915
	12.0	6.2	5.5	0.887

TABLE 4
COMPARISON BETWEEN AISI SPECIFICATION AND DESIGN GUIDE FOR COMPRESSION MEMBERS
(SECTION 3.7)

Section (dxt)	Unbraced Length (In.)	Nominal Capacity (in-Kips) Specification (Kips)	Guide (Kips)	Guide / Specification
Fy = 33ksi				
8.0C.06	96	17.45	14.70	0.842
	144	14.47	11.82	0.817
9.5C.06	96	17.51	14.54	0.831
	144	14.44	11.57	0.801
8.0C.12	48	52.48	50.67	0.965
	72	50.52	49.36	0.977
	96	47.03	46.65	0.992
	144	36.68	36.86	1.005
9.5C.12	48	53.40	52.75	0.988
	96	47.70	48.27	1.012
	144	36.98	37.64	1.018
2.5C.05	96	3.68	3.68	1.000
6.0C.05	96	6.56	5.84	0.890
8.0C.05	96	8.80	7.79	0.885
10.0C.05	96	8.69	7.51	0.865
Fy = 55ksi				
8.0C.06	96	22.43	17.41	0.776
9.5C.06	96	22.39	17.27	0.771
	144	14.95	10.10	0.676
8.0C.07	96	27.98	23.46	0.839
8.0C.12	48	74.57	73.03	0.979
	96	61.87	60.88	0.984
	144	38.79	35.27	0.909
9.5C.12	96	62.19	61.65	0.991
	1 44	38.60	34.90	0.900
11.0C.12	96	62.09	61.81	0.996

Appendix A
Example Problems



Cross-Sectional Properties

$a = d - t$

$c = D - \frac{t}{2}$ (lipped)

$c = 0$ (no lips)

$B = b - t$ (lipped)

$B = b - \frac{t}{2}$ (no lips)

$A = a + 2B + 2c$

Channel:

$$I_x = t\left\{\frac{a^3}{12} + \frac{c^3}{6} + \frac{Ba^2}{2} + \frac{c}{2}(a - c)^2\right\}$$

$$I_y = t\left\{\frac{B^3}{2} + 2cB^2 + \frac{B^3}{6} - \frac{1}{A}(B^2 + 2CB)^2\right\}$$

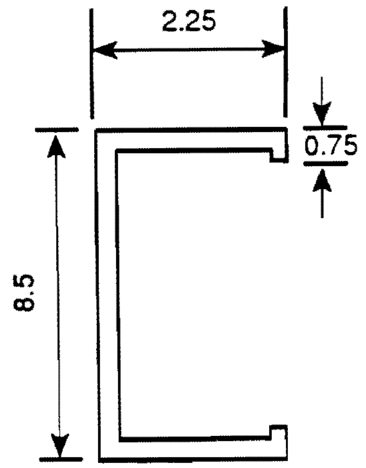
Zee:

$$I_x = t\left\{\frac{a^3}{12} + \frac{Ba^2}{2} + \frac{(c \sin \theta)^3}{6} + \frac{c}{2}(a - c \sin \theta)^2\right\}$$

$$I_y = t\left\{\frac{2B^3}{3} + \frac{(c \cos \theta)^3}{6} + 2c\,(B + \frac{c}{2} \cos \theta)^2\right\}$$

Fig. A1 – Typical Cross-Section Dimensions

Example Problem No. 1 – Estimate Flexural Capacity of Stiffened Cee



r = 0.25 in.

t = 0.075 in.

F_y = 50 ksi

Estimate flexural capacity if member is fully braced and braced at 4 ft and 8 ft intervals. Assume $C_b = 1.0$.

(a) L_b = 0 ft, fully braced

$S_f = 2.458\ \text{in}^3$

$d' = 8.50 - 2(0.075) = 8.350$ in.

$d'/t = 111.33$

$F_n = [1.21 - 0.00034(D'/t)\sqrt{F_y}]\,F_y \leq F_y$

$= [1.21 - 0.00034(111.33)\sqrt{50}]50 = 47.12\ \text{ksi} < 50\ \text{ksi}$

$= 47.12$ ksi

$w = 2.25 - 2(0.075 + 0.25) = 1.60$ in.

$w/t = 21.33 < 60$ ok

$S = 1.28\sqrt{E/F_y} = 1.28\sqrt{29500/50} = 31.09$

$w/t/S = 0.686$

$R_1 = 1.227 - 0.284[(w/t)/S] \leq 1.0$

$= 1.227 - 0.284[0.686] = 1.032 > 1.0$

$= 1.0$

$d/b = 0.75/2.25 = 0.33$ and $F_n < F_y$, $R_2 = 1.0$

$R_f = R_1 R_2 = 1.0$

$M_n = F_n S_f R_f = 47.12\ \text{ksi}\ (2.458\ \text{in}^3)(1) = 115.82$ Kip-in

$M_a = M_n/1.67 = 69.35$ Kip-in

(b) $L_b = 4.0\ \text{ft} = 48$ in

$I_y = 0.488\ \text{in}^4$

$M_e = 0.42\,\pi^2 E\, C_b\, d\, I_y / L^2$

$= 0.042\pi^2 (29500) (1.0) (8.50) (0.488) / (48)^2 = 220.15$ in – k

$M_y = S_f F_y = 2.458 (50) = 122.90$ in - k

$M_e > 0.5 M_y$ therefore

$M_c = M_y [1.0 - M_y / (4M_e)]$

$= 122.90 [1.0 - 122.90 / (4) (220.15)] = 105.75$ in – k

$R_w = 1.21 - 0.00034 (d'/t) \sqrt{Fy}$

$= 1.21 - 0.00034 (111.33) \sqrt{50} = 0.942$

$R_f = 1.0$

$M_n = R_w M_c R_f$

$= 0.942 (105.75) (1.0) = 99.62$ in - k

$M_a = M_n / 1.67 = 59.65$ in - k

(c) $L_b = 8.0$ ft = 96 in

$M_e = 0.42\pi^2 (29500) (1.0) (8.50) (0.488) / (96)^2 = 55.04$ in – k

$M_e \leq 0.5 M_y$ therefore

$M_c = M_e = 55.04$ in - k

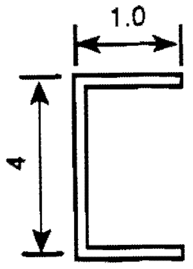
$R_w = 0.942$

$M_n = R_w M_c R_f$

$= 0.942 (55.04) (1.0) = 51.85$ in -k

$M_a = M_n / 1.67 = 31.04$ in -k

Example Problem No. 2 – Estimate Flexural Capacity of Unstiffened Cee



r = 0.25 in.

t = 0.075 in.

F_y = 50 ksi

Estimate flexural capacity if member is fully braced and braced at 2 ft and 4 ft intervals. Assume $C_b = 1.0$.

(a) L_b = 0 ft., fully braced

$S_f = 0.403\ \text{in}^3$

$d' = 4 - 2\,(0.075) = 3.86$ in.

$d'/t = 55.14$

$F_n = [1.21 - 0.00034\,(d'/t)\sqrt{F_y}]F_y \le F_y$

$= [1.21 - 0.00034\,(55.14)\sqrt{50}]50 = 53.87 > 50$

$= 50$ ksi

$w = 1.0 - (0.25 + 0.075) = 0.68$in.

$w/t = 9.71 < 30$ ok

$S = 0.42\sqrt{E/F_y} = 0.42\sqrt{29500/50} = 10.20$

$R_f = \ = 0.19[(w/t)/S] + 1.19 \le 1.0$

$= 1.119 - 0.19[9.71/10.2] = 1.01 > 1.0$

$= 1.0$

$M_n = F_n\,S_f\,R_f$

$= (50\ \text{ksi})(0.403\ \text{in}^3)\ (1) = 20.15$ kip-in.

$M_a = 12.07$ Kip-in

(b) L_b = 2.0 ft = 24 in.

$I_y = 0.030\ \text{in}^4$

$M_e = 0.42\,\pi^2\,E\,C_b\,d\,I_y\,/\,L^2$

$= 0.42\,\pi^2\,(29500)\,(1.0)\,(4)\,(0.030)\,/\,(24)^2 = 25.47$ in – k

$My = S_f Fy = 0.403 (50) = 20.15$ in - k

$M_e > 0.5 M_y$ therefore

$M_c = M_y [1.0 - M_y / (4M_e)]$

$= 20.15 [1.0 - 20.15 / 4(25.47)] = 16.16$ in - k

$R_w = 1.21 - 0.00034 (d'/t) \sqrt{F_y}$

$= 1.21 - 0.00034 (55.14) \sqrt{50} = 1.077 \leq 1.0$

$R_w = 1.0$

$R_f = 1.0$

$M_n = R_w M_c R_f$

$= 1.0 (16.16) (1.0) = 16.16$ in - k

$M_a = M_n / 1.67 = 9.68$ in - k

(c) $L_b = 4.0$ ft = 48 in

$M_e = 0.42 \pi^2 (29500) (1.0) (4) (0.030) / (48)^2 = 6.37$ in - k

$M_e \leq 0.5 M_y$ therefore

$M_c = M_e = 6.37$ in - k

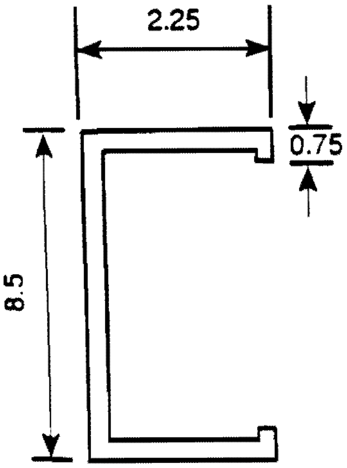
$R_w = 1.0$

$M_n = R_w M_c R_f$

$= 1.0 (6.37) (1.0) = 6.37$ in - k

$M_a = M_n / 1.67 = 3.81$ in - k

Example Problem No. 3 – Estimate Axial Load Capacity of Stiffened Cee



$r = 0.25$ in.

$t = 0.075$ in.

$F_y = 50$ ksi

Estimate the axial load capacity, P_a
Assume pinned ends and member length, L, is 8 feet.
$b/d = 0.26 < 0.4$, No lower limit on member length

$$\frac{KL}{r_y} = \frac{(1)(8x12)}{0.787} = 122$$

$$F_e = \frac{\pi^2 E}{(Kl/r_y)^2} = \frac{\pi^2(29500)}{(122)^2} = 19.57 \text{ ksi}$$

$$F_y/2 = 25 \text{ ksi} > F_e, F_n = F_e$$

$$w_D = 0.75 - (0.25 + 0.75) + 0.43 \text{ in.}$$

$$w_b = 2.25 - 2(0.25 + 0.75) = 1.60 \text{ in.}$$

$$w_d = 8.50 - 2(0.25 + 0.75) = 7.85 \text{ in.}$$

$$\Sigma w = (0.43 + 1.60)\,2 + 7.85 = 11.91 \text{ in.}$$

$$S = 1.28\sqrt{E/F_y} = 1.28\sqrt{29500/50} = 31.09$$

$$C_c = \sqrt{2\pi^2 E/F_y} = \sqrt{2\pi^2(29500)/50} = 107.9$$

$$\begin{aligned} R_3 &= 1.50 - 0.21[(\Sigma w/t)/S] + 0.01\,[(\Sigma w/t)/S]^2 \leq 1.0 \\ &= 1.50 - .21[11.91/0.75/31.09] + 0.01\,[11.91/.075/31.09]^2 \\ &= 0.69 < 1.0 \\ &= 0.69 \end{aligned}$$

$$\begin{aligned} R_4 &= 0.175\,[(KL/ry)/C_c] + 0.825 \leq 1.0 \\ &= 0.175[122/107.9] + .825 = 1.02 > 1.0 \\ &= 1.0 \end{aligned}$$

$$R_c = R_3 R_4 = 0.69$$

$$P_n = F_n A_g R_c = (19.57 \text{ ksi})(1.028)(0.69) = 13.88 \text{ kips}$$

$$P_a = P_n/1.92 = 7.23 \text{ kips}$$